York City—Feb. 18-21, 1946

JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

(ACI PROCEEDINGS Vol. 42)

Vol. 17 | September 1945 | No. 1

CONTENTS

Papers and Reports 1-84

Concrete Construction in the National Forests .... CLIFFORD A. BETTS 1

Lapped Bar Splices in Concrete Beams .... RALPH W. KLUGE and EDWARD C. TUMA 13

Tests of Prestressed Concrete Pipes Containing A Steel Cylinder .... CULBERTSON W. ROSS 37

Field Use of Cement Containing Vinsol Resin ...... CHARLES E. WUERPEL 49

Job Problems and Practice 85-92

Non-Skid Concrete Surfacing for Wooden Trestles at Mud Mountain Dam .... H. H. ROBERTS 85

Exposure of Concrete to High Temperature 88

Derrick Stone and Cobbles in Mass Concrete 89

Effect of Brine on Concrete 90

Current Reviews 93-104

News Letter 1-12

Report of Nominating Committee ● New ACI Charter ● New Members ● Who's Who ● Honor Roll ●

to provide a comradeship in finding the best ways to do concrete work of all kinds and in spreading that knowledge

ADDRESS: 7400 SECOND BOULEVARD, DETROIT 2, MICH.

$7.50 by the year

$1.50 per copy

DISCUSSION

Discussion closes March 1, 1946

Concrete Construction in the National Forests—Clifford A. Betts

Lapped Bar Splices in Concrete Beams—Ralph W. Kluge and Edward C. Tuma

Tests of Prestressed Concrete Pipes Containing A Steel Cylinder—Culbertson W. Ross

Field Use of Cement Containing Vinsol Resin—Charles E. Wuerpel

Vol. 17—No. 1 September 1945 (Proceedings Vol. 42)

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

Published by the American Concrete Institute. The Institute was founded 1905; incorporated in the District of Columbia in 1906 as The National Association of Cement Users; the name changed 1913 by charter amendment; reincorporated, with new statement of objects, August 8, 1945. The Journal is issued six times yearly in the months of January, February, April, June, September and November under the authority of the

Papers and other contributions prepared with a view to Journal publication should be submitted in triplicate, addressed: Secretary, Publications Committee, American Concrete Institute, 7400 Second Boulevard, Detroit 2, Michigan. Publication of a contribution does not imply the acquiescence of critics (whose advice is sought by the Committee, prior to acceptance) or of the Institute in the opinions which it expresses nor the approval of data or practice which the contribution records. Institute authority attaches only to Standards formally adopted as provided in the By-Laws. A committee report implies merely the joint contribution of an appointed group.

Subscription price $7.50 per year payable in advance. To members, $7.50 per year, included in the annual dues. (A special dues rate of $3.00 per year applies for "a student in residence at a recognized technical or engineering school" and includes Journal subscription.

Bound volumes 1 to 40 of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE (1905 to 1944) are for sale as far as available, at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

Publication address: 7400 Second Boulevard, Detroit 2, Michigan. Printed in U. S. A. Entered at the Post Office at Detroit, Michigan, as mail of the second class under provisions of the Act of March 3, 1879.

the ACI JOURNAL

is edited by the Secretary of the Publications Committee under the direction of the Committee

It is the policy of the American Concrete Institute to encourage participation by its members and others in the work of extending the knowledge of concrete and reinforced concrete as a basis for improved products and structures.

To this end the Board of Direction has assigned to the Publications Committee the responsibility of selecting for publication such papers, committee reports, discussions and other contributions or parts of such contributions, as in the judgment of the Committee, seem to offer most of value in attaining Institute objectives within space requirements consistent with budget limitations.

AMERICAN CONCRETE INSTITUTE

NEW CENTER BUILDING
DETROIT 2, MICHIGAN

PLEASE NOTE

1 As this JOURNAL issue goes tardily to press (acute manpower shortage at our printers) the Board of Direction is holding its first postwar meeting in Chicago, and out of it may come important plans for the future, including a full-blown 1946 Convention, New York City, Feb. 18-21, 1946.

2 Many readers (in spite of repeated announcements apparently are unaware of the availability of separate prints of each paper and report. Many who are aware of their availability have not been aware that the Institute, organized and financed as it is, is not in a position to make free distribution of its literature. See the new announcement which tops the first page of each paper and report—any one of them in the current issue may be had at 25 or 50 cents each. In quantities the prices are lower—for large quantities much lower.

3 Many papers and discussions are submitted for consideration of the Publications Committee in a single copy of the manuscript. Three copies are required. In fact all prospective contributors should have a copy of "American Concrete Institute Publications Policy" (an 8-page reprint from the September 1941 Journal). It will be sent without charge, on request.

4 Discussion closes March 1, 1946 on papers published in this issue.

[To facilitate selective distribution, **separate prints** of this title (42-1) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Mar. 1, 1946]

Title 42-1 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
(copyrighted)

Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **September 1945**

Concrete Construction in the National Forests*

By CLIFFORD A. BETTS†

SYNOPSIS

How U. S. Forest Service applies the fundamentals of good concrete without elaborate control measures to countless small, isolated jobs in the 175,000,000 acres of National forests and along 100,000 miles of roads serving these areas, with pictures to show some of the variety of the work done.

Concrete technique on comparatively small jobs scattered over the 175,000,000 acres of the National Forests or along the 100,000 miles of roads that serve these areas, must of necessity be radically different from that used on a large concentrated dam project or on urban programs near transportation facilities and central mixing plants. The total yardage placed on a large number of minor projects may, and generally does, exceed that of spectacular big jobs. The total value of the small buildings, bridges, and other structures may transcend that of the more publicized ones, yet the opportunity for technical refinements and special equipment is with the big job. Add to this the fact that periodic inspection and maintenance are more common on the major installations, and it becomes evident that on far-flung work programs such as those of the National Forests, the training of hundreds of scattered workers to build durable, fool-proof structures is as difficult as it is desirable.

Nevertheless, progress is being made even on small jobs and discoveries made on the large projects are being applied with success. High-early-strength cement, internal vibrators, moisture control, absorptive form lining, Bentonite-cement grout—all find applications on Forest Service construction.

Simplification of concrete techniques so that any capable field man can produce a dense, workable concrete mix, makes it possible to get

*Received by the Institute August 1944.
†Engineer, Division of Engineering, U.S. Forest Service, Washington.

Fig. 1—Smallwood bridge on Spring Creek, Missouri. It is typical of the low water bridges used where the floodwaters cover so much of the wide valleys that high water bridges would be extremely long and their high cost could not be justified by the traffic. Gage posts on each end of the bridge indicate the depth of the water over the bridge in the early stages of the flood and serve as a guide to the motorist as to his chance of crossing the river.

away from the frequently inapplicable 1:2:4 mix and to design economical mixes with a minimum of effort.

To this end, simple instructions and charts have been carefully worked out without the scientific refinements that are justified on the big job but with the *same basic principles.* The "Water Developments and Sanitation Handbook" of the Forest Service contains complete instructions for producing impervious concrete; the "Truck Trail Handbook" carries specifications for concrete suitable for culverts, bridges, posts, and paved sections; the "Improvement Handbook" includes stucco and plaster data as well as information on reinforced structural concrete. Forms and reinforcement fasteners are described in some detail. It is in this dissemination of handbooks, specifications, and instructions that an organization such as the Forest Service has certain advantages over the individual. There is also the aid rendered by competent inspection and exchange of ideas within the organization.

By providing field forces with laboratory screens and scales to be passed from one job to another, and by making provision for essential

Fig. 2—Inlet of 48 inch pipe culvert at Station 669-65 Douglas City—Peanut Forest Highway, Trinity National Forest, Calif.

Fig. 3—Culvert Intake and drain on Forest Highway, New Hampshire.

control tests by state highway, commercial, or other laboratories so that mistakes will be discovered and not recur, local workers can turn out creditable, long-life structures in large numbers. Men interested in producing good concrete can become proficient in a comparatively short time and then replace the incompetent. Adequate control that will maintain a uniform standard in spite of changes in materials is the major problem on remote jobs which are often finished before standard strength tests can be reported.

Inasmuch as cement and reinforcing steel are covered by mandatory standard government specifications, considerable attention is given to the selection of aggregates. It has been found that it pays to prospect

Fig. 4—Tripoli Bridge, White Mountain National Forest, New Hampshire.

Fig. 5—High Knob Dam, Jefferson Nat'l. Forest, Virginia. Aggregates delivered down hillside by chute to mixer platform; cement stored in log cabin; concrete wheeled to forms; skip serves various levels.

Fig. 6—Green Pastures Dam, Jefferson National Forest, Virginia.

Fig. 7—Herrington Dam side channel spillway, Maryland.

Fig. 8—Quannah Creek dam, Okla.

Fig. 9—Donner Creek Bridge from the West, Tahoe City-Truckee Highway, Tahoe National Forest, Calif.

TABLE 1—BASIC DATA FOR DESIGNING MIX

	Cement	Sand	Gravel	Mixed * Aggregates	Water
Wt. per cu. ft. (dry rodded)	94	104	106	126	62.4
Specific gravity	3.1	2.65	2.65	—	1.0
Fineness Modulus	—	3.05	7.30	5.86	—
% Voids	51.5	36	36	24	—

* 1 part sand to 2 parts gravel.

TABLE 2—COMPUTATIONS FOR MIXES AND YIELDS

	Parts by Vol.	Absolute Vol. of 1 cu. ft.	Weight	Absolute Vol. of Mix.	Sax. Cem. per c.y. Concrete	abs. vol. solids per cu. yd. concrete	Yield = Lbs. per cu. yd. concrete	Parts by wt.
	(dry rod-ded)	(lbs. per c.f. ÷ sp. gr. x wt. c.f. water)	lbs.	(c. f. per 1 bag batch)	(27 c. f. divided by abs. vol.)		(Wt. x Sax/c. y.)	(Yield divided by wt. cem.)
Cement	1	x $\frac{94}{3.1 \times 62.4}$	= 94	÷ 193 = .49	x 4.58	= 2.24	431	1
Sand	2.5	x $\frac{104}{2.6 \times 62.4}$	= 260	÷ 162 = 1.61	x 4.58	= 7.37	1191	2.75
Gravel $\frac{3}{16}''$ to $1\frac{1}{2}''$	4.5	x $\frac{106}{2.65 \times 62.4}$	= 477	÷ 165 = 2.89	x 4.58	= 13.24	2186	5
Water	* .9	x $\frac{62.4}{1 \times 62.4}$	= 56.2	÷ 62.4 = .90	.....	.....	257	.....
TOTAL				5.89	27 / 4.58	$\frac{22.85}{27}$ = .85 = density as placed		

$$\frac{\text{* 6.75 gals. per sack (1 c.f.)}}{\text{7.5 gals. per cu. ft.}} = \text{Water cement ratio}$$

$$\frac{\text{Wt. Water}}{\text{Wt. cement}} \times 1.504 = \text{" " "}$$

TABLE 3—TRIAL CONCRETE MIXES
Guide for Small Jobs Where Tests are not Justified

Estimated Comp. Strength at 28 days (Lbs. per sq. in.)	Max Size Aggregate (Inches)	Sacks Cement Per C.Y. Concrete	Max. Water Per Sack Cement (Gallons)	Lbs. of surface-dry, 2.60 sp. gr. aggregate per sack cement		Mix by Weight
				Fine	Coarse	
2,000+	1	5	8	275	370	1 = 2.9 = 3.9
2,000	2	4½	8	295	430	1 = 3.1 = 4.6
2,000	3	4	8	305	520	1 = 3.2 = 5.5
2,500+	1	5½	7	235	320	1 = 2.5 = 3.4
2,500	2	5	7	245	380	1 = 2.6 = 4.0
2,500	3	4½+	7	255	450	1 = 2.7 = 4.8
3,000+	1	6	6½	215	290	1 = 2.3 = 3.1
3,000	2	5½	6½	225	355	1 = 2.4 = 3.8
3,000	3	5	6½	235	410	1 = 2.5 = 4.4
3,500−	1	6½	6	185	275	1 = 2.0 = 2.9
3,500	2	6	6	195	320	1 = 2.1 = 3.4
3,500	3	5½	6	215+	375+	1 = 2.3 = 4.0
4,000−	1	7+	5½	160−	255	1 = 1.7 = 2.7
4,000	2	6½+	5½	170−	290	1 = 1.8 = 3.1
4,000	3	6+	5½	180−	350	1 = 1.9 = 3.7
4,500−	1	8	5	135	225	1 = 1.4 = 2.4
4,500	2	7½	5	145	265	1 = 1.5 = 2.8
4,500	3	7	5	155	300	1 = 1.6 = 3.2

Use maximum proportion of well-graded coarse aggregate consistent with good workability. To increase slump, reduce the quantities of aggregates, maintaining w/c ratio specified. When specific gravity is not 2.60, multiply pounds of aggregate by ratio of actual specific gravity to 2.60.

all available sources of sand and gravel (pit run or crushed), local, or commercial. In most pits, the overburden, gradation, and structural properties of the materials are fairly easily determined even though some screening of samples or even laboratory tests may be involved. Not only are the standard A.S.T.M. tests for "*Organic Impurities in Sand*" and "*Clay and Silt in Sand*" used, but knowledge acquired through use of local sand, gravel, and rock on roads is also brought into play in recognizing suitable aggregates. It is important to differentiate between dirty, bond-weakening coatings and fine rock powder that fills voids. Frequently the installation of a screening plant or sand washer, or both, is called for and pays dividends. Here is where the portable crushing and screening plants that turn out road materials can serve a dual purpose by also producing aggregates.

Laboratory determinations are usually limited to the minimum; i.e.: 1) Sieve analysis of fine and coarse aggregates; 2) Specific gravity and weight per cubic foot of aggregates; 3) Absorption of aggregates; 4)

Fig. 10—Victoria Dam, Black Hills, South Dakota.

Fig. 11—Two (90-ft.) span rigid frame concrete bridge constructed by Forest Service over E. Branch Pemigewasset River, Swift River Forest Highway, near Lincoln, N. H. White Mtn. Nat'l Forest.

Moisture content of aggregates; 5) Strength tests—(disclosing structural value of aggregates).

Fineness modulus, that big name for a simple, convenient way of designating average size, is used by some field engineers as an indicator of the relative coarseness of the grading. Others use it for aggregate-void mix design, but this is the exception not the rule.

More often envelope curves are developed to fit local materials and grading is kept within these limits. Tables showing the allowable percentages of various sizes of aggregates are also supplied to field men. Permissible tolerance in size variation usually allows considerable latitude in selection. The use of well-graded aggregates is stressed even though it is rarely feasible to separate the coarse aggregate into more than two sizes.

(The author presents Tables 1-3 as reference material for those not experienced in design.)

The choice of maximum size of aggregate follows standard practice except that there is a tendency to utilize "plums" (not closer than 12 in. to each other) where the mixer is unable to handle large aggregate and where the cost of hauling in cement is very high.

Most of the concrete mix design instructions have been built around the water-cement ratio and trial batch methods. In trying to avoid the old pitfall of adding water when the mix is unworkable, it is customary to vary the proportion of sand and cement, retaining the maximum amount of coarse aggregate that is consistent with good workability. This amounts to adding between 3 and 4 pounds of cement for every 1 percent increase in sand.

Water-cement ratios to meet various conditions are tabulated for convenient use. Allowance is, of course, made for moisture in aggregates on most of the jobs, sometimes by weighing, sometimes by empirical rules checked by adding various quantities of water to the dry sand and observing its appearance.

When no facilities are available for proportioning by weight, volumetric measurements are adjusted for bulking. Slumps of 3 to 4 inches predominate.

Cement contents are kept within prescribed limits (say 4 to 7 sacks per cu. yd.) unless written permission to use other amounts is given. Admixtures are rarely used. Data such as that contained in Table 10 "Mixes for Small Jobs," ACI Journal, November 1943, P. 111, are distributed to the field as guides.

Especial care in curing is required as these are non-technical techniques that can be applied in remote localities and have sufficient influence on the ultimate end products to justify the time spent.

Finishing with rough surfaces that harmonize with nature is encouraged; rubbing to a shiny reflecting surface is frowned on.

The "*Handy 40 Rules*" are rough but helpful:

1. Forty per cent of the total dry weight of sand and gravel equals the *weight of sand.*
2. Forty degrees Fahrenheit equals *minimum permissible placing temperature.*
3. Forty seconds equals *minimum mixing time.*
4. Forty minutes equals average *initial setting time.*
5. Forty days equals desirable *curing period.*
6. Forty years equals expected *life of concrete* without maintenance.

Take for example, a typical bridge abutment job in the mountains several miles ahead of finished road grading. Cement may be brought in on pack animals or by trailbuilder and trailer. Sand and gravel may be screened from the stream bed.

A mix is designed by the forest engineer to give the required strength and density utilizing a minimum amount of cement and the best combination of aggregate available. Screen sizes are specified.

Forms of rough lumber (from a nearby mill) supported by hewn logs or poles (cut on the job) give a rough finish, but there is no sloppy concrete, no segregation, no placing in freezing weather without proper precautions.

Concrete resulting from these simple methods of control has such strength and durability that greater refinements would not produce commensurate savings, nor would they be practical on the scattered projects involved. Without the achievements of concrete technicians on major jobs recorded in technical literature as a reference and guide, the success of these relatively rough and ready methods would not be possible. As it is, valuable time and manpower are used where they will do the most good.

[To facilitate selective distribution, **separate prints** of this title (42-2) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Mar. 1, 1946]

Title 42-2 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
(copyrighted)

Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **September 1945**

Lapped Bar Splices in Concrete Beams*

By RALPH W. KLUGE†
Member American Concrete Institute
and EDWARD C. TUMA†

SYNOPSIS

An investigation was conducted to determine the general behavior and strength of lapped bar splices which varied in length and method of splicing. The maximum bond resistance developed in the splice and the slip of bar was determined for two types and sizes of reinforcement. The resulting data clearly illustrated the manner in which the stress was transferred from one lapped bar to the other and the relative merits of the two types of bars as well as the effectiveness of the two methods of splicing was shown.

INTRODUCTION

1. Object and scope of investigation

There is little information available in the technical literature concerning the behavior of lapped reinforcing bar splices. As far as is known to the authors, published data pertain to relatively long laps involving plain round bars.‡ It therefore seemed desirable to conduct a study of this type of splice including such variables as length, method of lapping, and type of deformed bar.

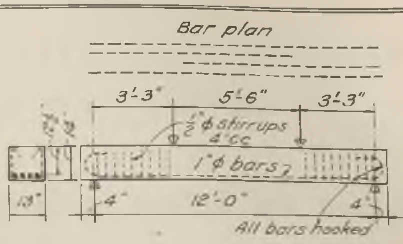
Specifically, the purpose of the tests described in this paper was: first, to determine the distribution of stress along the lapped bars and the accompanying bar slip; second, to compare the effectiveness of the lapped bars spaced 1½ bar diameters apart with that of the lapped bars in contact with each other; and, third, to compare the behavior of two types of reinforcing bars, one having a relatively greater lug height than

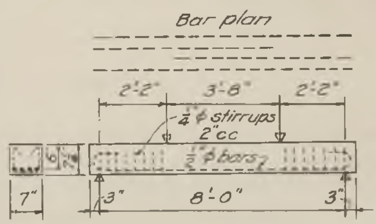
*Received by the Institute Mar. 26, 1945.
†National Bureau of Standards, Washington, D. C.
‡Technologic Papers of the Bureau of Standards No. 173, "Tests of Bond Resistance between Concrete and Steel," by W. A. Slater, F. E. Richart, and G. G. Scofield, or "Structural Laboratory Investigations in Reinforced Concrete Made by Concrete Ship Section, Emergency Fleet Corporation," by W. A. Slater, Proc. American Concrete Institute, V. XV, 1919.

TABLE 1

Bars	Series	Specimen	Nominal Length of Lap, in.
1" φ bars	A	A-50	50
		A-40	40
		A-30	30
		A-20	20
		A-40X	40
		A-30X	30
		A-20X	20
	B	B-40	40
		B-30	30
		B-20	20
		B-30X	30
		B-20X	20
½" φ bars	A	A-50	25
		A-40	20
		A-30	15
		A-20	10
		A-40X	20
		A-30X	15
		A-20X	10
	B	B-40	20
		B-30	15
		B-20	10
		B-10	5
		B-30X	15
		B-20X	10
		B-10X	5

Design Details of Beams





Note –
In specimens designated with the letter "X" following specimen number, lapped bars were in contact with each other. In all other beams, bars were spaced 1½ dia. clear.

the other. Both bars were considered to be among the best available as far as bonding properties were concerned.

The test program was divided into two parts as indicated in table 1. One part covered the tests of large beams containing bars 1 inch in diameter, and the other, tests of small beams reinforced with bars ½ in. in diameter. For each size of bar, two sets of tests were made, designated series A and series B. All tests were made on each of the two types of bars.

The tests of series A were primarily concerned with the manner in which the tensile stress or bond stress varied along the lapped bars and the magnitude of the slip at various points. The data also served to compare the behavior of the two types of bars and the relative effectiveness of the two methods of lap splicing. The length of splice in this series varied from 20 to 50-bar diameters and only a single test was made for each length of lap, type and size of bar.

Series B was planned to provide values of average maximum bond stress for several lengths of lap. However, there were few well defined bond failures and, notwithstanding the use of high-yield-strength reinforcement, many specimens failed by yielding of the bars beyond the

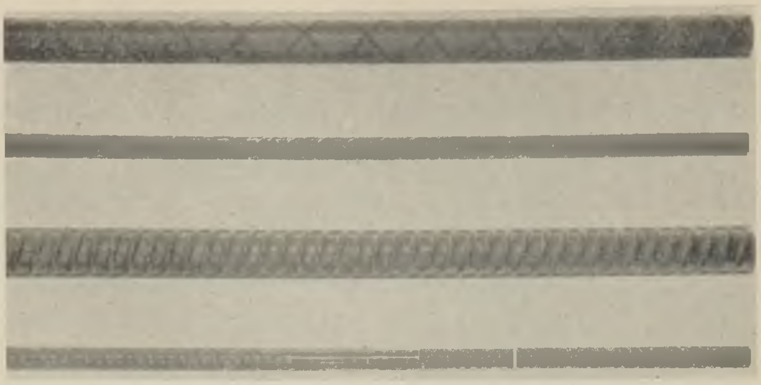


Fig. 1—Reinforcing bars (2 No. 1 at top; 2 No. 2 below)

splice. These tests, however, had some value insofar as indicating the effectiveness of the shorter laps. In this series, the length of the splices varied from 10 to 40-bar diameters and in all but a few instances duplicate tests were made.

DESCRIPTION OF TEST SPECIMENS

2. Design of beams

Details of the test specimens are shown in the diagrams of Table 1. As indicated in the bar plan, each beam contained a single lap splice and two continuous bars. The beams were designed to permit a clear spacing of 1½-bar diameters between all of the bars at a section through the splice and their effective depth was such that theoretically, for a stress of 20,000 psi in the continuous bars, the concrete was stressed to 1,500 psi in compression, assuming a modular ratio of $n = 8$. This design resulted in a section, outside the region of the lap splice, containing 1.4 percent of tensile reinforcement.

The span of the large beams (containing the 1-in. bars) was 12 ft. and that of the small beam (with the ½ in. bars) was 8 ft. Load was applied to the beams at two points, so spaced as to provide a region of constant moment sufficient in extent to accommodate the longest lap splice of 50-bar diameters. The actual lengths of lap were slightly greater than the given nominal values in order to accommodate an even multiple of a 5-in. spacing of gage holes. The exact values were ¼ in. and ½ in. greater than the nominal lengths for the ½ in. and 1 in. bars, respectively.

The reinforcement consisted of two types of bars, illustrated in Fig. 1, and designated by number. Bar No. 1 had average lug heights of 0.020 in. and 0.027 in., whereas Bar No. 2 had average lug heights of 0.031 in. and 0.063 in. for the ½ in. and 1 in. bars, respectively. The approximate bearing areas of the lugs per lineal inch are given in Table 2. For the

TABLE 2—DIMENSIONS OF REINFORCEMENT

Type	Bar size	*Diameter	*Area	Av. lug height	Lugs per yd.	Approximate bearing area of lugs per lineal inch
	in.	in.	sq. in.	in.		sq. in. per in.
Bar No. 2	½	0.49	0.190	0.031	128	0.16
	1	1.00	0.787	0.063	80	0.42
Bar No. 1	½	0.51	0.198	0.020	100	0.08
	1	1.00	0.785	0.027	50	0.11

*As determined by weight-length method.

TABLE 3—MECHANICAL PROPERTIES OF REINFORCEMENT
(Each value is the average of three tests)

Test Series	Type	Bar size	Yield point	Tensile strength	Elongation in 8 in.
		in.	psi.	psi.	percent
A	Bar No. 2	½	50,000	79,800	20.4
	Bar No. 1	½	61,800	90,000	22.0
	Bar No. 2	1	45,000	71,600	26.2
	Bar No. 1	1	51,600	97,200	18.8
B	Bar No. 2	½	61,800	97,400	16.6
	Bar No. 1	½	55,000	92,600	18.0
	Bar No. 2	1	60,400	94,200	18.8
	Bar No. 1	1	53,000	92,300	23.1

specimens of series A, the reinforcing bars were of intermediate grade or better, whereas the bars used in beams of series B were exclusively of high-strength steel, having a yield point of from 53,000 to 62,000 psi. Their dimensions and mechanical properties are listed in Tables 2 and 3, respectively.

The concrete was designed for a compressive strength of 4,500 psi at 28 days and consisted of moderate-heat-of-hardening portland cement and washed Potomac River sand and gravel, the latter having a maximum size of ¾ in. The proportions of cement, sand, and gravel in the mix were, respectively, 1:2.5:3.3 by dry weight; and the net water content, 7½ gal. of water per sack of cement, corresponding to a water-cement ratio of 0.67 by weight. The resulting concrete had an average slump of 3½ in. and developed an average compressive strength of 4,700 psi tested dry at 28 days.

3. Construction of specimens

Four beams were generally cast at one time, two small beams and two large beams, both pairs containing splices of equal length in terms of the

Fig. 2—Reinforcement for 30-bar diameter splice. Lapped bars in contact. Specimen type A-30X.

bar diameter. One of each pair contained Bar No. 1 and the other Bar No. 2. The reinforcement was rigidly supported on wire chairs as well as on tapered wooden inserts. Removal of the latter from the cast beam provided openings to the bars for strain measurements. Spacing of the inserts is shown in Fig. 2 for beams of series A. The beams of series B contained gage holes, similarly spaced, but confined to the region outside the limits of the splice.

The free ends of the lapped bars were saw cut in order to avoid the distorted ends generally accompanying shear cuts. Where no space was provided between the lapped bars, they were firmly wired together at the center and near the ends of the lap with soft iron wire. The concrete was vibrated into place in the beams and the control cylinders. Twenty-four hours after casting, the side forms of the beams and cylinders were removed. The test specimens were then wrapped with several layers of wet burlap and cured in this condition for a period of 7 days, after which they were permitted to dry in the laboratory for 21 days before testing.

DESCRIPTION OF TESTS

4. Procedure

To facilitate the measurement of strain, the beams were tested in an inverted position on standard beam supports resting on the platen of a 600,000-lb. capacity hydraulic testing machine equipped with a 30,000-lb. capacity gage. The load was applied near the ends of the specimens through a pair of wide flange I-beams resting on rollers and bearing plates, which can be observed in Fig. 3 and 4. The weight of the loading beams and auxiliary equipment was approximately 2,400 lb. and was considered as part of the total applied load.

Strain-gage readings were obtained with a 5-in. Whittemore gage for 8 to 10 increments of load. In the tests of series A, strain measurements were made at 2½ in. and 5 in. intervals on the ½ in. and 1 in. bars, respectively, the intervals extending from the free ends of the lapped bar to the load points and at a number of selected points on the continuous

Fig. 3—Eight-foot beam of series A under load.

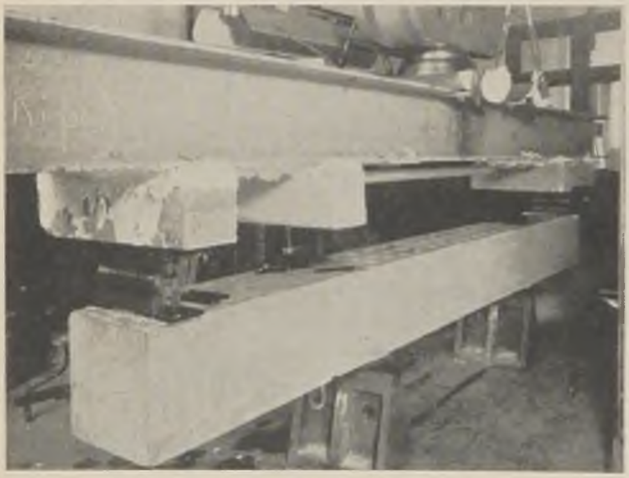
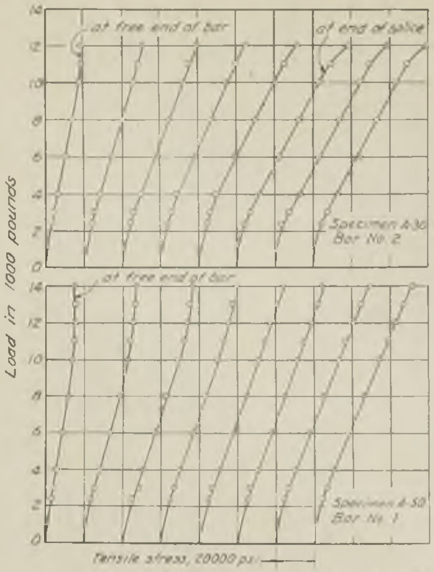
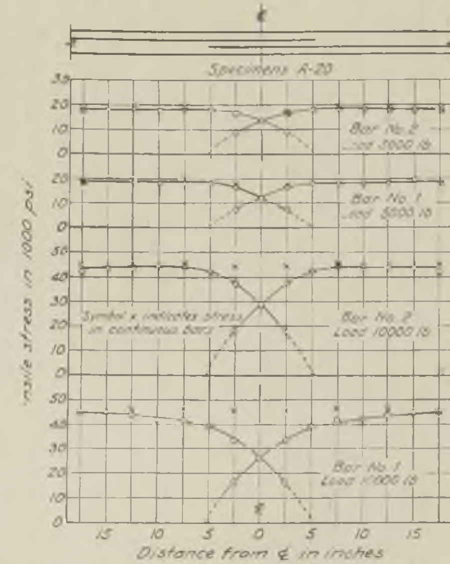


Fig. 4 — Twelve-foot beam of series A under load.

bars. Slip of the lapped bars was determined with the strain gage by observing the relative movement between gage holes on the lapped bars and those on the continuous bars one gage length removed. The readings were corrected for strain in the bars as well as for the angle of the line of measurement. Measurements of strain in the tests of series B were confined to gage lines outside the region of the splice, thus avoiding the uncertain effect of openings in the concrete on the bond stresses along the lapped bars. The average bond stress could be determined from the observed tensile stress in the bars at the ends of the lap.

During the latter stages of the test, the beams were carefully scrutinized for possible longitudinal cracks which, in some beams, formed in the concrete along the lapped bars when the maximum load was reached.





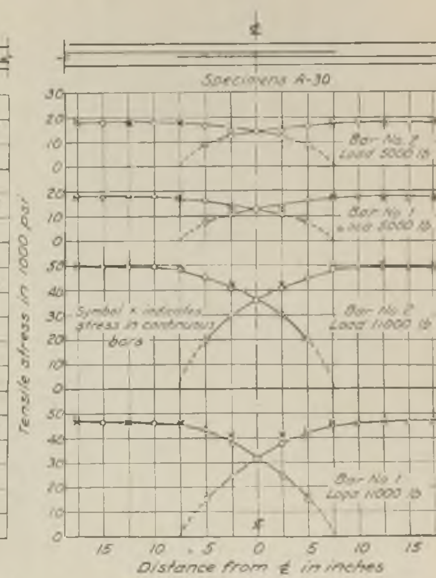


Fig. 5 (left)—Typical load-stress curves for successive gage lines along lapped bars.

Fig. 6 (center)—Distribution of stress along lapped bars—Length of splice 20-bar diameters. ½ in. bars.

Fig. 7 (right)—Distribution of stress along lapped bars—length of splice 30-bar diameters. ½ in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

TEST RESULTS OF SERIES A

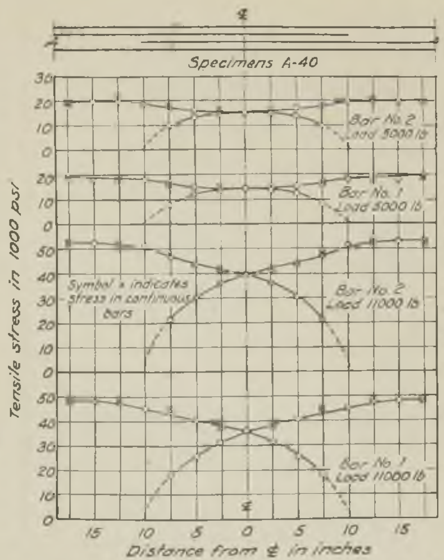
5. Distribution of stress along lapped bars

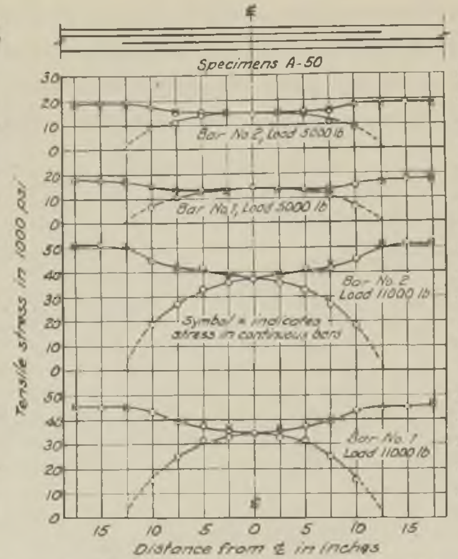
The strains at corresponding gage lines on each of the two bars of the lap were averaged and converted to stress on the assumption that E_s was 30,000,000 psi, and load-stress diagrams plotted for all of the gage points along the bar. Typical diagrams are shown in Fig. 5. The curves are arranged in the order that the gage lines appear on the lapped bar, starting at the free end. After initial cracking of the concrete, the stress in the bar at various points was closely proportional to the load until either the bond stress within the gage line approached a maximum or the yield point of the steel was reached. The curves for the portion of the bar near the free end clearly show the maximum tensile stress, which is also a measure of the maximum bond stress that the bar was capable of developing within the first few inches of embedment.

The variation of tensile stress along the lapped bars of the various specimens is shown in Fig. 6 through 13. The plotted values were taken from the load-stress curves described above for loads near the maximum and for loads which produced a stress of about 18,000 to 20,000 psi in the bars outside the region of the splice. The pair of curves shown on each axis, one for each bar of the splice, are identical except that they are turned end for end. Both the "No. 1" and "No. 2" bars are represented in each figure.

The slope at any point on the curves is obviously a measure of the bond stress developed between the bar and the concrete at that point. The greatest bond stress was developed at the free end of the bar, and this stress decreased fairly uniformly in the 20- and 30-diameter laps over the entire splice. In the longer splices, however, the bond stress, after a similar decrease reached a low value within the middle third, then increased towards the end of the lap. In some instances, where the stresses at the loaded end were less than 30,000 psi, the bars within the middle third of the splice did not develop any bond stress. This was particularly evident in the 40-diameter splices and the 50-diameter splices of the "No. 2" bars. From the standpoint of efficiency, such a splice is longer than necessary.

The stress in the continuous bars at various points in the region of the splices is also indicated on the diagrams by a small cross. Except for splices 20-bar diameters long, the effect of the lapped bars was to decrease greatly the stress in the continuous bars within the limits of the lap, in many instances very closely approaching the stress in the portion of the lapped bar from the center of the lap to its loaded end. In beams containing the 20-diameter splices, the stress in the continuous bars was, in general, fairly constant over the entire lap, the effective steel area of





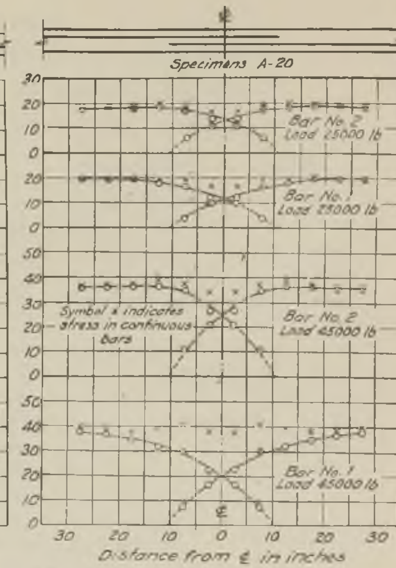


Fig. 8 (left)—Distribution of stress along lapped bars—length of splice 40-bar diameters. ½ in. bars.

Fig. 9 (center)—Distribution of stress along lapped bars—length of splice 50-bar diameters. ½ in. bars.

Fig. 10 (right)—Distribution of stress along lapped bars—length of splice 20-bar diameters. 1-in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

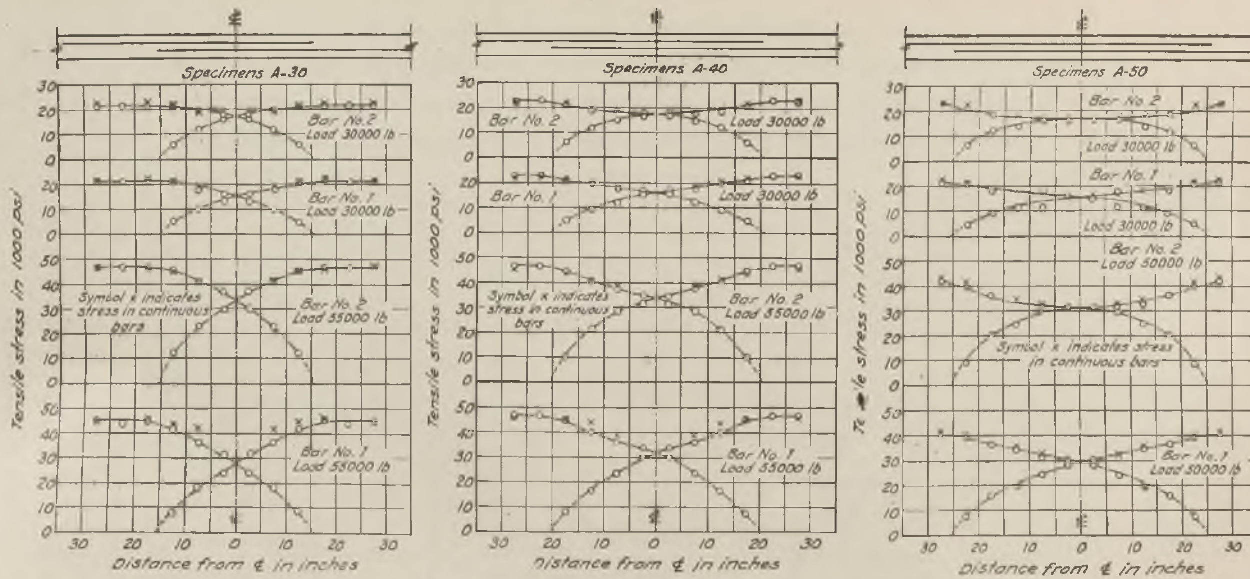
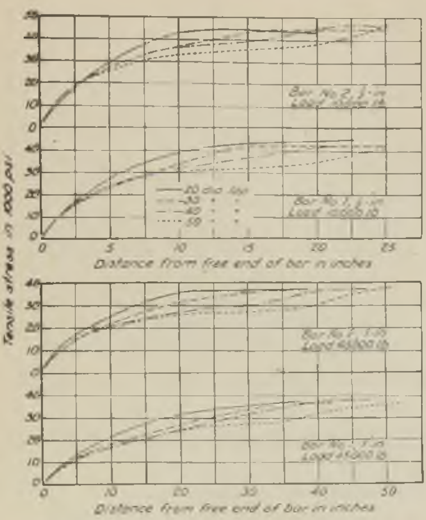


Fig. 11 (left)—Distribution of stress along lapped bars—length of splice 30-bar diameters. 1 in. bars.

Fig. 12 (center)—Distribution of stress along lapped bars—length of splice 40-bar diameters. 1 in. bars.

Fig. 13 (right)—Distribution of stress along lapped bars—length of splice 50-bar diameters. 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.



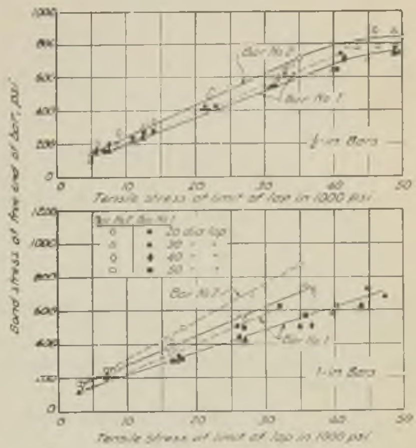


Fig. 14 (left)—Distribution of stress along lapped bars for various lengths of splice.

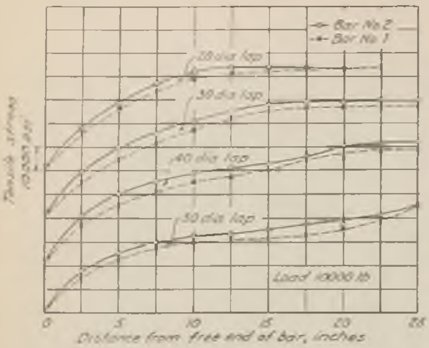
Fig. 15 (right)—Relation between bond stress at free end of bar and tensile stress at limit of lap.

Stress values are the product of the modulus of elasticity and observed strains.

the lapped bars apparently being equivalent to a single bar at any section.

A detailed comparison between the stress distribution curves for each length of lap is shown in Fig. 14. The curves represent the stress distribution for a load of 10,000 lb. on the beams containing the ½ in. bars and 45,000 lb. for the beams containing the 1-in. bars. These loads stressed the reinforcement beyond the splice to 45,000 psi and 37,000 psi, respectively. The slopes of the curves in the vicinity of the origin, which represents the free end of the bar, are almost identical for each set of curves, consequently the bond stress developed within the first few inches of bar embedment was practically alike, regardless of the length of splice for a given load and type of bar.

It is to be expected that some relationship exists between the bond stress at the free end of the lapped bar and the tensile stress in the bar at the other limit of the splice. Fig. 15 indicates, within certain limits, a linear relationship which, for each type of bar, apparently is independent of the length of splice for laps of 30-bar diameters or more. Thus, the data for the 30-, 40- and 50-diameter laps appear to group themselves about individual straight lines, one for Bar No. 1 and one for Bar No. 2, between tensile stresses of 5,000 psi and 40,000 psi. The 20 diameter lap splices also show a similar relationship, but the slopes of the lines are, in some instances, greater than those of the longer splices. Where they differ, the data for the 20 diameter lap are indicated by a broken line.



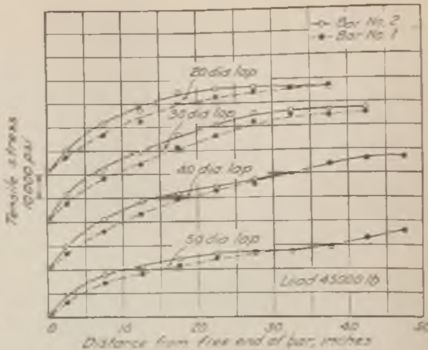


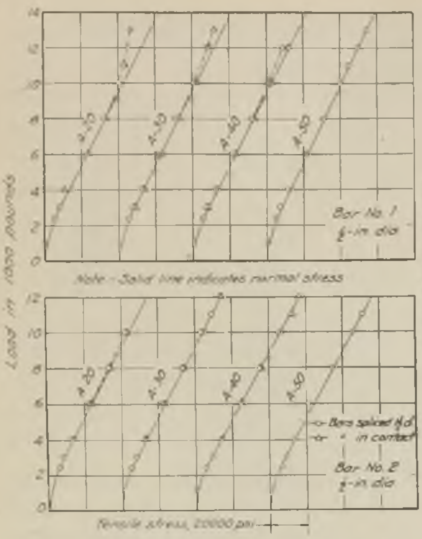
Fig. 16 (left)—Distribution of stress along lapped bars compared for "No. 1" and "No. 2" ½ in. bars.

Fig. 17 (right)—Distribution of stress along lapped bars compared for "No. 1" and "No. 2" 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

The distribution of tensile stresses along the lapped bars for a given load and for various lengths of lap is compared in greater detail for both types of bar in Fig. 16 and 17. In every instance, the No. 2 bars picked up stress more rapidly at their free ends than the No. 1 bar, although the rate of increase of stress beyond an embedment of 5-bar diameters was not significantly different. In a well designed splice, the *normal* stress, that is, the stress the bar would normally carry if it were continuous, should be reached at the end of the splice. In Fig. 18 and 19, the observed stresses in the lapped bars at the end of the splice are plotted with respect to the load applied to the beams for each length of lap. Also shown by solid lines is the load-stress relationship that would obtain if the bars were continuous. This *normal* stress was assumed to be the same as the average stress observed in both the continuous and lapped bars well outside the limits of the splice. The deviation of the observed stress from the *normal* stress beyond a value of about 35,000 psi is evident for most of the beams reinforced with No. 1 bars. The exceptions are the 50-diameter lap and possibly the 40-diameter lap. On the other hand, with the exception of the 20-diameter lap, the observed stresses in the No. 2 bars are practically identical with the *normal* stresses up to the yield point of the steel, which had a value of about 50,000 psi and 45,000 psi for the ½ in. and 1-in. bars, respectively.

It is evident that, if the stress in the lapped bar at the end of the splice is not equal to the *normal* stress, the adjacent continuous bars assume a greater proportion of the total tensile stress. Actually, this was not a serious matter in the beams which were tested with laps greater than 20 diameters; for even at the higher stresses the difference between the actual and *normal* stress was not more than 8 to 10 percent, probably half of



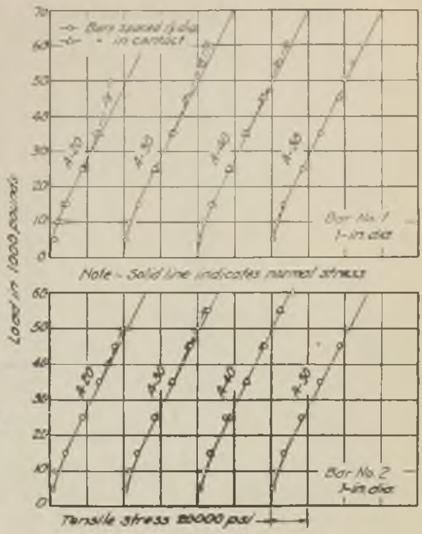


Fig. 18 (left)—Load-stress relation for lapped bars at limits of the splice—Beams of series A. ½ in. bars.

Fig. 19 (right)—Load-stress relation for lapped bars at limits of the splice—beams of series A. 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

which was assumed by the two continuous bars at that point. Consequently, the load which would normally stress the reinforcement, beyond the splice, to its yield point was not materially affected by the slight shifting of stress. This was not true, however, of the beams containing the 20-diameter lap splices, where the load at which yielding of the reinforcement occurred as a rule was somewhat lower than for the other beams.

6. Bond strengths

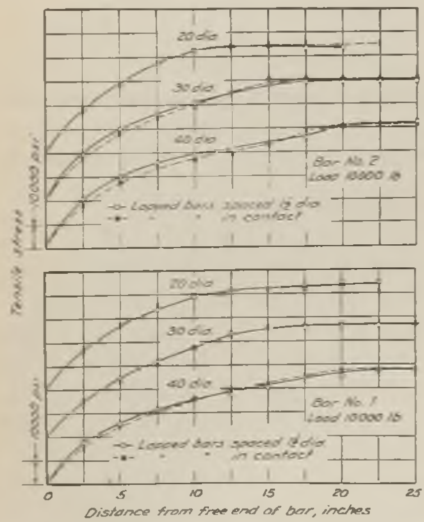
The load-stress curves for the gage line nearest the free ends of the lapped bars indicated that the tensile stress there reached a maximum value at approximately the same load in all of the specimens tested. Also, the maximum stresses at this point were reasonably alike for a given type and size of bar regardless of the length of lap splice. However, there was a marked difference in the values for the two types of bars. Using the maximum tensile stresses scaled from the curves, the bond strength developed by the lapped bars for an embedded length of about 3 in. was computed for each specimen and the bond stresses thus obtained are listed in Table 4. The values are somewhat higher than would be expected from standard pull-out tests, but compare favorably with results from unpublished data of similar bars in beam tests.

TABLE 4—MAXIMUM BOND STRESS DEVELOPED AT FREE END OF LAPPED BARS

(Av. stress for an embedment of approx. 3 in.)

Specimen	½-in. bars		1-in. bars	
	No. 2	No. 1	No. 2	No. 1
A-50	850	745	880	755
A-40	970	840	960	655
A-30	880	810	1,010	705
A-20	890	745	1,010	755
Average	895	785	965	718
*A-40x	880	675	755	630
*A-30x	920	675	1,010	755
*A-20x	950	810	1,000	760
Average	917	720	922	715

*Lapped bars in contact with each other.



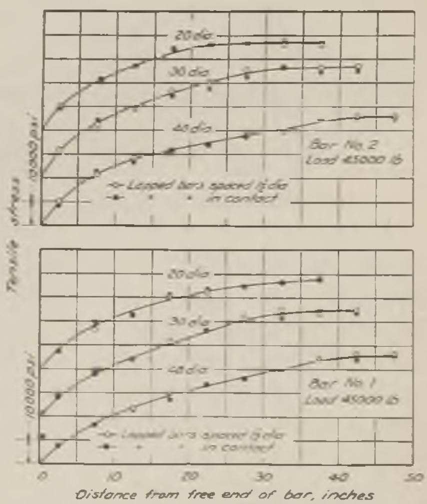


Fig. 20 (left)—Distribution of stress along the lapped bars of two types of lap-splices. ½ in. bars.

Fig. 21 (right)—Distribution of stress along lapped bars of two types of lap-splices. 1 in. bars.

Stress values are the product of the modulus of elasticity and observed strains.

7. Comparison between methods of lapping

The results of the two methods of lapping, one in which the lapped bars are spaced 1½ diameters clear and the other where they are in contact, are compared in Fig. 20 and 21. Here the stress distribution curves are shown for a given load and for each length of splice. There is practically no difference in the stress between the curves representing each type of lap at any point for the 1-in. bars and little difference for the ½-in. bars.

These differences are probably no greater than would be obtained in duplicate tests. A further comparison is shown in Fig. 18 and 19, in which the load-stress curves for the lapped bars at the limits of the lap are shown for both types of splice. Here, again, the differences are small and in most instances negligible. It is also evident from Table 4 that it made little difference in the bond strength in these tests whether the lapped bars were spaced 1½ diameters apart or in contact.

8. Distribution of slip along lapped bars

There were large differences in slip between certain specimens and these differences were not always consistent. It is highly probable that wide variations in slip data, obtained in the manner described herein, would also be observed in duplicate tests. It should be noted that the slip at each point along the lapped bar was reckoned from a point directly opposite it on the adjacent continuous bar, and consequently does not necessarily indicate the relative movement of the bar with respect to the concrete.

Slip starts, quite naturally, at the free end of the bar where the bond stress is always the greatest and progresses along the bar as the tensile stresses increase at the loaded end. In this respect, the behavior of the bar differs from that in the usual pull-out or beam tests for bond. The distribution of the slip along the lapped bars for various lengths of splice is shown in Fig. 22 and 23. The data are plotted for two loads, one of which produced a tensile stress of approximately 18,000 psi and the other about 40,000 psi at the loaded ends of the lapped bars. Most of the evidence seems to indicate that the distribution and magnitude of the slip is independent of the length of the splice. There appears to be a serious departure from this principle with the No. 1 bars at higher loads. The discrepancy is not readily accounted for except in so far as a part of the variation might be accidental. Within the range of working stresses, however, the length of lap seems to have little or no effect upon the magnitude of the slip for that bar.

With one exception, there was little difference in slip between the No. 1 and No. 2 ½-in. bars for a given tensile stress at the loaded ends of the lapped bars. The exception was the 20-diameter lapped No. 1 bar which showed a considerably greater slip than any of the other lapped bars of this size. The 1-in. No. 1 bars, however, exhibited a decidedly greater slip for a given load or end stress than the 1-in. No. 2 bars or either of the ½ in. bars. The only explanation that can be offered for the dissimilar behavior of the two sizes of No. 1 bars is the possible effect of the lug height. Table 3 shows that the 1-in. No. 1 bar had a much lower ratio of lug height to diameter of bar than the ½-in. bar of the same make, whereas this ratio for the two sizes of No. 2 bars was almost alike.

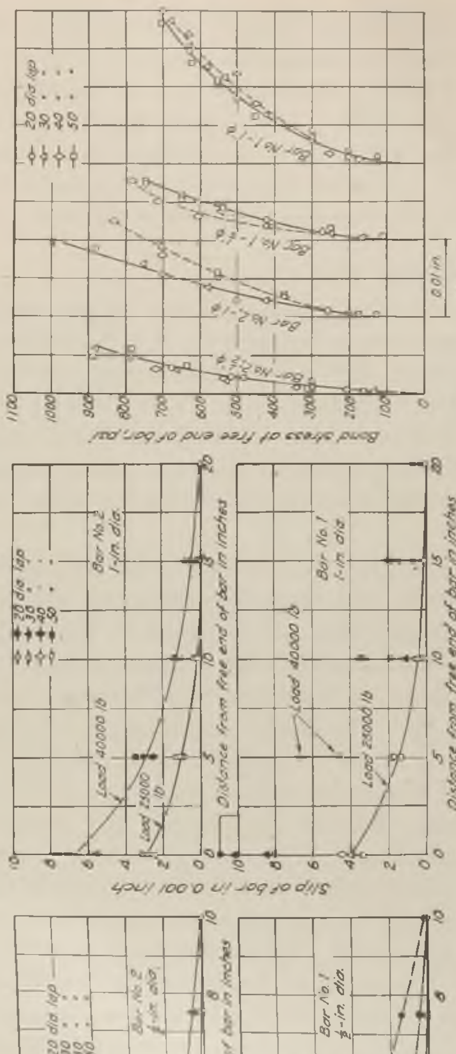


Fig. 22 (left)—Distribution of slip along lapped ½ in. bars.

Fig. 23 (center)—Distribution of slip along lapped 1 in. bars.

Fig. 24 (right)—Relation between slip at free end of bar and bond stress near free end.

The difference in the slip behavior of the two types and two sizes of bars is more clearly illustrated in Fig. 24 which shows the relation between slip at the free end of the lapped bar and the bond stress developed within the first few inches of the free end. In the range of bond stresses above 400 psi, the end slip of the ½-in. No. 2 bar is somewhat less than the 1-in. bar of the same make and as previously noted the slip of the ½-in. No. 1 bar is very much less than the 1-in. No. 1 bar for all bond stresses.

It is to be noted that slip is not directly proportional to the bond stress developed by the bar, but apparently is some exponential function of this stress, the expression for the relationship depending upon the type of bar.

TEST RESULTS OF SERIES B

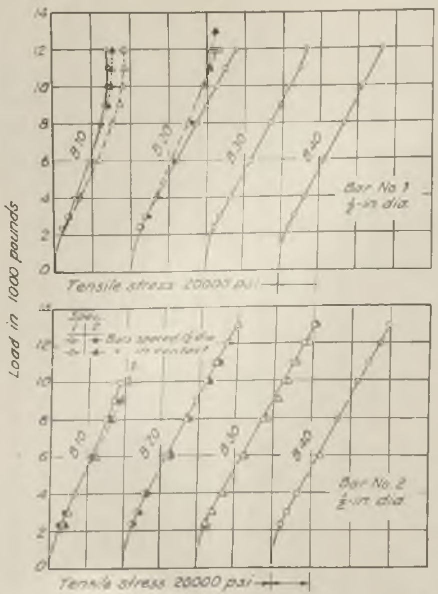
9. General manner of failure

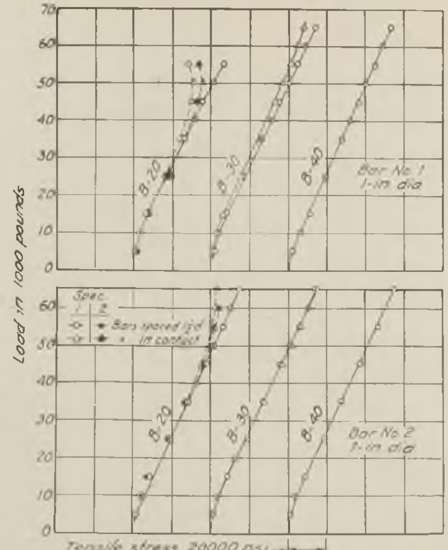
As stated in Section 3, the tests of series B were planned to determine the average maximum bond stress developed by the lapped bars for various lengths of lap. Although the specimens contained high-yield-strength reinforcement, many of the beams failed in tension. All of the 30 and 40-diameter lap splices developed the yield strength of the bar beyond the ends of the lap and all of the 10-diameter splices failed in bond. On the other hand, about half of the 20-bar diameter splices failed in tension and the remainder failed in bond. Many of the bond failures were not well defined because, in some instances, there was a gradual shifting of stress from the lapped bar to the continuous bars as the bond strength was reached. Some of the bond failures, however, were very evident from an examination of the load-stress curves of the lapped bars outside the limits of the splice. The strain in the bar at that point reached a maximum, as shown in Fig. 25 and 26 and either remained at that value or decreased with an increase in load. The breakdown of bond was also reflected, to some extent, by a change in the slope of the load-strain curves for the continuous bars at this section of the beam.

10. Average bond strength

Table 5 lists the maximum average bond stress developed by the various lapped bars. The values are based on the maximum tensile stress reached at a point just beyond the limits of the splice and on the total length of bar embedment, which was considered to be the distance from the free end of the bar to the mid-point of the first gage line.

The characteristic increase in the average bond strength for decreasing lengths of bar embedment is indicated in the table. Obviously, the bond stresses given for those specimens which failed in tension are not necessarily the average maximum values the bars are capable of developing for their particular length of embedment. Where bond failures are indicated,





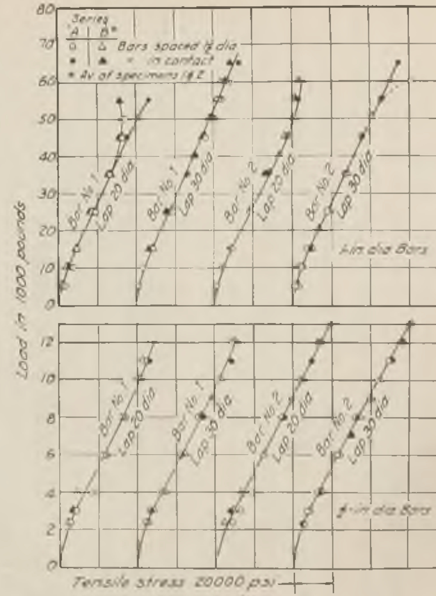


Fig. 25 (left)—Load-stress relation for lapped bars outside limits of splice—beams of series B. ½ in. bars.

Fig. 26 (center)—Load-stress relation for lapped bars outside limits of splice—beams of series B. 1 in. bars.

Fig. 27 (right)—Comparison between load-stress diagrams of series A and series B. Stress outside limits of splice.

Stress values are the product of the modulus of elasticity and observed strains.

TABLE 5—AVERAGE BOND STRESSES DEVELOPED BY LAPPED BARS JUST BEFORE FAILURE

Tests of Series B

Specimen	No.	Length of embedment	Bar No. 2		Bar No. 1	
			Tensile stress	Bond stress	Tensile stress	Bond stress
		in.	psi.	psi.	psi.	psi.
			½-in. bars			
B40	1	22¾	*60,000	325	*52,000	300
B30	1	17¾	*60.000	420	*55,000	385
B30x	1		*60,000	420	—	—
B20	1	12¾	*60,000	580	*55,000	535
	2		*60,000	580	43,500	425
B20x	1	12¾	*60,000	580	47,000	460
	2		*60,000	580	43,500	425
B10	1	7¾	39,000	625	28,500	455
	2		45,000	720	28,500	455
B10x	1	7¾	45,000	720	36.800	590
	2		40,500	650	30,600	490
			1-in. bars			
B40	1	43	*60,000	350	*53,000	310
B30	1	33	*60,000	455	*53,000	405
B30x	1	33	*60,000	455	**49,000	370
B20	1	23	*60,000	655	45,000	490
	2		43,500	475	30,000	330
B20x	1	23	43,500	475	30,000	330
	2		45,000	490	33,800	370

*Tension failures.
**Tension failures in continuous bar.

the performance of the lapped bars, which were in contact with each other again compare favorably, with those spaced 1½ diameters apart.

11. Effect of gage holes on the average bond stress

Undoubtedly the exposure of the bar at each gage hole affected the bond strength in the immediate vicinity of the hole. However, there was some evidence to indicate that the average bond strength of the entire embedded length of the bar was not significantly affected by these holes. Within the region of the splices in beams of series A, the holes in the con- concrete exposed approximately 4 percent and 7 percent of the surface area of the 1-in. and ½-in. bars, respectively. Fig. 27 shows load-stress diagrams for the 20-and 30-diameter lapped bars at a point 2½ in. beyond the ends of the splices. Comparison of these diagrams reveals little, if any, difference in the data from tests of series A and series B at this point. The data from series B, represented by solid symbols in the diagrams, are the average values obtained from duplicate tests, whereas the open symbols represent but a single test. Ordinarily, a comparison of the maximum loads between corresponding beams, at least for the shorter splices, would indicate the effect of exposing the reinforcement in this manner. However, ultimate failure of most of the beams in series A was due to yielding of the reinforcement.

DISCUSSION OF RESULTS

12. General discussion

It is recognized that many of the data presented are based on single tests, from which sound conclusions may not always be drawn. It is significant, however, that, except where observations of bar slip were concerned, the data were consistent from specimen to specimen. For example, comparisons of bond stress and its manner of distribution were made between the two types of splices and, although each comparison was based on a single test, the six different specimens exhibited consistently similar characteristics. Differences in the behavior of the two types of bars were likewise consistent.

Curves showing the distribution of stress along the lapped bars should, to some extent, be considered as qualitative. They indicate the general manner of stress distribution for each length of splice. Average values from two or more tests would perhaps differ from those shown.

It should be noted also that the splices which were tested were single bar splices flanked by two continuous bars. Different results might have been obtained had the specimens contained multiple lap splices without the accompanying continuous bars. Previous unpublished tests of beams constructed of Haydite concrete and containing multiple splices only, indicated that a greater cover than normally provided around the outermost bars of the beam was necessary if they were to assume their share of the total stress. Lapping of all bars at a section of a flexural member is, of course, not to be recommended in practice. The presence of continuous bars prevents the sudden and violent failure that may occur when all bars are lapped.

SUMMARY OF TEST RESULTS

13. General summary

The following briefly summarizes the results of the tests:

(1) The bars lapped 30-bar diameters or more developed, at the limits of the splice, the yield strength of the steel, a value which in some instances was as much as 60,000 psi.

(2) There was little difference observed in the behavior of splices containing bars spaced 1½ bar diameters clear or with the lapped bars in contact. It should be noted that, as far as bond resistance is concerned, the bars used in these tests contained a type of lug pattern probably superior to many commercial bars.

(3) Bond stress was the greatest near the free end of the lapped bar. The maximum values occurred at approximately the same tensile stress at the loaded end of the bar, regardless of the length of lap. A possible exception to this was the 20-bar diameter splice.

(4) With two exceptions, the magnitude and distribution of the slip along the lapped bars was similar for a given type and size of bar, regardless of the length of lap.

(5) The general behavior of the ½-in. bars was similar to the 1-in. bars, in so far as bond and tensile stresses were concerned. Differences in bar slip are noted in item 8.

(6) There was some evidence to indicate that the gage holes in the concrete of the beams did not seriously affect the strength of the splice.

14. Comparison of the two types of bars

(7) Bar No. 2, which had a considerably greater bearing area of the lug per inch length of bar, not only picked up stress at its free end in the splice more rapidly but developed greater maximum bond strengths than bar No. 1.

(8) The 1-in. diameter No. 1 bar exhibited a considerably greater slip for a given bond stress than any of the other bars. The differences between the two types of bar of the ½-in. size were minor. Both showed slightly less end slip than the 1-in. No. 2 bar for a given bond stress at the free end of the bar.

To facilitate selective distribution, **separate prints** of this title (42-3) are currently available from ACI at 25 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Mar. 1, 1946

Title 42-3 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
(copyrighted)

Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **September 1945**

Tests of Prestressed Concrete Pipes Containing A Steel Cylinder*

By CULBERTSON W. ROSS†

SYNOPSIS

Tests were made on prestressed reinforced concrete pipes of a type containing a steel cylinder. Data were obtained by tests under hydrostatic pressure, in crushing and in bending. The mechanical properties of the several parts, and the strain changes of the pipes under load are reported.

INTRODUCTION

A type of prestressed reinforced concrete pipe containing a steel cylinder was subjected to tests by the National Bureau of Standards. The pipes were furnished by, and the tests were made in cooperation with, the Water Division, Engineer Department of the District of Columbia. This report gives the results of hydrostatic pressure, bending, and crushing tests. The strain changes during the tests and the properties of the component parts of the pipes are also given. These will enable an engineer to judge the suitability of similar pipes for particular applications.

DESCRIPTION OF THE PIPES

The pipes consisted of a concrete-lined 16-gage welded sheet steel cylinder upon which was a wrapping of steel wire with a mortar covering, as indicated in Fig. 1. The dimensions of the pipes and their several parts are given in Table 1.

The lining of the 20-in. pipes was of crushed stone concrete and that of the 30-in. pipes was of gravel concrete. The linings were cast by spinning the steel tubes as the concrete was placed. The spinning expelled much

*Received by the Institute Feb. 26, 1945.
†National Bureau of Standards, Washington, D. C.

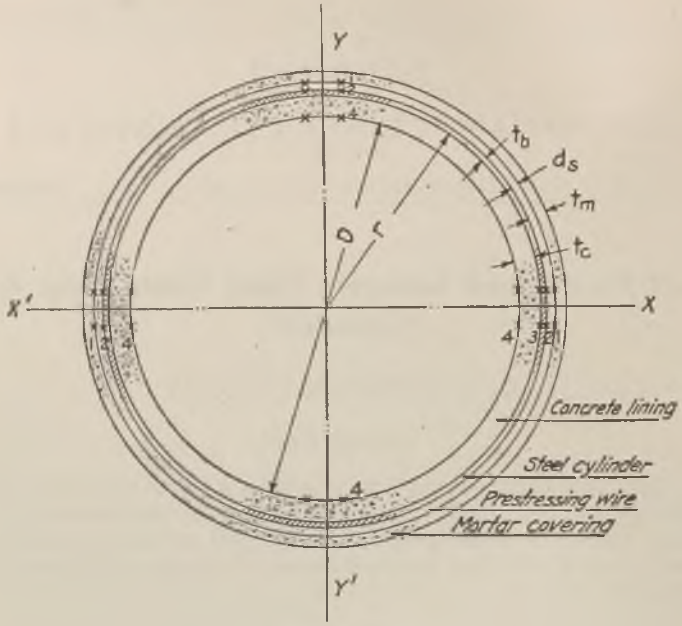


Fig. 1—Diagram of the pipe

The small "x" marks indicate the location of the strain gage stations.

of the water and compacted the concrete. A layer of material was formed at the inner surface of the lining which consisted chiefly of cement, whose thickness in the 20-in. pipes was about 1/16 in. and in the 30-in. pipes about 1/4 in. and in the latter the layer contained a network of shrinkage cracks.

The cylinder was wound with wire under high tensile stress after the lining had hardened. The wire was bonded to the pipe and protected from corrosion by the covering of mortar.

The device for sealing the pipes at the joints consisted of two steel rings of heavier gage than the cylinder and of different diameters, welded to the ends of the cylinder. A rubber ring was placed in a groove in the smaller ring of two adjacent pipes and compressed between the outer surface of one ring and the inner surface of the other as the pipes were drawn together*.

SCOPE OF TESTS

Hydrostatic pressure tests were made on one 20-in. and two 30-in. pipes, beam tests on one 20-in. and one 30-in. pipe, and crushing tests on

*More complete descriptions of the pipes and joints as well as test data on the mechanical properties of the pipes are given in the "Report on Lock Joint Prestressed Concrete Cylinder Pipe," Underwriters Laboratory, Inc., Nov. 24, 1944.

TABLE 1—DIMENSIONS OF PIPE AND MECHANICAL PROPERTIES OF MATERIALS

Dimension or property	Symbol	20-in. Pipe	30-in. Pipe
Internal diameter of pipe, in.	D	20	30
Length, ft.		16	16
Thickness of concrete lining, in.	t_c	1.22	1.85
Thickness of steel cylinder, in.	t_b	0.06	0.06
Thickness of mortar cover, in.	t_m	0.75	0.75
Diameter of wire, in.	d_s	0.19	0.19
Spacing of wire, in.		1.00	0.80
Cross-sectional area of wire per inch length of pipe, one wall, sq. in.	A_s	0.0284	0.0354
Cross-sectional area of cylinder per inch length of pipe, one wall, sq. in.	A_b	0.06	0.06
Cross-sectional area of lining per inch length of pipe, one wall, sq. in.	A_c	1.22	1.85
Radius of cylinder, in.	r	11.22	16.85
Yield point of wire (offset 0.2 percent), psi.	Y_s	170,000	170.000
Yield point of cylinder (offset 0.2 percent), psi.	Y_b	33,000	33,000
Tensile strength of wire, psi.	T_s	208,000	208,000
Tensile strength of cylinder, psi.	T_b	52,000	52,000
Elongation of wire (in 8-in. length), percent		4	4
Elongation of cylinder (in 2-in. length), percent		30	30
Modulus of elasticity of wire, psi.	E_s	27,500,000	27,500,000
Modulus of elasticity of cylinder, psi.	E_b	29,100,000	29,100,000
Compressive strength of concrete of lining (3.7 by 2.1 by 1.8-in. prisms), psi.		—	7,600
Modulus of elasticity of concrete of lining, psi.	E_c	—	4,200,000
Ratio E_b/E_c	n	—	7
Prestrain in wire	—	—	0.00237
Prestrain in cylinder	—	—	0.00020
Prestress in wire, psi.	f'_s	—	+65,000
Prestress in cylinder, psi.	f'_b	—	−5,800
Prestress in concrete, psi.	f'_c	—	−1,050
Maximum pressure sustained by pipe, psi.	P_m	740	(570 (600
Working pressure (original design), psi.	P_w	192	192
Crushing strength, lb./ft.	—	12,300	13,000
Beam strength, 14-ft. span, lb.	—	55,000	140,000

two 4-ft. lengths of the 20-in. and two of the 30-in. pipes. The two crushing test specimens of the 20-in. pipe were taken from the less stressed portions of the pipe used in the beam test. Those for the crushing test of the 30-in. pipe were cut from an untested pipe and the remaining portion of this pipe was the source of material for the tests of mechanical properties and prestress.

For the hydrostatic test, each end of the pipe was closed by a bulkhead which was sealed by the fittings used for connecting pipes in water mains. Each bulkhead consisted of a steel ring, corresponding to one of those at the end of the pipe, and a steel plate welded to one edge of the ring. The bulkheads and the rubber rings were forced onto the ends of the pipe by the plates of the testing apparatus. The prepared pipe was first filled completely with water and then an additional amount was pumped in to build up the pressure. The pressure increased rapidly until the pipe started to expand because of yielding of the reinforcing, after which it increased slowly. About two hours of pumping were required for the pressure to reach its maximum value, of which one and a half hours were required for the last 100 psi. increase. Strain changes in the wire were measured at X_1 and X'_1 (Fig. 1) at each 50 psi increment of pressure. The pressure was held constant while the readings were being made for a period of about 5 min. at each increment. A 2-in. Whittemore strain gage was used for the measurements of the 20-in. pipe and a 5-in. Whittemore gage for those on the 30-in. pipes.

For the beam tests, the pipes were supported and the load applied through V-blocks having interior angles of 120 deg. The blocks were cut from timbers of nominal size 12 by 12 in., giving a bearing length along the side of the pipe of about 11½ in. Timber struts, size 4 by 4 in., were placed vertically in the pipes at the center and at the supports to help prevent local crushing of the pipe. The pipes were loaded at the center of a 14-ft. span, measured center to center of the blocks. The deflection, measured with a surveyor's level, was observed for the 20-in. pipe only.

In the crushing tests, 4-ft. lengths of pipe were tested by the three-edge bearing method, described in A.S.T.M. Specification C 14-41. Measurements were made of the decrease in the vertical diameter, of the change of the strain at the inner surface of the lining at the top and bottom, and at the sides, and of the change of the strain in the tube and wire at the top and sides. A 5-in. Whittemore strain gage was used for the measurements made on the 30-in. pipe specimens and the outside of the 20-in. pipe specimens. The measurements of the lining of the 20-in. specimens were made with a 2-in. Whittemore gage.

The tensile strength, the yield strength, the modulus of elasticity and the elongation of the wire and of specimens of the steel cylinder were measured by the methods of test of A.S.T.M. E 8-42.

Two prisms approximately 3.7 by 2.1 by 1.8 in. and two approximately 2.1 by 2.1 by 1.8 in. were cut from the lining of a 30-in. pipe. The prisms were tested in compression and the modulus of elasticity measured with the pressure applied to the 2.1 by 1.8-in. faces. The prisms were dry from several weeks exposure to the air when tested.

The prestrain in the lining, cylinder and wire of the 30-in. pipe, from which the crushing strength specimens had been cut, was found by measuring the change in length of the parts with a 5-in. Whittemore strain gage when the prestress was released by cutting out a 12- by 12-in. section. The lining of the pipe was air dry at the time the tests were made.

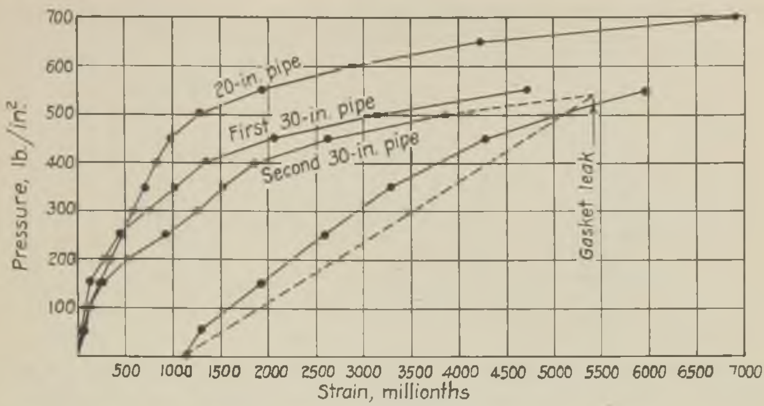


Fig. 2—Increase of the strain in the wire during the hydrostatic tests of three pipe.

TESTS AND RESULTS

Hydrostatic pressure tests

The maximum pressure sustained by the 20-in. pipe was 740 psi.

The maximum pressures sustained by the two 30-in. pipes were 570 and 600 psi. A leak appeared along a longitudinal seam of the first pipe at about 200 psi causing the mortar to be damp for a length of about 10 ft. The leak stopped as the test was continued and the damp spots disappeared completely before the end of the test. In the test of the second 30-in. pipe, the pressure was released at a pressure of 540 psi because of a gasket leak. The leak was repaired and the pressure again increased until the pipe failed.

As the pressure increased, cracks appeared in the mortar covering of the three pipes at about 200 psi and became rather numerous and large as the maximum pressure was approached. At the upper limit of the pressure one or more of the wires broke, neighboring turns loosened, and the cylinder was expanded outward in a ring around the pipe which was from 8 to 12 in. wide and about 1 in. high. The cylinders did not leak in spite of the large deformation of the expanded portion and still held a pressure substantially higher than the designed pressure (192 psi).

Strain changes in the wire were measured at X and X′ (Fig. 1) during the tests and are shown in Fig. 2.

Beam tests

Cracks became visible in the outer mortar covering of the 20-in. pipe at a load of about 20,000 lb. and in the 30-in. pipe at about 80,000 lb. The maximum load supported by the 20-in. pipe was 55,000 lb. The failure occurred by the buckling of the cylinder and the spalling of the lining at

TABLE 2—DEFLECTION OF THE CENTER OF THE 20-IN. PIPE RELATIVE TO THE ENDS OF THE SPAN DURING THE BEAM TEST

Load	Deflection
Lb.	In.
5,000	0.02
10,000	.04
20,000	.12
30,000	.20
40,000	.36
50,000	1.14
54,000	1.70

the top of the pipe. The maximum load supported by the 30-in. pipe was 140,000 lb. Failure occurred by the cylinder tearing at the bottom.

The deflections of the 20-in. pipe under load are given in Table 2.

Crushing tests

The data from the measurements of the strains at various points in the pipes are shown in Fig. 3 and 4 for one specimen of 20-in. and one of 30-in. pipe. The locations of the points at which strains were measured are shown in Fig. 1. The strains given for the inside of the lining at the top and bottom include the widths of the cracks at those points. After the lining cracked, the widths of the cracks in inches are approximately twice the value of the strain for the 20-in. pipe because of the 2-in. gage length and five times the value shown for the 30-in. pipe because of the 5-in. gage length (Fig. 3 and 4, Y_4 and Y'_4).

The maximum loads sustained by the 20-in. pipe specimens were 49,800 and 48,500 lb. or an average of 12,300 lb. per ft. of length. The maximum loads for the 30-in. pipe specimens were 51,500 and 52,000 lb., or an average of 13,000 lb. per ft. of length. The specimens failed by the crushing of the concrete of the lining under the top loading block.

Tests of materials of the pipes

The tensile strength, the yield strength, the modulus of elasticity and the elongation of the wire and of specimens of the steel cylinder are given in Table 1.

The average compressive strength of the 3.7- by 2.1- by 1.8-in. was 7,600 psi. and of the 2.1- by 2.1- by 1.8-in. prisms was 10,100 psi.* The modulus of elasticity of the taller prisms was found to be 4,200,000 psi.

The prestrain in the lining at the inner surface was found to be 0.00046 (point X_4, Fig. 1), at the outer surface, adjacent to the cylinder (point X_3)

*The 28-day strength of 6- by 12-in. cylinders, made from concrete of the lining before it was placed in the pipe, was reported by the District of Columbia to have ranged from 3,000 to 5,000 psi, with an average of about 4,000 psi.

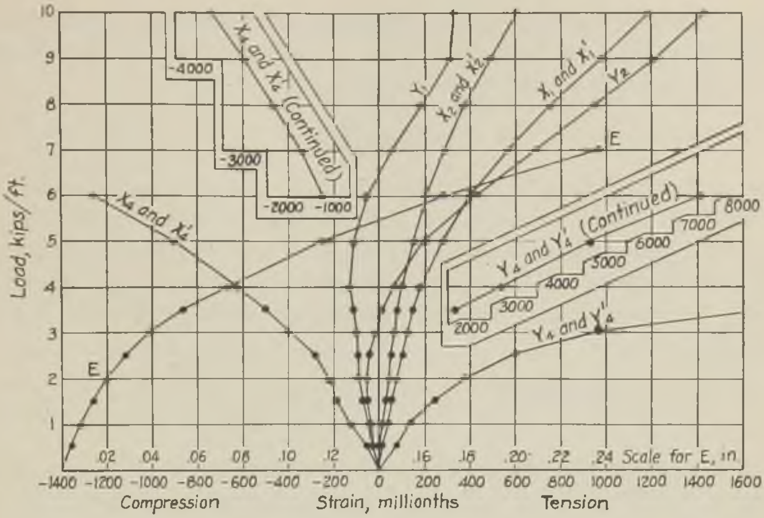


Fig. 3—Change of the strain at various points of the 20 in. pipe during the crushing test.

Y_1	wire at the top.	X_1&X'_1	wire at the sides.
Y_2	cylinder at the top.	X_2&X'_2	cylinder at the sides.
Y_4&Y'_4	inner surface of lining at top and bottom.	E	decrease in vertical diameter.
X_4&X'_4	inner surface of lining at sides.		

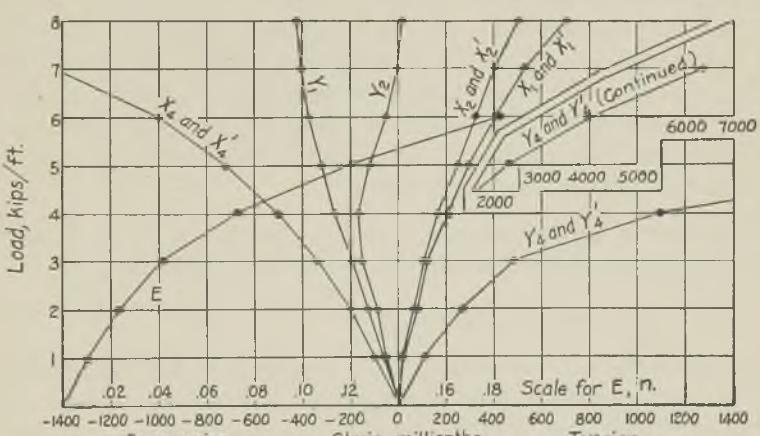


Fig. 4—Change of the strain at various points of the 30-in. pipe during the crushing tests.

Y_1	wire at the top.	X_1&X'_1	wire at the sides.
Y_2	cylinder at the top.	X_2&X'_2	cylinder at the sides.
Y_4&Y'_4	inner surface of lining at top and bottom.	E	decrease in vertical diameter.
X_4&X'_4	inner surface of lining at sides.		

0.00010, in the cylinder (point X_2) 0.00020, and in the wire (point X_1) to be 0.00237. The stress in the wire when it was wound was reported by the District of Columbia to have been 86,000 psi.

DISCUSSION

The design of this type of prestressed concrete pipe is similar to that described by Crepps*, except that the effect of the cylinder must also be considered. The spacing and size of the wire and the prestresses may be chosen so that there will be no circumferential stress in the concrete at the working pressure. Under these conditions, the working pressure is given by (Table 1):

$$P_w = \frac{-n\,f'_c\,(A_s + A_b + A_c/n)}{r}$$

It was reported that the pipes tested had been designed for a working pressure of 192 psi (112 psi operating head plus 80 psi allowance for water hammer head), based on slightly different values for the constants than those given in Table 1. Pressure in excess of this value existing for short periods should not injure the pipe, as any small cracks in the lining would close again when the excessive pressure is removed. However, pressures or loads which cause the strain in either the wire or the cylinder to exceed their elastic limits might cause permanent cracks in the concrete. An inspection of Fig. 2 indicates that the yield point of either the wire or cylinder was reached at a pressure of about 500 psi in the 20-in. pipe and at about 400 psi in the 30-in. pipes. The values of the yield point and the prestrain of the wire and cylinder of the 30-in. pipe indicate that the change in slope of the pressure-strain curves at about 400 psi. pressure was caused by the yielding of the cylinder. A less well marked change in slope of the pressure-strain curve for the 30-in. pipes may be noted between 150 and 200 psi, which should indicate approximately the pressure at which the concrete cracked.

When there is no pressure acting on the pipe, the prestress in the wire balances the prestress in the cylinder and lining. Therefore

$$f'_s A_s + f'_b A_b + f'_c A_c = O$$

The stress in the wire when the pipe is wound must be higher than the desired final prestress because of the allowance which should be made for shrinkage, plastic flow, and the elastic deformation of the cylinder and lining. For the 30-in. pipes, the winding stress of 86,000 psi may be compared with the stress computed from the measured prestrain of 65,000 psi.

*Wire-Wound Prestressed Concrete Pressure Pipe, Ray B. Crepps, ACI JOURNAL, June 1943; *Proceedings*, V. 39, p. 545.

However, the latter prestress value would probably be higher if the lining were wet.

The pressure which will cause these pipes to burst can be estimated approximately from the properties of the wire and material of the cylinder from the equation

$$P_m = \frac{T_s A_s + Y_b A_b}{r}$$

This gives P_m as 705 psi for the 20-in. pipe and 555 psi for the 30-in. pipes. These pressures were exceeded in the tests.

While the tests were made primarily to investigate the properties of the pipes, it may be noted that the joints also successfully withstood the maximum applied pressures.

To facilitate selective distribution, **separate prints** of this title (42-4) are currently available from ACI at 50 cents each—quantity quotations on request. **Discussion** of this paper (copies in triplicate) should reach the Institute not later than Mar. 1, 1945

Title 42-4 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 42**

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
(copyrighted)

Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **September 1945**

Field Use of Cement Containing Vinsol Resin*

By CHARLES E. WUERPEL†
Member American Concrete Institute

SYNOPSIS

The results obtained from 22,398 test specimens manufactured in connection with extensive construction, principally during the period 1941-1944, are presented together with a discussion of the experience with handling concrete containing over 2,000,000 barrels of Vinsol resin cement during this period in 168 structures. Comparisons are drawn between concrete made with plain cement and with cement interground with Vinsol resin as they affect the compressive and flexural strength, the unit weight and the mixing, placing and finishing operations.

INTRODUCTION

The use of portland cement containing Vinsol resin, as an interground addition, in field construction was inaugurated within the Department in April 1941 in the concrete walls of a flood control structure at Corning, N. Y. This type of cement was used upon the recommendation of this laboratory as an experiment designed to reduce to a practicable minimum the excessive bleeding and resultant sand-streaking which occurred in the vertical sections of the walls when plain cement was used. The experiment was a success and additional cement of this type was used in a flood wall at Binghamton, N. Y., commencing in May 1941.

The highly salutary effect of Vinsol resin, as an interground addition to portland cement, upon the durability of concrete obtained by this laboratory in tests at Treat Island, Maine (1*), together with information received of similarly desirable properties exhibited in experimental highway projects (2, 3, 4, 5, 6, 7, 8, 9) and the practical improvements of workability and bleeding in the two flood protection structures led to

*Received by the Institute Jan. 8, 1945.
†Principal Engineer, Central Concrete Laboratory, North Atlantic Division, Corps of Engineers, U. S. Army, Mount Vernon, New york.
*Figures in parentheses refer to bibliography appended to this paper.

TABLE 1—CEMENT COMPANIES AND MILL LOCATION FROM WHICH VINSOL RESIN CEMENT HAS BEEN SHIPPED TO PROJECTS

Cement Company	Mill Location
Allentown Portland Cement Co.	Evansville, Penn.
Alpha Portland Cement Co.	Cementon, N. Y.
Alpha Portland Cement Co.	Jamesville, N. Y.
Coplay Cement Co.	Coplay, Penn.
Federal Portland Cement Co.	Buffalo, N. Y.
Giant Portland Cement Co.	Egypt, Penn.
Glens Falls Portland Cement Co.	Glens Falls, N. Y.
Hercules Portland Cement Co.	Stockertown, Penn.
Huron Portland Cement Co.	Oswego, New York
Keystone Portland Cement Co.	Bath, Penn.
Lawrence Portland Cement Co.	Northampton, Penn.
Lawrence Portland Cement Co.	Thomaston, Maine
Lehigh Portland Cement Co.	Alsen, New York
Lehigh Portland Cement Co.	Buffalo, New York
Lehigh Portland Cement Co.	Fogelsville, Penn.
Lehigh Portland Cement Co.	Fordwick, Virginia
Lehigh Portland Cement Co.	Ormrod, Penn.
Lone Star Cement Corp.	Hudson, New York
West Penn. Cement Co.	Butler, Penn.
National Portland Cement Co.	Brodhead, Penn.
Nazareth Cement Co.	Nazareth, Penn.
North American Cement Corp.	Alsen, New York
North American Cement Corp.	Howe Caves, New York
Penn-Dixie Cement Corp.	Bath, Penn.
Penn-Dixie Cement Corp.	Clinchfield, Ga.
Penn-Dixie Cement Corp.	Nazareth, Penn.
Penn-Dixie Cement Corp.	Portland Point, N. Y.
Universal Atlas Cement Co.	Hudson, New York
Universal Atlas Cement Co.	Leeds, Alabama
Universal Atlas Cement Co.	Northampton, Penn.
Whitehall Cement Co.	Northampton, Penn.

cement of this type being specified for the runways, roads and buildings of the Rome Air Depot, Rome, N. Y. in September 1941. The 435,706 barrels of SS-C-206a cement containing Vinsol resin (a similar specification to that later accepted by the A.S.T.M. as C 175-42T) used in the various elements in this project up to date was produced by 13 mills operated by 9 companies and constituted the first use of this type of cement in major amounts in this country.

The excellent results obtained during the construction of the three projects discussed above led to a more general requirement for this type of cement in construction by the Department in the northeastern portion of the United States. Up to August 24, 1944, 2,037,851 bbl. of cement containing Vinsol resin have been tested by this laboratory at 31 mills for use on 168 separate projects. The mills which have manufactured the cement are listed in Table 1. The types and number of structures in which the cement has been used are listed in Table 2. It is the purpose of this paper to present the results obtained from tests on the numerous specimens prepared in the field and a discussion of the experience gained with this type of cement in concrete construction. It is proposed to

TABLE 2—TYPES AND NUMBER OF STRUCTURES IN WHICH CEMENT CONTAINING VINSOL RESIN HAS BEEN USED

Type of Structure	Number
Pavements (Access roads and airport runways)	70
Floors	
Hangar	7
Warehouse	39
Building	76
Walls	
Flood	8
Warehouse	38
Building	76
Retaining	11
Sea	1
Target Butts	8
Foundations	
Gun Blocks (Major)	4
Footings	100
Total Number	438

present the results of the extensive laboratory investigations conducted concurrently in a separate paper.

Cement containing Vinsol resin has been used in field construction only during the period of the national emergency when demands for speed were great, the supply of skilled inspectors and technicians was absorbed largely into the armed forces and equipment was frequently below standard. Accordingly, the control of concrete mixtures and the accuracy with which records were kept was imperfect. Concrete made with plain cement was not subject to the effect of this lowered efficiency to the degree that prevailed for concrete containing Vinsol resin; use of plain cement was not new and was reasonably well standardized. Use of cement containing Vinsol resin was new and its effect on mixture design, the behavior of the plastic mixture and on the hardened concrete was not well understood. Frequently, cement containing Vinsol resin was placed in mixtures which had been designed for plain cement. In most of these cases, some adjustment of the mixture proportions was made because of the radical increase in consistency or slump, but the nature of the adjustment was not always recorded and may have been a simple reduction in water content, or cement content.

Except for the inadequacy of field records, the irregularities of design and their effects on the hardened concrete might have been analyzed with greater benefit from the experience. However, without such records, it is possible only to present the test results as obtained and use limited instances, where the causes of abnormalities were known, to guess the cause for other similar and sometimes dissimilar abnormalities.

The importance of the effect of the amount of entrained air on the strength of the concrete was not recognized during the early use of Vinsol resin. Accordingly, no routine efforts were made or considered necessary to determine or regulate the quantity of entrained air by field tests on the plastic concrete. The necessity for measures of control beyond those provided by the cement specification (quantitative limit on Vinsol resin content) was demonstrated when very low strengths developed in the concrete placed in the aprons and turn-arounds at Grenier Field, Manchester, N. H., during the autumn of 1942. Density tests of cores extracted from the work early in 1943 indicated an air content in some locations as high as 15 per cent and a general air content of approximately 11 per cent.

A generally similar experience occurred in the autumn of 1942 in the development of abnormally high air contents and correspondingly low compressive and flexural strengths in sections of the runways at Trumbull Airfield, Groton, Conn., in which Vinsol resin was an interground addition to the cement.

These two major and three other minor occurrences of an abnormal air content, focused the attention of the laboratory on the importance of field tests for measurement and control of the air content. The fact that the cements used on these and all other projects were tested by this laboratory and found to contain Vinsol resin in amounts within the limits specified (0.025 — 0.045 per cent by weight of cement) and to develop acceptable strength in mortar tests, indicated strongly that limiting the amount of interground Vinsol resin was inadequate for desired results. Laboratory investigations were started to determine why certain cements caused excessive air-entrainment while others with a similar or lesser amount of Vinsol resin caused normal air entrainment. The nature of these investigations, and the efforts made to eliminate this variable and the results obtained were presented in the A.S.T.M. Bulletin for Oct. 1944 (10) and in the 1944 Annual Report of Committee C-1 of the A.S.T.M. (11). Amplification of the laboratory work will be presented in a later paper.

With the conditions of imperfect records in mind as a limitation of accuracy, the results of tests made on field concrete are presented.

STRENGTH

Field control cylinders*

Test cylinders representing concrete placed on projects constructed by the Districts of the North Atlantic Division and certain districts of other Divisions, are forwarded to this laboratory for test. Between January 1937 and March 1944, 37,910 such cylinders were tested, of which 16,819 were tested at the age of 28 days. Information regarding each cylinder is required to be furnished by the project to the laboratory. On the basis of this information, the data have been classified for analysis in the present

*The data referred to are taken from a report entitled "Compressive Strength at 28-Days Age of Job-Made Concrete Cylinders Containing Treated and Untreated Portland Cement" under the date April 1944 which was distributed in September 1944 within the Department by this office (12).

TABLE 3—NUMBERS OF SPECIMENS TESTED AT 28-DAYS

Classified by districts in which made, type of cement used and period in which tested

Period	1937-1943		1943-1944		1937-1944		Total
District	Plain	V. R.*	Plain	V. R.	Plain	V. R.	Total
Baltimore	1601	....	406	....	2007	....	2007
Boston	1383	69	599	66	1982	135	2117
New York	1997	20	583	645	2580	665	3245
Philadelphia	318	....	577	158	895	158	1053
Providence	3184	93	404	33	3588	126	3714
Syracuse	3307	938	138	300	3445	1238	4683
Total	11790	1120	2707	1202	14497	2322	16819
Total	12910		3909		16819		....

*Use commenced April 1941.

paper. For the purpose of compressive-strength analysis, classification groups have been made on the following bases:

(1) Reported water-cement ratio,
(2) Reported type of cement used (plain or containing Vinsol resin), and
(3) Period in which tested (before or after March 1943).

As additional information, a classification of the specimens as to source, number and type is given by districts in Table 3.

(a) *Water-cement ratio.* The number of specimens of concrete made with plain cement and cement containing Vinsol resin reported for the various water-cement ratios is given in Table 4 and indicated graphically in Fig. 1. The largest groups of specimens made with plain cement and with cement containing Vinsol resin are found to have reported water-cement ratios of 6.0 and 5.0 gal. per bag, respectively. This evidence reported by the field substantiates laboratory experience which has indicated that concrete of similar cement factor and degree of workability can be obtained with cement containing Vinsol resin using a water-cement ratio approximately one gallon per bag less than would be needed with plain cement.

(b) *Compressive strength.* A tabulation of average compressive strengths of cylinders classified by water-cement ratio, type of cement, and period in which tested, is given in Table 5. The relationship between compressive strength and water-cement ratio for each type of cement, in each period, and for the data as a whole, are indicated by the graphs in Fig. 2. Fig. 2 (D) shows the comparability of the compressive strength-water-cement ratio relationship for VR treated and plain-cement concrete when proper cognizance is taken of the effect of VR in increasing workability as mentioned in (a) above. Also shown on these graphs is the curve indicating the values assumed by this laboratory for the compressive strengths normally to be anticipated at 28-days for concrete made with plain

TABLE 4—NUMBERS OF SPECIMENS TESTED AT 28-DAYS

Classified by Reported Water-Cement Ratio, and Type of Cement Used

Reported Water-Cement Ratio	Plain Cement	Cement with V. R.
gal. per bag		
3.0	22	...
3.5	3	14
4.0	200	101
4.5	278	105
4.75	....	182
5.0	903	853
5.25	20	57
5.5	3943	648
5.75	93	..
6.0	7373	277
6.25	7	4
6.5	1136	35
6.75	195	3
7.0	203	41
7.5	86	1
8.0	31	....
8.5	3	....
9.5	1	....
11.0	....	1
Total	14497	2322

cement conforming to Federal Specification SS-C-206a and the water-cement ratios shown.

Notes to Table 5 indicate the selectivity exercised in the plotting of the data in Fig. 2. Five other values (Note 3, Table 5) were modified or omitted prior to the plotting of Fig. 2. In four of these cases evidence cast serious doubt on the accuracy of the reported water-cement ratio for cylinders from one particular project or district. In these cases, the doubtful value was omitted. In the fifth case, a peculiarly faulty distribution of data occurred which, due to the domination of successive weighted averages by data from different sources, gave an anomalous relation. These data are tabulated:

	Plain Cement — 1943-44			
	W/C = 7.5		*W/C = 8.0*	
	No. of Tests	*psi.*	*psi.*	*No. of Tests*
Baltimore	3	4360	4135	2
New York	20	2570	——	—
Providence	6	3285	3275	16
Weighted averages	——	2905	3370([a])	—

([a]) Omitted value.

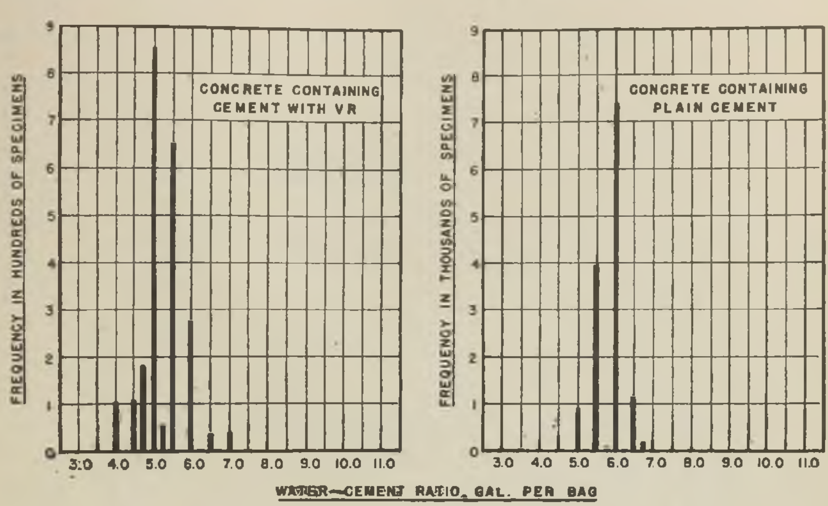


Fig. 1—Relation of frequency of specimens to water-cement ratio of concrete containing cement with and without vinsol resin.

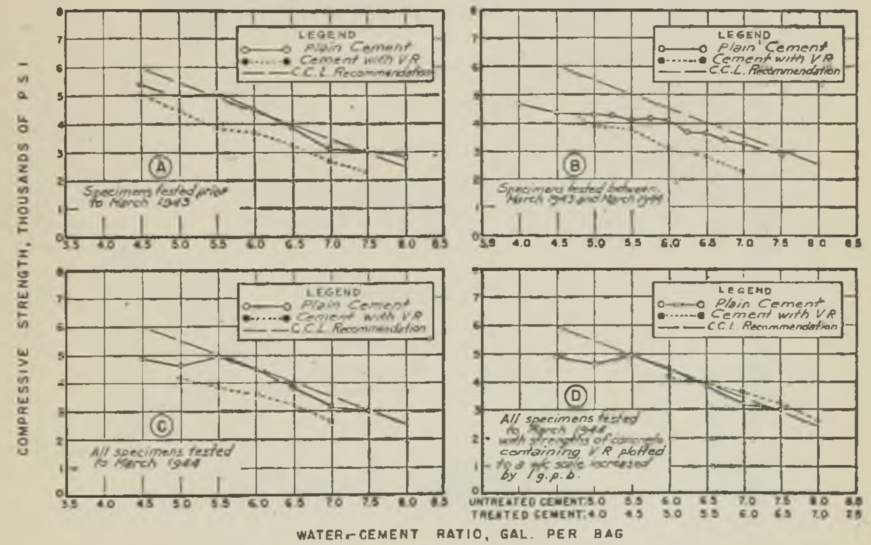


Fig. 2—Relation of compressive strength of job cylinders to water-cement ratio of concrete containing cement with and without vinsol resin (age: 28 days)

It is probable that all tabulated values are incorrect with respect to the reported water-cement ratio, however, the general average value of 2905 psi, for a water-cement ratio of 7.5 gallons per bag is considered to be reasonable.

TABLE 5—AVERAGE COMPRESSIVE STRENGTHS OF SPECIMENS TESTED AT 28 DAYS

Classified by reported water-cement ratio, type of cement, and period in which tested.

Period: 1937 to March 1943

Plain Cement							Cement with V. R.						
Report. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Disc-arded	Remain-ing Spec.	Corr. Comp. Str. psi	Notes	Reprt. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Disc-arded	Remain-ing Spec.	Corr. Comp. Str. psi	Notes
3.0	1	7000	1	0	....	(*)							
3.5	3	3750	3	0	....	(*)							
4.0	62	4385	62	0	....	(†)							
4.5	150	5350	0	150	5350	....	4.5	1	5090	0	1	5090	....
5.0	512	4590	128	384	4960	(‡)	5.0	410	4495	0	410	4495	....
5.5	3405	5065	0	3405	5065	....	5.5	437	3920	0	437	3920	....
6.0	6594	4540	0	6594	4540	....	6.0	219	3760	0	219	3760	....
6.5	869	3875	0	869	3875	....	6.5	15	3265	0	15	3265	....
7.0	122	3130	0	122	3130	....	7.0	37	2680	0	37	2680	....
7.5	57	3025	0	57	3025	....	7.5	1	2250	0	1	2250	....
8.0	13	2790	0	13	2790	....							
8.5	1	3435	1	....	....	(*)							
9.5	1	910	1	....	....	(*)							
Total	11790	....	196	11594	....	....	Total	1120	......	....	1120	....	....

*Insignificant representation.
†Inaccurate W/C data, water in aggregate ignored.
‡Special Cases—see text.

Table 5 continued on opposite page

TABLE 5—Continued from opposite page

Period: March 1943 to March 1944													
Plain Cement							Cement with V. R.						
Report. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Dis-carded	Remain-ing Spec.	Corr. Comp. Str. psi	Notes	Report. W/C Ratio	No. of Spec.	Comp. Str. psi	No. Dis-carded	Remain-ing Spec.	Corr. Comp. Str. psi	Notes
3.0	21	3475	21	0	....	(†)	3.5	14	3275	14	0	....	(†)
4.0	138	4710	0	138	4710	....	4.0	101	3590	101	0	....	(†)
4.5	128	4360	0	128	4360	....	4.5	104	3780	104	0	....	(†)
5.0	391	4310	0	391	4310	....	4.75	182	4275	0	182	4275	....
5.25	20	4265	0	20	4265	....	5.0	443	3965	0	443	3965	....
5.5	538	4100	0	538	4100	....	5.25	57	4415	57	0	....	(‡)
5.75	93	4155	0	93	4155	....	5.5	211	3780	0	211	3780	....
6.0	779	4105	0	779	4105	....	6.0	58	3035	0	58	3035	....
6.25	7	3665	0	7	3665	....	6.25	4	3460	4	0	....	(*)
6.5	267	3620	0	267	3620	....	6.5	20	3370	18	2	2810	(‡)
6.75	195	3405	0	195	3405	....	6.75	3	3420	3	0	....	(*)
7.0	81	3715	24	57	3260	(‡)	7.0	4	2245	0	4	2245	....
7.5	29	2905	0	29	2905	....	11.0	1	1930	1	0	....	(*)
8.0	18	3370	18	0	....	(‡)							
8.5	2	3375	2	0	....	(*)							
Total	2707	....	65	2642	....	....	Total	1202	....	302	900	....	....
Both Periods: 1937 to March 1944													
4.5	278	4895	0	278	4895	....							
5.0	903	4470	128	775	4630	....	5.0	853	4220	0	853	4220	....
5.5	3943	4935	0	3943	4935	....	5.5	648	3875	0	648	3875	....
6.0	7373	4495	0	7373	4495	....	6.0	277	3607	0	277	3607	....
6.5	1136	3815	0	1136	3815	....	6.5	35	3325	18	17	3210	....
7.0	203	3365	24	179	3170	....	7.0	41	2635	0	41	2635	....
7.5	86	2985	0	86	2985	....							
Total	13922	....	152	13770	....	....	Total	1854	....	18	1836	....	....

*Insignificant representation.
†Inaccurate W/C data, water in aggregate ignored.
‡Special Cases—see text.

TABLE 6—RELATIONSHIP OF WATER-CEMENT RATIO AND CEMENT FACTOR TO COMPRESSIVE STRENGTH OF JOB CYLINDERS—28 DAYS—ROME AIR PORT

Cement Factor	Water—Cement Ratio						
	4.8	5.0	5.25	5.5	5.75	6.0	6.5
4.5	....	....	....	....	....	3475(88)	....
5.0	....	3960(26)	....	4190(21)	3980(111)	3345(10)	3870(22)
5.5	....	4210(59)	....	3560(78)	....	3045(57)	....
5.75	5040(141)	....	....	....	....	....	....
6.0	....	4570(51)	4690(55)	4275(59)	....	....	....

NOTE: Figure in parentheses following each compressive strength value indicates number of cylinders averaged.

By the exercise of the selectivity described, the values were obtained from which the relatively smooth curves shown in Fig. 2 (A) and (B) were constructed to indicate the relationship between compressive strength and water-cement ratio for the two periods. A comparison of the differences between the curves for comparable cement type for the different periods indicates the necessity of caution in combining the data to obtain the trend of the data as a whole. The combined weighted-average compressive-strength values tabulated in the lower part of Table 5 and plotted in Fig. 2 (C) and (D), represented only those water-cement ratios for which, for either type of cement, a representative number of specimens were tested during each period.

Of all of the available data in Table 5 the majority was used in preparing the graphs in Fig. 2 as the following figures indicate:

Graph	Percentage of all available data included	
Fig. 2 (A)	98	97
Fig. 2 (B)	91	
Fig. 2 (C) and (D)	93	

In view of the wide variety in type of structure built with cement containing Vinsol resin at the Rome Air Depot and the considerable amount of cement used therein, a summary of the strengths developed by the various mixtures used is given in Table 6. The next largest use of this type of cement was in the Stewart Field area, Newburgh, N. Y., where 419,913 barrels from 12 mills have entered six major projects to date. The details on strength developed by the concrete control cylinders in these projects are given in Table 7.

(c) *Type of cement.* In the foregoing reference has been made to apparent inaccuracies in reporting water-cement ratios. Similarly, it is believed that, in some cases, incomplete information was given on the type of cement used. Since the "Field Data" report form issued by this laboratory prior to the compilation of data did not require, specifically, that the presence of additions to the cement be recorded, it is believed that greater likelihood exists that specimens containing cement with

TABLE 7—MIXTURE DATA AND STRENGTHS OF CONTROL SPECIMENS AIR-FIELD PAVING—STEWART FIELD AREA

Project	W/C	C.F. bags	Compressive Strength of Test Cylinders at Ages Shown, psi. 7 day No.	7 day Max.	7 day Min.	7 day Aver.	28 day No.	28 day Max.	28 day Min.	28 day Aver.	90 day No.	90 day Max.	90 day Min.	90 day Aver.	Coarse Aggregate
NY-73	5.0	6.2	..	..	..	..	24	4250	2145	3490(*)	..	..	..	..	Gravel
	5.0	6.1	..	..	..	..	18	5605	2520	4130(†)	..	..	..	..	Gravel
NY-149	4.75	6.0	..	..	..	..	145	6640	2825	4205	..	..	..	..	Crushed
	5.0	6.0	..	..	..	..	53	5860	3125	4185	..	..	..	..	Dolomite
NY-544	5.0	6.1	82	4250	1645	3100	155	6250	2655	4600	11	6000	3535	4685	Dolomite
NY-193	5.0	6.0	..	..	..	..	25	4535	2590	3415	..	..	..	..	Dolomite
NY-324	4.8	6.7(‡)	..	..	..	..	38	5645	3820	4650	..	..	..	..	Dolomite
NY-325	4.8	6.5(‡)	..	..	..	..	50	5770	3395	4450	..	..	..	..	Dolomite
T & WA (‡)	4.9	6.2	82	4250	1645	3100	508	6640	2145	4305	11	6000	3535	4685	

Project	Flexural Strength (Modulus of Rupture) of Test Beams, psi. 7 day No.	7 day Max.	7 day Min.	7 day Aver.	28 day No.	28 day Max.	28 day Min.	28 day Aver.	60 day No.	60 day Max.	60 day Min.	60 day Aver.	90 day No.	90 day Max.	90 day Min.	90 day Aver.
NY-149	..	..	..	..	..	..	..	..	..	..	..	..	28	790	525	665
NY-544	77	904	350	560	131	904	416	690	10	820	630	745	32	895	525	720
T&WA (§)	77	904	350	560	131	904	416	690	10	820	630	745	60	895	525	695

*1942
‡High cement content made necessary by extreme harshness imparted by flat, angular and elongated stone.
§Totals and Weighted Averages
† 1943

Vinsol resin have been included in the plain cement group than vice versa. Since no specimens containing Vinsol resin were reported from the Baltimore District, a spot check was made of data from that source to endeavor to uncover an example of this condition. In the case of one project (BA-31), the cement shipment records indicate the exclusive delivery of cement containing Vinsol resin to the project in cars, the numbers of which are given in the "Field Data" reports as sources for cement used in the cylinders. The "Field Data" reports however, failed to state the fact that the cement contained Vinsol resin.

Airport evaluation program

A total of 236 slabs of concrete, varying from 3 ft. by 3 ft. to 6 ft. by 8 ft. were removed from the runways or aprons at 26 airports in the northeastern portion of the United States, in connection with an airport pavement evaluation program. The slabs, removed after periods in place varying from six weeks to several years, afforded opportunity to obtain additional information on the strength of pavement concrete made with cement containing Vinsol resin. Also, some indication of the effect of Vinsol resin in the cement as compared to plain cement and the effect of the type of coarse aggregate was developed. These relationships however, must be interpreted with caution, because the various conditions of cement type and coarse aggregate type seldom existed on the same project. Thus the important effects of brand of cement, water-cement ratio, cement content, fine aggregate, weather conditions, age, type of curing, contractor's equipment and personnel, and engineering control were not and could not be evaluated. For instance, the abnormal air-entrainment developed on two of the jobs (Grenier Field and Trumbull Airport) in which gravel was used probably would not have been different had crushed stone been used. In another case, extremely variable water control was indicated on one of the older plain cement jobs where gravel was used as coarse aggregate. It just happens that no such major variables are known to have occurred on the projects in which crushed stone was used as the coarse aggregate.

The concrete slabs were trucked to this laboratory where 6 in. by 36 in. beams were extracted by sawing and 6 in. diameter cores were extracted by means of a diamond core drill. The depths of the beams and lengths of the cores, being functions of the depth of the slabs, varied from 4 in. to 11 in. Where the slab thickness exceeded 6 in., the excess was removed by sawing. Where the thickness was less than 6 in. no adjustment was made other than a correction for h/d in the cores. Cores were tested at the greatest lengths practicable after removing major irregularities by sawing. Some of the slabs contained wire-mesh reinforcement, but the effects of such reinforcement on the strengths of the beams, cubes and cores were minimized if not removed entirely by selective sawing and drilling. In general, 5 beams and 3 cores were extracted from each slab. After the beams had been tested in flexure, compression tests were made on the broken pieces (modified cube). The tests on the beams, modified cubes and cores were conducted in accordance with prevailing A.S.T.M. Standards with the following exceptions:

(a) The total breaking loads on the modified cubes were corrected by the factors given in A.S.T.M. C 42-42 (Sec. 7) to eliminate the effect of varying depths of specimens on the apparent compressive strength.

(b) The beams tested in flexure in the later portion of the program were tested with the surface as cast in compression. This was done on orders from the Office, Chief of Engineers. There was no detectable evidence that the orientation of the beam in the testing machine affected the apparent flexural strength of the concrete.

Character of concrete. It was not possible to obtain a precise description of the proportions used in all of the concrete pavements, but it can be stated that the basic policy of the specifications for this class of work was adhered to in a majority of cases. On this basis, the concrete in the pavements contained coarse aggregate of 2 in. maximum size; an average cement content of 6.0 bags per cu. yd., varying in certain cases, from 5.5 to 6.7 bags per cubic yard (with the higher factors being used, generally, in connection with crushed stone coarse aggregate); a slump averaging 2½ in., and water-cement ratios varying from a minimum of 4.7 gals. per bag for cement containing Vinsol resin to a maximum of 6.3 gals. for plain cement. The age of the concrete slabs varied widely—from a minimum of 45 days to a maximum of several years. However, the influence of the age factor is not considered to be significant because all types of concrete are represented in essential equality in the age range.

Flexural strength—sawed beams. The results obtained from the 1524 flexural tests (third point loading) on sawed beams are summarized in Table 8 by projects, basic types of cement and basic types of coarse aggregate. These same data are abstracted in Table 9.

Compressive strength—modified cubes. The results obtained from the 1441 compressive tests made on modified cubes from sawed beams are summarized in Table 8 by projects, basic types of cement, and basic types of coarse aggregate. These data are abstracted in Table 10.

Compressive strength—cores. The results obtained from the 437 cores drilled (diamond bit) from the slabs are summarized in Table 8 by projects, basic types of cement and basic types of coarse aggregate. These data are abstracted in Table 11.

Miscellaneous pavement cores. In addition to the cores taken from the Pavement Evaluation slabs, a total of 2177 6-inch diameter cores were extracted from pavements in situ at military airports in this region. These cores were tested in compression in a moist condition after their bulk displacement had been determined. The strength data and certain

TABLE 8—SUMMARY OF STRENGTH AND DENSITY DATA OBTAINED FROM CONCRETE SLABS TAKEN FROM AIRFIELD PAVEMENTS

Field	No. of Slabs Tested	Flexural Strength, psi. Sawed Beams				Compressive Strength, psi. Modified Cubes from Sawed Beams				Compressive Strength, psi. 6-in Diamond Drilled Cores				Density—Cores Actual Unit Weight lb./cu. ft.			Ref. See Foot Notes
		No. of Tests	Max.	Min.	Aver.	No. of Tests	Corrected to h/w = 2 Max.	Min.	Aver.	No. of Tests	Corrected to h/d = 2 Max.	Min.	Aver.	Max.	Min.	Aver.	
Plain Portland Cement—Gravel Corase Aggregate																	
Millville	6	54	810	460	635	54	7500+	4145	5655	18	7555	5045	5970	153.1	147.5	151.2	
Dover	8	77	760	450	625	76	6825	4670	5795	24	6395	4840	5550	153.7	149.4	151.2	(1)
New Castle	2	18	735	530	645	18	5425	4425	4945	6	5320	4565	5100	—	—	—	
Dow	6	59	1105	610	800	59	7930	4340	5525	18	6200	4215	5400	153.7	146.2	150.3	
Presque Isle	6	50	825	610	700	50	6175	4370	5285	18	6530	4515	5475	151.2	147.5	148.3	
Mitchell	28	272	1035	220	645	260	7960	2070	5450	83	8350	2245	5680	159.4	136.7	147.3	(2) (3)
Green	11	89	830	460	670	75	6870	3015	5445	26	9215	3200	6675	154.4	145.6	151.0	
Westover	2	14	760	515	645	10	6200	4415	5105	0	—	—	—	—	—	—	
Stewart	1	10	1150	735	900	10	5630	3520	4870	3	6090	5950	5995	153.7	146.9	151.4	(4)
Bedford	4	40	965	590	780	20	5745	3330	4820	12	8345	6025	6940	144.1	143.1	143.6	(5)
Grenier	3	30	810	455	635	30	6285	3495	4925	9	6680	5180	5915	165.6	150.0	152.5	(1)
Totals and Averages	77	713	1150	220	700	662	7960	2070	5345	217	9215	2245	5825	159.4	136.7	149.7	
Revised (See Note 8)	74	683	1150	220	705	632	7960	2070	5290	208	9215	2245	5865	159.4	136.7	149.4	(8)
Plain Portland Cement—Crushed Stone Coarse Aggregate																	
Middletown	4	40	1095	770	980	30	6525	4670	5640	10	7985	4025	5815	156.2	151.2	154.0	
Suffolk	10	92	995	535	870	92	7155	3910	5460	29	6595	4450	5660	155.0	148.1	151.3	
Westover	3	26	900	620	750	25	7175	4090	5640	0	—	—	—	—	—	—	
Totals and Averages	17	158	1095	535	865	147	7175	3910	5580	39	7985	4025	5735	156.2	148.1	152.7	
Cement with Vinsol Resin—Gravel Coarse Aggregate																	
Rye Lake	11	101	750	480	605	103	5895	3550	4795	33	6360	3875	5200	151.9	141.9	148.2	
Fort Dix	7	70	780	470	600	70	5925	2460	4630	21	6105	3800	5010	147.5	144.4	146.1	
Trumbull	6	60	710	275	530	60	6375	1775	4255	0	—	—	—	—	—	—	
Stewart	5	50	1125	540	730	40	6675	2885	4770	15	6290	3745	5150	146.4	135.4	141.9	(2) (6)
Grenier	5	48	680	220	470	48	5925	1075	3270	15	6680	1520	3550	151.2	128.9	140.9	
Totals and Averages	34	329	1125	220	585	321	6675	1075	4345	84	6680	1520	4785	151.9	128.9	144.3	
Revised (8)	29	281	1125	275	615	273	6675	1775	4615	69	6360	3745	5120	151.9	135.4	145.4	(8)

—Table 8 continued on opposite page

TABLE 8—Continued from opposite page

Field	No. of Slabs Tested	Flexural Strength, psi — Sawed Beams: No. of Tests	Max.	Min.	Aver.	Compressive Strength, psi — Modified Cubes from Sawed Beams: No. of Tests	Corrected to h/w = 2: Max.	Min.	Aver.	6-in Diamond Drilled Cores: No. of Tests	Corrected to h/d = 2: Max.	Min.	Aver.	Density—Cores, Actual Unit Weight, lb./cu. ft.: Max.	Min.	Aver.	Ref. See Foot Notes
Cement with Vinsol Resin—Crushed Stone Coarse Aggregate																	
Stewart	10	99	820	650	710	89	5845	3880	4820	30	6450	4070	5115	156.2	149.4	152.1	
Galeville	7	70	940	660	785	70	7165	5250	6035	21	8485	5545	6565	154.4	146.2	150.5	
Montgomery	7	67	995	520	795	67	7260	3780	5840	21	7890	4750	6675	156.2	148.7	154.0	
New Hackensack	4	40	1010	710	850	40	5925	4200	5045	12	6970	5720	6335	153.7	149.4	151.4	
Westover	4	17	1030	580	865	14	7080	3935	5740	0	—	—	—	—	—	—	
Rome	3	31	960	730	845	31	6845	4595	5915	3	6705	4385	6100	151.9	150.0	150.6	(7)
Totals and Averages	35	324	1030	520	810	311	7260	3780	5565	97	8485	4070	6160	156.2	146.2	151.7	

NOTES:
(1) Includes two locations in which the pavement contained wire-mesh reinforcement.
(2) Includes one location in which no modified cube tests were made because beams were less than 5-in. thick.
(3) Includes 11 locations in which the pavement contained wire-mesh reinforcement.
(4) High-Early-Strength cement used.
(5) Includes two locations in which no modified cube tests were made because beams were less than 5-in. thick.
(6) Includes one location in which 1 per cent $CaCl_2$ was added in addition to Vinsol resin.
(7) Includes three locations in which the pavement contained wire-mesh reinforcement.
(8) Omitting data from Grenier Field.

TABLE 9—MODULUS OF RUPTURE—SAWED BEAMS

Third point loading; Age: Greater than 45 days

	Plain Cement		Cement + V.R.	
	Gravel	Stone	Gravel	Stone
No. of tests	713	158	329	324
Max. strength, psi	1150	1095	1125	1030
Min. strength, psi	220	535	220	520
Aver. strength, psi	700	865	585	810
Mean aver. (gravel & stone) psi	780		700	
Mean aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	90			
Weighted aver. (gravel & stone)	730		695	
Weighted aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	95			

TABLE 10—COMPRESSIVE STRENGTH—MODIFIED CUBES

Age: Greater than 45 days

	Plain Cement		Cement + V.R.	
	Gravel	Stone	Gravel	Stone
No. of tests	662	147	321	311
Max. strength, psi	7960	7175	6675	7260
Min. strength, psi	2070	3910	1775	3780
Aver. strength, psi	5345	5580	4345	5565
Mean aver. (gravel & stone) psi	5460		4955	
Mean aver. strength of concrete containing V.R. as *per cent* of con crete containing plain cement	91			
Weighted aver. (gravel & stone) psi	5390		4945	
Weighted aver. strength of concrete containing V.R. as *per cent* of concrete containing plain cement	92			

statistical constants are abstracted in Table 12. It will be observed that the general average strength of the cores containing Vinsol resin is 93.5 per cent of the cores containing plain cement in spite of inclusion in the average of cores taken from Grenier and Trumbull where the air content was abnormally high. It will be observed, further, that the standard deviation and coefficient of variation of the strengths of the cores containing Vinsol resin are somewhat lower than they are for the strengths of cores containing plain cement.

The frequency distribution of the compressive strengths of the cores from four projects is plotted in Fig. 3. These data will be referred to

TABLE 11—COMPRESSIVE STRENGTH—AIRPORT EVALUATION CORES

Age: Greater than 45 days

	Plain Cement		Cement + V.R.	
	Gravel	Stone	Gravel	Stone
No. of tests	217	39	84	97
Max. strength, psi	9215	7985	6680	8485
Min. strength, psi	2245	4025	1520	4070
Aver. strength, psi	5825	5735	4785	6160
Mean aver. (gravel & stone) psi	5780		5470	
Mean aver. strength of concrete containing Vinsol resin as *per cent* of concrete containing plain cement	95			
Weighted aver. (gravel & stone) psi	5810		5520	
Weighted aver. strength of concrete containing Vinsol resin as *per cent* concrete containing plain cement	95			

later in connection with a discussion of the strength-density relationship.

Discussion. The data on strength presented above should be evaluated with the following factors in mind:

A very large number of tests (22,398) is included; all made by the same laboratory.

The field control specimens were molded from concrete actually placed in structures and stored initially in the field under varying conditions of temperature and moisture.

Other specimens were removed from the finished structures and, therefore, subject to the variables of composition and curing which are not characteristic of laboratory test specimens.

Data on water-cement ratio and type of cement were obtained from field reports.

With the foregoing in mind, the test data indicate the following:

(1) An approximate correlation exists between the compressive strength and reported water-cement ratio of job-made cylinders. This relationship of strength to water-cement ratio is in reasonable accord with that assumed by the laboratory as representing the strengths which might normally be expected.

(2) A lower order of compressive strengths was indicated for concrete placed in 1943-44 than for that placed prior to 1943.

This is regarded as reflecting (1) the use of more coarsely ground cement manufactured under WPB Limitations Order L-179, and (2) the effect of emergency conditions on the quality of field control; especially, on the accuracy with which specified water-cement ratios were maintained and reported in practice, since the decrease in compressive strengths was

TABLE 12—SUMMARY OF COMPRESSIVE STRENGTH DATA—MISCELLANEOUS DRILLED CORES

Concrete Pavements

Project Name	No. of Cores	Compressive Strength, psi. Max.	Min.	Aver.	Stan. Dev. psi.	Coeff. of Var. —
CEMENT WITH VINSOL RESIN						
Westover Field	20	8755	4950	6765		
Trumbull Airport	22	5000	1560	3210*		
Grenier Field	100	6490	1415	3570*		
Fort Dix Airfield	44	6520	2430	4015		
Rome Air Depot	232	8480	4040	6575	699	10.6
Rye Lake Airport	165	6025	2575	4260		
Stewart Field (Apron)	86	6445	2255	4390		
Stewart Field (Runways)	565	8515	3105	5340	972	18.2
Galeville Airfield	310	7985	3785	6130	929	15.2
Montgomery Airfield	203	6540	3480	4820	586	12.2
New Hackensack Airfield	40	6065	3885	4900		
West Point Roads	25	7190	1940	4655		
Syracuse Air Base	21	7535	4970	6365		
Totals and Averages	1924	8755	1415	5290	861	15.4
PLAIN CEMENT						
Westover Field	15	8615	3200	6480	1854	28.6
Grenier Field	12	6275	3230	5115	908	17.7
Bedford Airport	10	7230	3510	5765	1230	21.3
Suffolk Airport	20	5360	3800	4580	479	10.5
Dow Field	30	7065	4880	5980	573	9.6
Mitchel Field	152	7835	2315	5660	858	14.4
Rome Air Depot	14	8120	4915	6560	1481	22.3
Totals and Averages	253	8615	2315	5655	1055	17.8

(1) Abnormally high air content.

greater in the lower range of reported water-cement ratios than in the normal or higher range.

The anomalies in the data indicate the importance of proper direction of inspecting personnel to determine and report the *actual net water-cement ratio* entering the concrete sampled for testing.

(3) Since the largest number of specimens containing Vinsol resin were at a water-cement ratio one gallon per bag less than that at which the largest number of plain cement specimens are found, and since the cement factors and slumps are similar, it is concluded that the field evidence substantiates the laboratory experience that, when cement con-

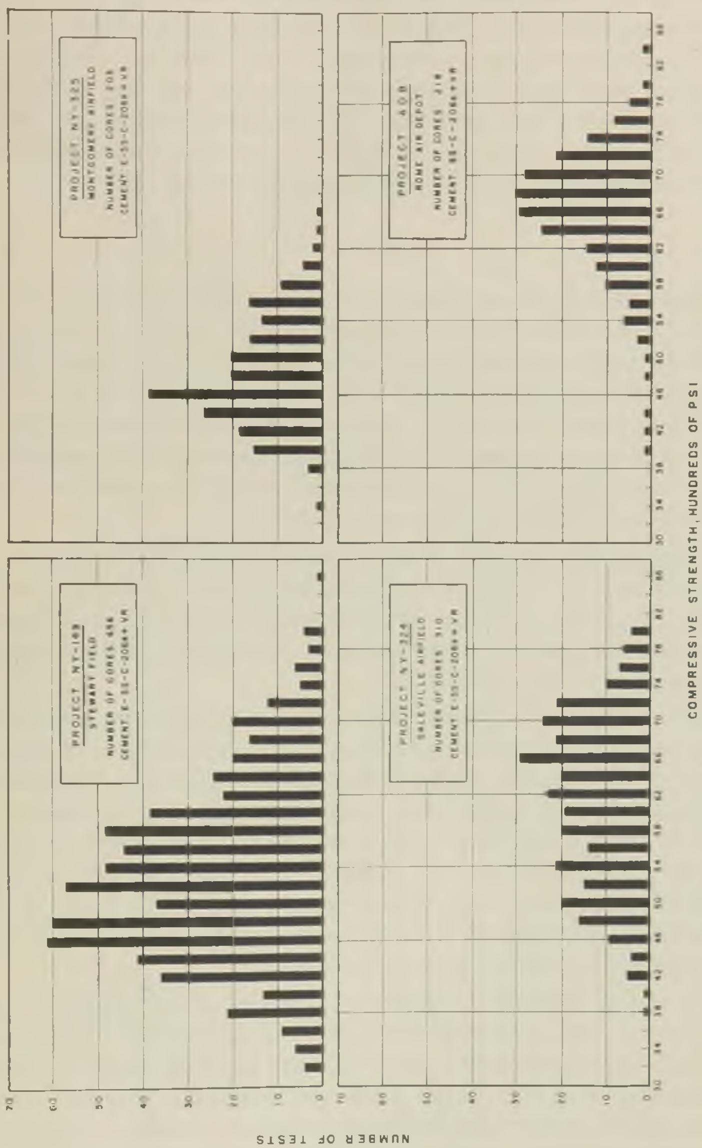


Fig. 3—Frequency distribution of compressive strengths of concrete cores drilled from airfield pavements.

taining Vinsol resin is used with a water-cement ratio one gallon per bag lower than would be required for plain cement, an equivalent degree of workability can be obtained with similar cement factors.

(4) When recognition is given to the statement in (3) above, and the strength values for concrete containing Vinsol resin are plotted to a water-cement ratio scale increased by one gallon per bag from those reported to have been used, and when abnormal air contents are avoided, it is evident that the strengths developed by concrete containing Vinsol resin are comparable to those of plain cement concrete.

DENSITY

No regular attempt having been made by the field inspection forces to determine the amount of air entrainment in the plastic concrete, fragmentary data only are available on the air content of the concrete on the numerous projects where cement containing Vinsol resin was used. Available data came, principally, from the projects where spot checks were made at the specific request of this office. Except for the two projects mentioned available evidence indicates that the air content was in the range 3.5—7.0 per cent of the concrete volume.

Recent instructions require determination of the air content of the plastic concrete as a regular and routine procedure. However, for the purpose of this paper, principal dependence must be placed on density determinations on hardened specimens for information on the amount and uniformity of air-entrainment.

The density of the hardened concrete was determined in conjunction with compressive strength tests on 6-in. diameter cores taken from airport pavements. Determinations were by displacement and values expressed in terms of weight in pounds per cubic foot. These data are summarized in Table 8, for cores taken in the pavement evaluation program, and in Table 13 for additional cores taken in the field from pavements in the Stewart Field, New York, area. The air contents shown in Table 13 were calculated from the theoretical unit weight of the concrete, based on mixture proportions, and the actual unit weight.

Tables 8 and 13 indicate the degree of variation in unit weight in each of the projects. The frequency distribution of unit weight of the cores, taken from the four projects from which the greatest number of cores were extracted, is shown in Fig. 4. These data, together with the strength data shown in Fig. 3, were used to prepare the scatter diagrams (Fig. 5) as the relationship between compressive strength and actual unit weight of the concrete tested. The correlation coefficients calculated from these data are indicated in the illustration by numerical values and by regression lines drawn through the scatter diagrams. Additional strength-density

TABLE 13—DENSITY DATA—DRILLED CORES FROM AIRFIELD PAVEMENTS IN STEWART FIELD AREA

Cement: E-SS-C-206a + VR

Project	W/C	C.F.	Actual Unit Weight, lb./cu.ft.				Approx. Th. Unit Wt., lb./cu.ft.	Calculated Average Air Content per cent
			No.	Max.	Min.	Aver.		
Stewart Apron (NY-73)	5.0	6.1	75	148.1	132.5	142.2	149.8	5.1
Stewart Runways (NY-149)	4.8	6.0	529	158.1	146.9	152.6	159.3	4.2
New Hackensack (NY-193)	5.0	6.0	65	157.9	146.0	151.6	158.8	4.5
Galeville Runways (NY-324)	4.8	6.7	243	156.2	143.7	150.5	154.9	2.8
Montgomery Runways (NY-325)	4.8	6.5	217	157.6	144.4	151.6	157.3	3.6
Area Totals and Average	...	...	1129	.....	.....	.....	.....	4.0

*Some specimens tested in compression without unit weight determination; some cores upon which unit weight determinations were made were tested other than in compression.

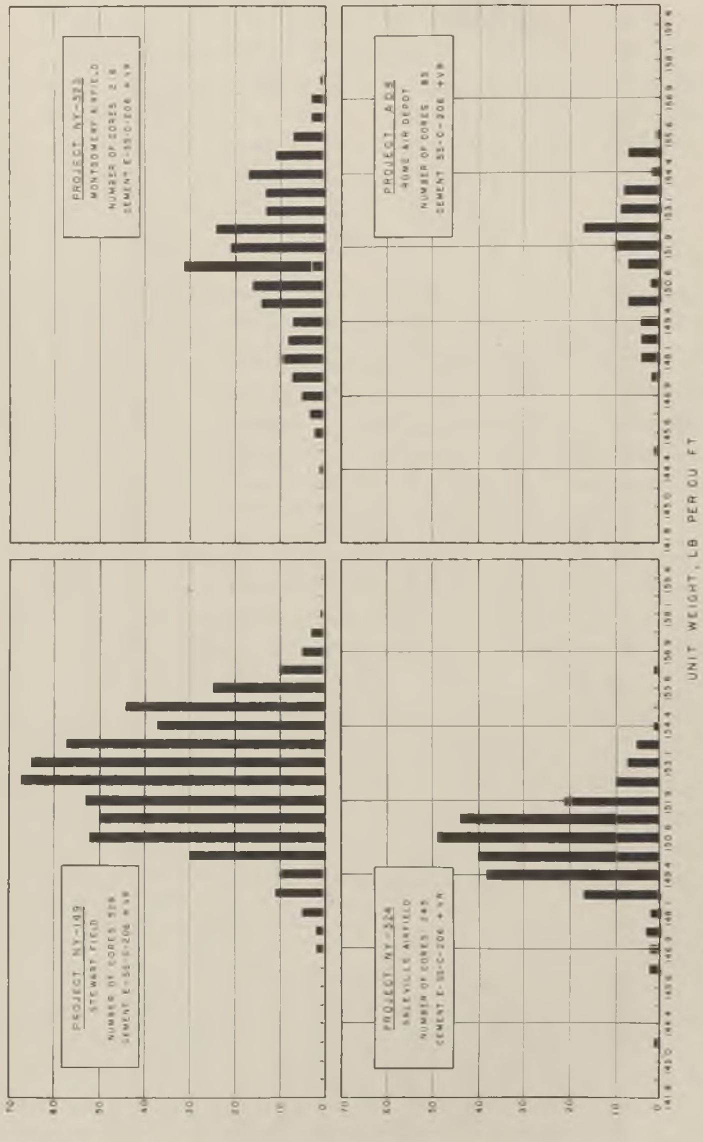


Fig. 4—Frequency distribution of unit weights of concrete cores drilled from airfield pavements.

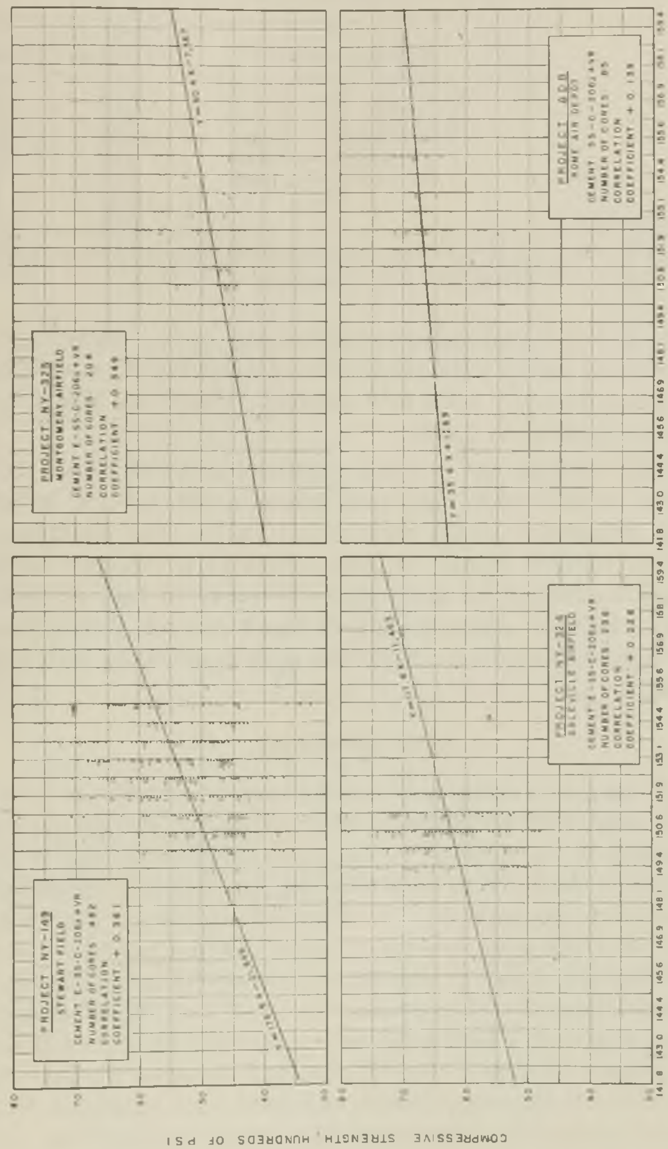


Fig. 5—Relationship between compressive strength and actual unit weight, concrete cores.

data are plotted in Fig. 6 and 7 based on cores extracted from Grenier Field and Trumbull Airport, respectively.

The regression equations and coefficients of correlation are:

Source	Regression equation	Coeff. of correlation	Number
B. of R. (*)	Y = 197 X − 24,400	0.60	46
NY-149	Y = 178.5 X − 21,849	0.36	482
NY-324	Y = 117.8 X − 11,463	0.23	236
NY-325	Y = 80.4 X − 7,367	0.35	204
ADB	Y = 35.6 X − 1,289	0.14	85
Grenier Field	—	0.90	97
Trumbull Airport	—	0.94	28

(*) Fig. 3, p. 1005 Proc., A.S.T.M. v. 43 (1943) Wing, S.P.; Jones, Valens; and Kennedy, R.E. "Simplified Test for Evaluating the Effectiveness of Concrete Mixers". Data represents plain cement and are included for comparison with the mixtures containing Vinsol resin.

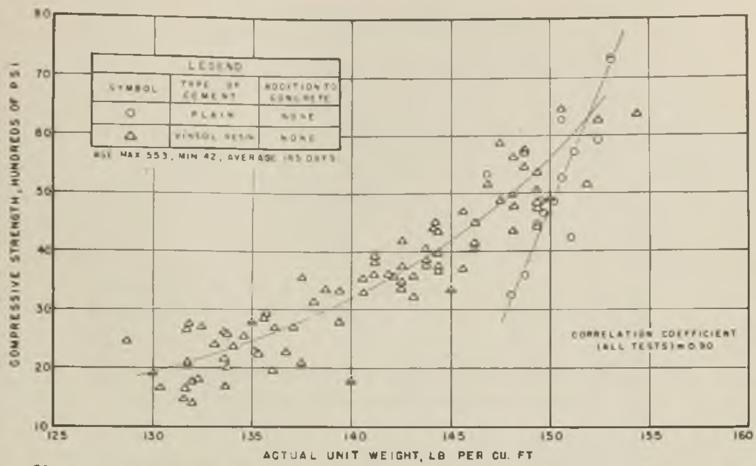
The data in Fig. 8 represent the relationship, or lack thereof, between the air content of plastic mixtures and the compressive strength of control cylinders made from the same field batches. This is the single instance in which such data are available.

These data confirm the general relationship which exists between compressive strength and density of concrete, but the low correlation coefficients obtained from the projects having the most significant data indicate that variable or low strengths obtained in concrete containing Vinsol resin should not be attributed arbitrarily to high or materially varying air content. The influence of water control, segregation, and degree of compaction play an important role in attaining high and uniform strength.

FIELD DURABILITY

The only tangible evidences of deterioration of concrete in field structures built since 1940 and brought to the attention of this office are in surface scaling in the runways at the Rome Air Depot and, to a much lesser extent, in the apron at Stewart Field, and rather severe cracking of the pavement at Grenier Field. The scaling at Rome was limited mainly to the area in which plain cement was used and in a large portion of the area in which one brand of cement containing Vinsol resin was used. Small isolated areas of scaling exist in the areas where three other brands of cement containing Vinsol resin was used. The scaling at Rome was made the subject of a special report entitled "Rome Air Depot—Surface Scaling of Concrete Runways" which was circularized within the Engineer Department by this laboratory under the date of October 1943.

The conclusion drawn in the report was that the scaling of the pavement is primarily of the "manipulation" type and the principal influences in its development, in their order of importance, were:



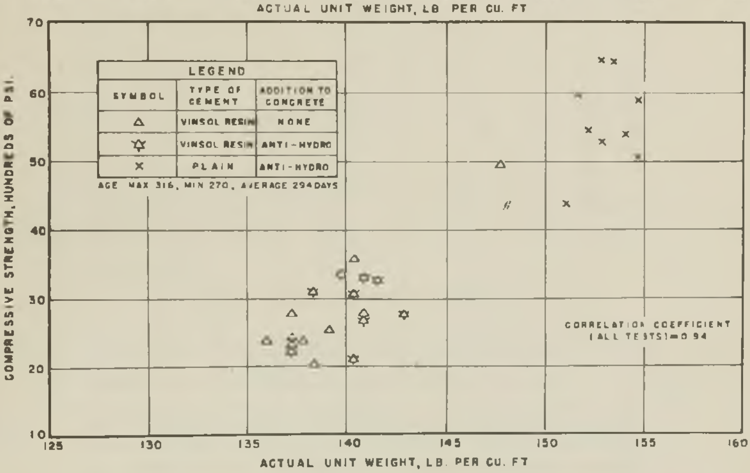


Fig. 6 (top)—Relationship between compressive strength and actual unit weight, concrete cores, Grenier Field, Manchester, N. H.

Fig. 7 (bottom)—Relationship between compressive strength and actual unit weight, concrete cores, Trumbull Airport, Groton, Conn.

(1) Excessive manipulation of the highly plastic mixture during the finishing operation.

(2) Exposure to frequent and severe freezing and salt applications very shortly after the end of the 14-day curing period.

(3) Inadequate air-entraining reaction of the Vinsol resin in one cement, or basic inferiority in that cement.

The scaling has been progressive in extent. The condition of the scaling in July 1943 and in October 1944 is summarized in Table 14. These data indicate clearly that despite the considerable extent of scaling in the con-

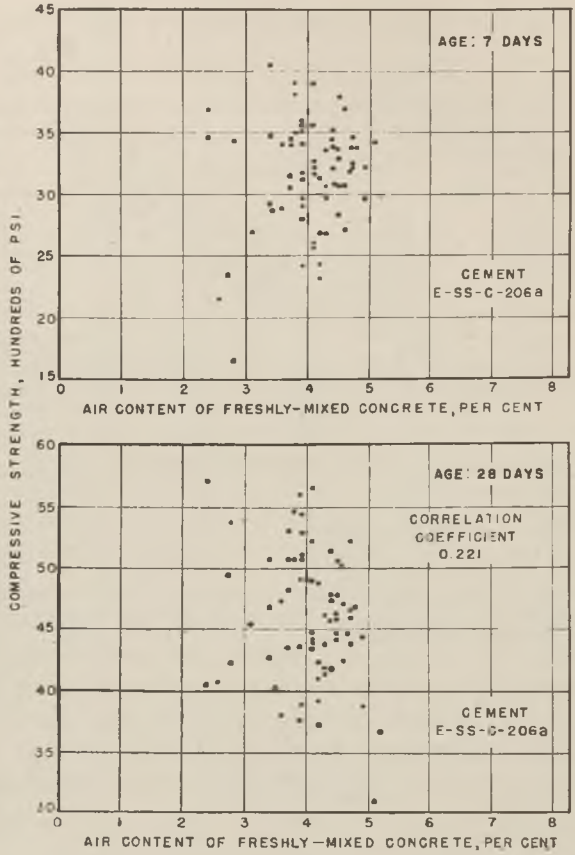


Fig. 8—Relationship between compressive strength of that cylinders and air content of freshly mixed concrete—N Y 544. Additional paving—Stewart Field landing mat.

crete containing Vinsol resin, this type of concrete is greatly superior to concrete made with the plain cement.

The scaling which has occurred in a small portion of the section of Stewart apron placed in 1942 varies from insignificant to moderate and appears to have occurred only in areas placed during or just prior to freezing temperatures without subsequent protection, or in areas where the concrete was placed on very wet and soft sub-grades or during rain.

The cracking at Grenier Field is believed to be due (1) to structural weakness in the concrete due to an excessive air content (2) to the concrete having been placed on a frozen sub-grade, and (3) to excessively heavy equipment running on surfaces placed on deep fill.

The subject of the durability of concrete made with and without Vinsol resin will be discussed in a separate paper.

TABLE 14—ROME AIR DEPOT

Summary of Pavement-Surface Scaling

Cement Brand	Addition	Scaling to July 1943 Blocks* Scaled—% Total (A)	Scaling to July 1943 Blocks* Scaled—% Total (B)	Scaling to Oct. 1944 Blocks* Scaled—% Total (A)	Scaling to Oct. 1944 Blocks* Scaled—% Total (B)	Increase in Scaling Blocks Scaled—% Total (A)	Increase in Scaling Blocks Scaled—% Total (B)
1	VR†	50.8	0.73	66.5	0.83	15.7	0.10
2	VR	1.6	0	6.0	0.10	4.4	0.10
3	VR	1.6	0	15.4	0.50	13.8	0.50
4	VR	1.4	0.1	5.6	0.40	3.2	0.30
1	—	93.5	19.4	96.8	37.10	3.3	17.70
1	Nat‡	83.3	8.1	99.1	18.3	15.8	10.20
2	VR+Nat**	0	0	5.4	0	5.4	0

*Block = 12.5 ft. by 20 ft.
†VR = Flake Vinsol resin interground at mill.
‡Nat. = 1 bag Natural cement replacing 1 bag of plain portland cement in mix.
**VR + Nat. = 1 bag Natural cement replacing 1 bag portland cement containing VR in mix.
(A) =Thin "manipulation" type scale.
(B) = Deeper progressive scale.

TIME AND TYPE OF MIXING

Attention was focused on the possible effect of time and type of mixing on the air content and strength of concrete containing Vinsol resin when it was determined that transit mixing (not agitation) of 20 minutes duration was used at Grenier Field during the period in which excessively high air contents were obtained. Information solicited from other agencies at that time indicated that mixing in transit mixers with sealed drums might be the cause of excessive air-entrainment in the concrete. A survey of military projects, using transit mixers and cement containing Vinsol resin, in this region, produced vague and conflicting information due to the incompleteness of records and the use of many different types of transit-mixers, each type differing somewhat in drum design, discharge mechanism, condition of blades, venting or water control. Additional variables were time of agitation (as distinguished from time of mixing), time of mixing and length of haul, plus brand of cement and type of aggregate.

An investigation of the conditions of mixing at Grenier Field in the spring of 1943 included the use of several brands of cement in which the Vinsol resin * content varied. The sand content of the mixture was varied

*The flake or un-neutralized form.

from 35 to 25 per cent by weight of the aggregate. The mixing was done in the contractor's horizontal-discharge, closed-drum type of transit mixer. The time of mixing and load in the mixer were varied. In this investigation, it was observed that;

(a) Time of mixing had no great effect on the amount of entrained air, the air content of all mixtures being abnormally high irrespective of mixing time.

(b) Load in the mixer, up to the limit specified by the manufacturer, had no great effect on the amount of entrained air.

(c) The brand of cement had more effect upon the amount of entrained air than any other factor when Vinsol resin was present, .

(d) Reducing the amount of Vinsol resin to the minimum permitted by the specifications* was inadequate to hold the air content in the concrete to the desired limit with the cements and mixers used.

(e) The air content was affected appreciably by the sand content of the mixture (increasing in proportion with the sand content)..

(f) The homogeneity of the mixture obtained with the transit mixers used was unsatisfactory without excessive mixing time (10-15 minutes) or unless the drum load was considerably reduced from the rated capacity.

Additional but less formal investigations made subsequently at various other projects using transit mixers indicated that the air content might increase with mixing time to a varying extent. No data were obtained on the effect of mixing time on air content of concrete mixed in pavers or in stationary mixers because the mixing time was constant at 1 or 1½ minutes with this type of equipment. However, laboratory studies will be discussed in a subsequent paper.

The large amount of paving since 1941 under emergency conditions resulted in the use of many types and conditions of paving mixers and transit mixers; with pavers and transit mixers occasionally operating together and in at least one case supplying concrete to parallel strips of pavement. Full advantage could not be taken of the opportunity for careful recording of results due to shortage and inexperience of inspectors and the general rush of construction, but the overall experience with the various types of mixers handling concrete with plain cement or with cement containing Vinsol resin may be summed up, as follows:

(a) Less difficulty was experienced with pavers than with any type of transit mixers in obtaining the quality of concrete desired — with or without Vinsol resin.

(b) Due, possibly, to fortuitous circumstances; no concrete placed with pavers developed excessive air content or strengths lower than specified. The mixing time for pavers was 1½ minutes.

*Equivalent to A.S.T.M. C 175-42 T.

(c) Due, possibly, to inadequate maintenance conditions, occasional difficulties in consistency control were experienced with dual-drum pavers when portions of the second batch intermingled with the first.

(d) Good results were obtained throughout the placement period in some projects where transit mixers were used, especially where the mixing was accomplished after arrival of the truck at the point of deposit and vent holes were kept open and mixing time was limited. At one such job, Fort Dix, the taxiways were of concrete mixed in pavers and the aprons and hardstandings constructed, simultaneously, of concrete from transit mixers. Efforts toward control of air content and other characteristics of the mixture were particularly vigorous on this job, and the mixing time in transit mixers was held to a minimum. The average data for the two projects, based on information from field reports, indicate the results obtained were similar (Table 15).

(e) The most frequent complaints on the performance of transit mixers concerned inability to control water content with resultant variations in consistency and strength.

(f) Performance in transit mixers appears to be improved when the slump is increased to about 4 inches, permitting the mixture to surge in the drum; and when the load is reduced from 57.5 per cent to approximately 40 per cent of the volume of the drum.

(g) It has not been practicable to establish that venting of the drum has any effect on the amount of air entrained in the concrete.

(h) In general, the high-discharge type of transit mixers appears to handle the drier consistency mixtures more effectively than the horizontal-discharge type.

(i) Study of the mixing action in various types of transit mixers indicates that, as a class, they were not designed to produce rapid mixing of low-consistency concrete. The concrete occupies space greater than the radius of the drum when the load is up to the manufacturer's normal recommendation of 57.5 per cent of the volume of the drum. The spiral mixing blades are less than ½ radius deep, therefore, the slow revolution of the drum results in general folding action with a slight forward component when the relatively dry consistency typical of paving mixtures is used. This is in contrast to the paving mixer in which the mixing action is by "bucket throw" and the mixing blade area is considerably greater, the speed of revolution is faster and the volume ratio of concrete to drum is much less than in transit mixers. The rolling and folding action in a transit mixer appears to entrap rapidly a considerable amount of air even though the uniform distribution of the mixture components may not be complete. The writer concludes that the general tendency of transit-mixed

TABLE 15—FORT DIX PAVEMENT DATA

	PH-17—Taxiways	PH-12—Aprons and Hardstandings
Mixers:	Pavers (27E and 34E) Dual-drum	Transit Mixers—(Rex, Jaeger, Smith) horizontal discharge. Quadruple vents kept open and wash water wasted.
Mixing time:	1½ minutes	Aver. 5 min. (3-10 min.)
Design Mix:	1:1.84:4.07	1:1.84:3.97
Slump:	2—2½ in.	2—3 in.
Cement:	SS-C-206a+VR (3 brands)	SS-C-206a+VR (1 brand)
Fine aggregate:	Natural sand (Aver. F.M.: 2.26)	Natural sand (Aver. F.M.: 2.26)
Coarse aggregate:	Uncrushed gravel: 2 in. max.	Uncrushed gravel: 2 in. max.
Design W/C:	4.8 gal. per bag	4.8 gal. per bag
Aver. cement content:	5.8 bags/cu. yd.	5.9 bags/cu. yd.
Air content, per cent:	Aver. 3.7 (3-4.5)	Aver. 4.4 (2.5-5)
Strengths developed:		
Field: (a) Compressive:		
(drilled cores):	4840 psi. (13)*	4425 psi. (6)
(mod. cubes):	4995 psi. (40)	4295 psi. (20)
(b) Flexural: (sawed beams):	635 psi. (40)	595 psi. (20)
Control: (a) Compressive:		
(cylinders—28 days):	3355 psi. (101)	3800 psi. (52)
Standard deviation:	560 psi.	660 psi.
Coeff. of variation	16.7%	17.4%
(b) Flexural:		
(beams—6 in. x 6 in.):	490 psi. (190)	505 psi. (83)
Standard deviation:	67 psi.	93 psi.
Coeff. of variation	13.7%	18.6%
Aver. coeff. of Variation All specimens:	15.2%	18.0%

*Figure in parentheses indicate number of tests.

paving mixtures containing Vinsol resin to have abnormally high air contents is a function of *type* of mixing action rather than *time* of mixing although prolonged mixing may serve further to increase the air content.

PLACING AND FINISHING

Comparative information on placement and finishing of concrete containing Vinsol resin, difficult at best to present in a quantitative manner, is in the writer's experience made more difficult by the fact that the occasions were infrequent when concrete containing plain cement and cement containing Vinsol resin were placed simultaneously or under precisely similar conditions. However, it is the consensus of the project engineers, contractors and inspectors with whom the subject has been discussed that concrete containing Vinsol resin compares with concrete containing plain cement about, as follows:

(a) The slump of the mixture is more sensitive to change in water content.

(b) A mixture having a consistency of 2-in. slump is more readily placeable, that is, more mobile.

(c) Segregation of any component (water, sand or coarse aggregate) is less likely to occur, and when it occurs it is less serious in extent except that segregation of fine aggregate is most likely to occur, because it is most likely to be present in excess of the optimum quantity. In this connection, it should be recalled that the spheroids of entrained air act as a fine aggregate, therefore, the amount of sand usually considered normal for plain cement concrete will be excessive in concrete containing Vinsol resin by about the amount of the entrained air.

(d) Consolidation by vibration is effective, but any type of consolidation should be accomplished only to the degree necessary to effect compaction and with more care than is usually devoted to this important operation.

(e) The concrete mixture is "sticky" and there is a tendency for the surface to tear under the influence of a mechanical screed or finishing machine especially when the slump of the mixture is less than 2 in. There is the further common complaint from contractors using Vinsol resin for the first time that final finishing is more difficult to perform. Actually, the difficulties referred to are due to the basic difference in the nature of the material and can and have been effectively overcome by experience. The tendency to tear can be overcome by shortening the forward motion of the mechanical screed or finisher for each transverse stroke or by maintaining a slump of 2½ to 3 inches, or both. The difficulties in final finishing can be overcome by performing the work earlier than is done normally with plain cement, that is; by materially closing the gap between the preliminary mechanical and final finishing operations. This is made practicable and necessary by the reduced bleeding of Vinsol resin concrete. It appears to be the consensus of those who have learned to handle concrete containing Vinsol resin that the difficulties of finishing are no greater than with plain cement and that the shorter lag between screeding and final finishing results, usually, in an over-all economy.

Of *paramount* importance in the finishing of V. R. concrete is the *amount* of finishing; that is, finishing manipulations should be held to a minimum. The scaling of the concrete in the runways at the Rome Air Depot and a similar type of scaling just reported* at Westover Field, Massachusetts, was due to excessive finishing which resulted in an accumulation of sand mortar at the surface. It is this mortar layer which is scaling off due to different coefficients of expansion of the superficial layer and of the concrete. Reduction of the quantity of sand in the mixture, or of the sand-aggregate ratio, will reduce the tendency toward the formation of the superficial mortar layer, but this precaution should be reinforced by a minimum of passes with the mechanical finisher and manipulations in the floating, belting and final trowelling at edges and joints. These precautions which apply also to plain cement concrete pavements, where "manipulation scaling" is relatively common, are of greater importance when Vinsol resin is used, because of the undersirable and persistent tendency toward excessive use of sand and lack of realization of the deceptively high plasticity of such mixtures.

(f) The rate of early strength gain of the concrete mixture is retarded by low temperature to a slightly greater extent than is usual for plain cement concrete. The retardation usually does not affect the time at which the finishing operation may and should be accomplished, but it may delay the time at which it is desired to remove forms. It is most noticeable in pavement slabs which were not protected from low temperatures after the finishing operation and when it is desired to re-use side-forms within 24 hours.

CLOSURE

The experience gained with concrete containing Vinsol resin in the construction discussed herein has led to the conviction of the writer that the entrainment of minute discrete air voids in concrete in amounts not greater than 7 per cent and not less than 3 per cent (optimum 4 to 5 per cent) by means of an air-entraining agent similar to Vinsol resin constitutes a major improvement in the manufacture of concrete. The tangible benefits during construction are: (1) materially increased plasticity and placeability, and (2) materially reduced bleeding and segregation of coarse aggregate. The benefit to the finished structure is a material increase in durability. The liabilities or hazards are the possibility of excessive entrainment of air and some slight reduction in strength.

Definite progress has been made in minimizing the hazard of excessive air-entrainment by the modification of the cement specification (A.S.T.M.

*Too late for full discussion herein.

Designation: C175) in 1944 to limit the air entrained in a test mortar in lieu of the previous specification, under which most of the construction discussed was built, which placed only a quantitative limit on the Vinsol resin present. Recent experience with cement manufactured under this specification has indicated that normal and reasonably predictable air contents were obtained. Additional insurance against excessive air is afforded by the determination of the air-entraining characteristic of the mixture in the design stage prior to construction, and repeated tests of unit weight (A.S.T.M. Method C 138-44) in the field during construction.

Reductions in strength have been and can be minimized or eliminated by proper reductions in water-cement ratio and sand content in the mixture design and in the field.

However, it must be understood that the practice of air-entrainment produces what is in effect a new type of concrete mixture—one with three aggregates (air, sand and gravel or stone) instead of the conventional two. If one of these aggregates (air) is inflexibly associated with the cement, and therefore non-regulable in the field, full control of the mixture is not available to the engineer. Although fully aware of the usual inconveniences and normally opposed to the use of admixtures in the field, the writer believes that the ultimate benefits potentially available in the practice of air-entrainment will not be realized until the air-entraining agent is treated as an aggregate and batched mechanically, by closely regulable means, in the field.

BIBLIOGRAPHY

1. "Cement Durability Program—First Interim Report" by Charles E. Wuerpel, June 1942.

2. "Chloride Salts—Resistant Concrete Pavements" by Ira Paul; Amer. Assoc. of Highway Officials of the North Atlantic States, 14th Ann. Proc. 1938, pp. 144-167.

3. "Durability of New York State Concrete Pavements" by E. C. Lawton, ACI Journal, June 1939, *Proc.*, V. 35, p. 561.

4. "A Report of Certain Properties of Concrete in which 'Treated' Quick-Hardening Portland Cement Was Used" by G. O. Gardner, Symposium on Specifications for and Additions to Portland Cement, Part II, A.S.T.M. Committee C-1, March 1940 (unpublished).

5. "Experimental Concrete Road, Hudson, New York Plant of the Universal Atlas Cement Company" by O. L. Moore, Symposium on Specifications for and Additions to Portland Cement, Part II, A.S.T.M. Committee C-1, March 1940 (unpublished).

6. "Experimental Concrete Pavement in the City of Minneapolis" by H. G. Erickson, Bulletin of Minn. Fed. of Arch. and Eng. Soc., May 1940, pp. 9-11.

7. "Experimental Data in Connection with Chloride Salts–Resistant Concrete Pavement" by C. C. Ahles, *Explosives Engineer*, Sept. 1940, V. 18, p. 267.

8. "Pavement Scaling Successfully Checked" by O. L. Moore, *Engineering News-Record*, Oct. 1940, V. 125, pp. 471-474.

9. "Utah Studies Pavement Scaling" Anon., *Western Construction News*, Dec. 1940, V. 15, pp. 107-108.

10. "The Reaction of Vinsol Resin as It Affects the Air Entrainment of Portland-Cement Concrete" by Charles E. Wuerpel and Albert Weiner, Bull., A.S.T.M. Oct. 1944, No. 130.

11. "Appendix I—Report on Cooperative Studies of Proposed Method of Test for Air Content of Portland-Cement Mortar" by Sponsoring Committee on Portland Cement of Committee C-1 of the A.S.T.M.; Report of Committee C-1 on Cement, 1944.

12. "Compressive Strength at 28-Day Age of Job-Made Concrete Cylinders Containing Treated and Untreated Portland Cement" by Charles E. Wuerpel, Aug. 1944.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 42

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
(copyrighted)

Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN September 1945

Job Problems and Practice

Five cash awards—$50.00, $25.00 and 3 of $10.00 each are to be made for the best contributions to this department in the current volume year—Sept. 1945 to June 1946.

In JPP many Members may participate in few pages. So, if you have a question, ask it. If an answer is of likely general interest, it will be briefed here (with authorship credit unless the contributor prefers not). But don't wait for a question. If you know of a concrete problem solved —in field, laboratory, factory, or office—or if you are moved to constructive comment or criticism, obey the impulse; jot it down for JPP. Remember these pages are for informal and sometimes tentative fragments—not the "copper-riveted" conclusiveness of formal treatises. "Answers" to questions do not carry ACI authority; they represent the efforts of Members to add their bits to the sum of ACI Member knowledge of concrete "know-how."

Non-skid Concrete Surfacing for Wooden Trestles at Mud Mountain Dam (42-168)

By H. H. ROBERTS*

A concrete-surfaced wooden trestle provided an unusual means of trucking in materials for Mud Mountain Dam, approximately 40 miles south and east of Seattle, Wash. on the White River. It is a rock fill dam and was constructed by the Guy F. Atkinson Co. of San Francisco, for the U. S. Engineers during 1939 through 1941. The purpose of this structure is for flood control, no hydro-electric facilities being provided.

Quarry sites were approximately 4½ miles down-stream from the dam site. Suitable haul roads were built but about 5200 linear feet had to be traversed through sections where standard roadways could not be built, necessitating the use of timber trestles.

Twenty-cubic yard trucks were chosen for the rock haul and the question of skid-proofing the deck and the trestles became an important problem. At first a bituminous surface treatment was considered, but because of the deflection in the deck system, this was rejected. Concrete,

*Chief Engineer, Contractors Pacific Naval Air Bases, Port Hueneme, Calif.

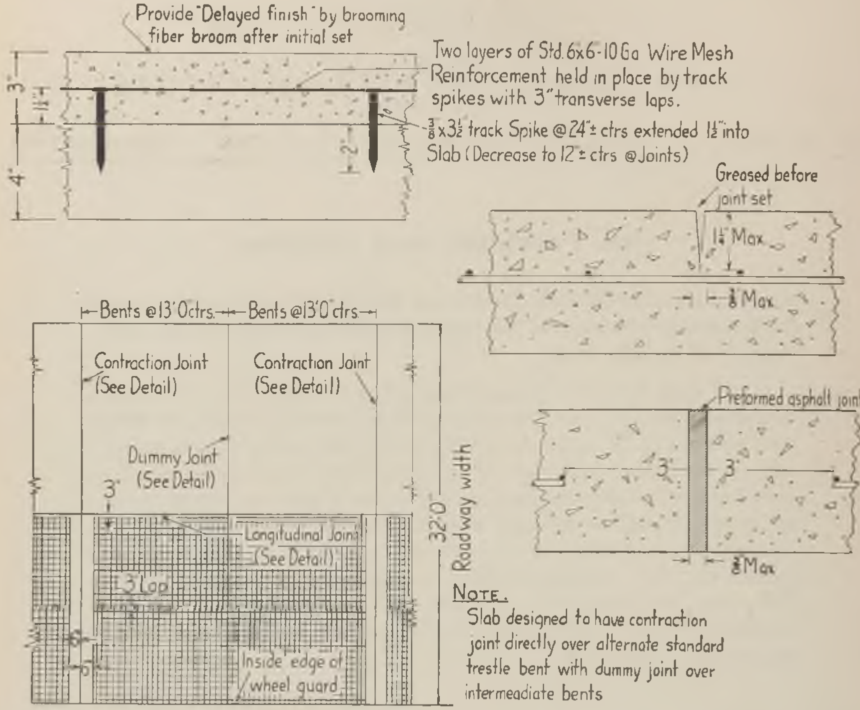


Fig. 1—Three inch concrete deck slab on wooden trestle. The slab section is at upper left; deck plan, standard panel 16 by 26 ft., lower left; top right, detail of typical dummy joint; and typical transverse and longitudinal contraction joint at lower right.

therefore, became our next consideration, and after thorough investigation it was decided that a concrete wearing course 3 in. thick would be built. (Note details Fig. 1).

It was observed that prior to placing the concrete course the deflection at mid-span under a 55 ton load (20 cu. yd. truck loaded) was from 1¼ to 1½ in. After the surfacing was added this was reduced to ¼ to ⅜ in., the concrete developing the full continuity of the deck members. More than 1,000,000 cu. yd. of quarry rock was hauled over these trestles during a six month period and no appreciable damage to the concrete surface nor the deck system was noted. It is true that considerable cracking occurred, but there was little or no concrete removed by the traffic. As to safety, it is interesting to note that there were no accidents caused by skidding during the complete hauling operation.

The cost of the concrete deck system was high but the no-maintenance feature, and the high-speed traffic in wet weather (approx. 110 in. average rainfall per year) justified the cost.

Two types of trestle construction were used: (1) the single lane having a roadway width of 15 ft.-0 in. in the clear and (2) the double lane having a roadway width of 32 ft.-0 in. in the clear. The single lane is supported by a 5-pile bent and the double by a 9-pile bent, all piles being driven to adequate bearing and satisfactory refusal.

Both deck systems were identical, differing only in width. The members used were: 12 x 14 in. caps, 8 x 18 in. stringers (solidly bridged), 3 x 10 in. bracing, 4 x 12 in. decking.

All trestles were surfaced with a 3 in. reinforced concrete slab placed in 26 ft. panels, the panel points and longitudinal joints being provided with a preformed expansion joint filler and dummy joints being formed at the midpoints of the panels. 12 x 12 in. wheel guards were used on both sides of the double and on the outside edge of the single trestles. A 3 ft. walkway on one side only completed the structure. On the walkway side and outside the railing 50 gal. fire barrels and ladders leading to the ground were provided for fire protection. These were placed at approximately 150 ft. centers.

STRUCTURAL QUANTITIES

Unit	Length lin. ft.	Timber f.b.m.	Piling lin. ft.	Wire mesh sq. ft.	Spikes lb.	Exp. jt. filler lin. ft.	Concrete cu. yd
Single Trestle	1912	619,488	25,812	57,360	3,000	3,022	266
Double Trestle	3134	1,711,164	76,783	200,576	8,500	7,134	928
Quarry Road Bridges	158	84,688	5,530				
TOTAL	5204	2,415,340*	108,125	257,936	11,500	10,156	1194

*In addition to the above, approx. 91,250 lb. misc. hardware, including drift pins, bolts, washers and boat spikes were used.

This is a summary of cost of the trestle slab concrete:

Single trestle: $\frac{1912 \text{ ft. x } 15^5 \text{ ft.}}{9}$ = 3293 sq. yd; \$6,324.25 or \$1.92 per sq. yd.

Double trestle: $\frac{3134 \text{ ft. x } 32 \text{ ft.}}{9}$ = 11,143 sq. yd., \$27,229.50 or \$2.44 per sq. yd.

Cost per cu. yd. for material hauled: $27{,}229.50 + 6{,}324.25 = \frac{33{,}553.75}{1{,}000{,}000} =$

\$0.0335 per cu. yd.

A 6½ sack, 1: 2¼: 4¾ concrete at 1 in. slump and using a 1 in. maximum size aggregate was used and was thoroughly vibrated by means of a heavy bull float upon which two "wiggle-tail" air vibrators were mounted. Following the bull float a wood trowel finish was added and all joints were made with a standard edging tool. Prior to initial set the surface was given a rough texture finish with a fiber broom, using a wavy longitudinal motion. This treatment provided an excellent non-skid finish. The surface was then cured, using Hunt process clear, and traffic was not admitted for 120 hours.

The concrete was all mixed in a central plant, transported to the site in dump trucks and dumped and hand shoveled into place. Close inspection during the mixing, placing and finishing resulted in an excellent job, this inspection being accomplished by contractors' forces.

It is the writer's opinion that this thin slab treatment can be used successfully for the surfacing of timber structures, thus insuring a safe wearing surface and almost entirely eliminating maintenance.

The writer designed the slab and supervised the construction while employed as assistant engineer for the Guy F. Atkinson Co., whose project manager was R. H. Northcutt; general superintendent, D. E. Root, and chief engineer, Donald O. Nelson.

Exposure of Concrete to High Temperature (42-169)

Q—I am desirous of obtaining some information regarding the behavior of concrete when exposed to high temperatures. I notice that building codes when specifying concrete for fire protection for steel beams, specify that it shall withstand temperatures varying from 1000 to 2500 or 3000 F. for periods ranging from one to four hours. I also understand that the Underwriters Laboratory have a requirement that concrete shall not be exposed continuously to temperatures greater than 500 F.

What I should like to get is a curve or charts showing the temperatures which concrete will withstand successfully in that interval between a few hours and continuous exposure.

A—While we are unable to find the kind of information you want, we have a letter from a competent ACI Member authority to this effect:

We know of no systematic and comprehensive investigation of the subject, and the available information is meager and that from different sources is somewhat contradictory. See Proceedings of the American Concrete Institute: V. 35, p. 292 and 417 (1939), and V. 36, p. 216 (1940). It is well known that the effects of the exposure of concrete to high temperatures depend significantly upon the kind of aggregate and the duration of the exposure. Concretes containing siliceous aggregates are not likely to lose strength rapidly if exposed to temperatures as high as 700 F, whereas those containing only aggregates of burned-clay, slag, cinder, pumice, limestone, and other non-siliceous materials lose strength slowly at temperatures less than about 800 F. Available information seems to indicate that concretes containing the better types of aggregates for such exposures would be affected by prolonged heating somewhat as follows:

300 F—Only very minor loss of strength; 600 F, possibly 25 percent reduction of strength; 800 F, possibly 40 percent reduction of strength; 1000 F, possibly 50 percent reduction of strength, but may remain intact; 1500 F, ultimately disintegrates.

For extremely high temperature services, the high alumina cement Lumnite is preferred and is used in the refractory concrete of metallurgical and ceramic furnaces.

Derrick Stone and Cobbles in Mass Concrete (42-170)

Q—To questions on the use of derrick stone and cobbles and test cylinders of concrete containing larger aggregate an ACI Member writes:

A—There is seldom any advantage in using derrick stone in mass concrete when cobbles are available which may be incorporated in the mix. The derrick stone requires a duplicate facility for handling only 20 percent of the mass which can usually be secured considerably cheaper by simply mixing and placing that much more concrete. Further, when derrick stones are used, wetter and richer concrete using smaller than 6-in. maximum size aggregate should be used to secure good embedment. Concrete containing 6-in. cobbles has been successfully mixed in a 1-yard paving mixer and probably 4½-inch maximum concrete could be mixed in a ½-yard mixer. However, unless the mixers of the tilting type are used there will be some difficulty discharging this type of concrete as dry as it should be for best results as mass concrete (1½-in. slump with good vibration). Also because batch to batch variations are greater for small batches, the largest practicable mixer is preferable.

Aggregate in test specimens should not be larger than one-third the diameter of the specimen and preferably not larger than one-fourth. As this results in 18 to 24 in. diameter cylinders for 6-in. maximum concrete few specimens of this size are made. Usually 6-in. diameter cylinders are made, sometimes 8-in., and aggregate larger than 2 or 2½ in. is screened or picked out of the concrete making up the specimen. The height of standard specimens is twice the diameter; however, considerable labor is saved on larger diameter cylinders having height only 1.5 times the diameter and results are the same when multiplied by 0.98. There is considerable conflict in data on the factor to apply to small specimens to secure the true strength of mass concrete. Probably the safest procedure is to be guided by results which indicate the mass strength to be somewhat less than that of 6-in cylinders, possibly by as much as 10 to 20 percent. See ACI *Proceedings*, 1935, p. 280.

Based on an average water requirement of 205 lb. per yd. of 6-in. maximum rounded gravel concrete and 220 for similar 4½-in. maximum concrete of 1½-in. slump, and a W/C of 0.65 for both, cement requirements would be 315 and 339 lb. per yd., respectively. Probably the same difference of 15 lb. of water per yd. would apply to other types of aggregate so that the estimated difference in cement content for the two maximum sizes would vary only with the water-cement ratio; the smaller the ratio, the greater would be the difference in cement content.

In some of these and related matters the Concrete Manual of the Bureau of Reclamation, fourth edition, may be of assistance.

Effect of Brine on Concrete (42-171)

Q—To "what might be the effects of concrete exposure to brine," we have this from an ACI Member.

A—It is not considered that dense, high quality concrete will be unfavorably affected by brine unless alternate soaking and drying occurs. The concrete should have a minimum of six sacks of cement per cubic yard of concrete and the water cement ratio, including the free or surface water in the aggregate, should not exceed 6 gal. per sack of cement (W/C = 0.53 by weight). If the fine aggregate available is lacking in fines it may be desirable to add a workability agent such as an air-entraining admixture to insure concrete of excellent workability without the use of excessive mixing water.

If the concrete will be alternately wet with the brine, and dried, some further protection should be provided. A 2-coat treatment of linseed oil is recommended in this case. The concrete should be seasoned and dried before the application of the linseed oil, or else treated with magnesium fluosilicate solution (2 lb. crystals per gallon of water) to "kill" the lime.

A part of PROCEEDINGS OF THE AMERICAN CONCRETE INSTITUTE Vol. 42

JOURNAL
of the
AMERICAN CONCRETE INSTITUTE
(copyrighted)

Vol. 17 No. 1 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN September 1945

Current Reviews

of Significant Contributions in Foreign and Domestic Publications, prepared by the Institute's Reviewers

Effect of type of test specimen on apparent compressive strength of concrete

BRYANT MATHER; ASTM 1945 Preprint. Synopsis by AUTHOR

Two programs of strength testing are reported, in each of which concrete from the same sample or batch was tested using either drilled cores or molded cylinders for comparison with modified cubes. A total of 2375 strength tests were made and from the resulting data it is suggested that the choice of type of specimen may considerably affect the compressive strength value obtained for the concrete.

Stone dust in aggregate

ARNOLD ARNSTEIN, *Concrete and Constructional Engineering*, V. 40, No. 3, (March 1945) pp. 43-44 Reviewed by GLENN MURPHY

The author reports tests of concrete made by the Public Works Department in Tel-Aviv. Although in some of the mixtures the material passing the No. 200 sieve was as much as 27 per cent (by weight) of the fine aggregate (greatly exceeding the 3 per cent limit given in the D. S. I. R. Code of Practice), all but two samples met the strength requirement. The water-cement ratios varied from 0.75 to 1.00 (by weight). The author concludes that the dust may be useful in decreasing harshness of mixtures.

Automatic accelerated freezing and thawing apparatus for concrete

CHARLES E. WUERPEL AND HERBERT K. COOK; ASTM 1945 Preprint. Synopsis by AUTHOR

A detailed structural and operational description is given of a newly developed apparatus for automatically and rapidly freezing and thawing 3½ by 4½ by 16-in. concrete specimens. The temperature of the capacity load of 102 specimens is reversed from 42 F. to 0 F. and from 0 F. to 42 F. in a 2-hr. cycle. Typical results of tests to determine the influence of aggregate type on the durability of concrete made with and without an air-entraining admixture are presented to illustrate the practicability of using the apparatus for acceptance testing based on freezing and thawing.

Portland cement dispersion by adsorption of calcium lignosulfonate

FRED M. ERNSBERGER and WESLEY G. FRANCE. *Industrial and Engineering Chemistry*, Industrial Edition. V. 37, No. 6, pp. 598-600, June 1945. Reviewed by R. N. YOUNG

Turbidity tests utilizing the Wagner turbidimeter were conducted on suspensions of portland cement in water, with and without additions of calcium lignosulfonate. Apparent increase in specific surface with addition of the dispersing agent reached maximum of 48 per cent. Determination of adsorption isotherm of calcium lignosulfonate by cement in water suspension is described, data are given for a typical cement. Experiments employing electrophoresis cell are described. Authors conclude: That the lignin salt is a colloidal electrolyte, that anions of lignin salt are positively absorbed by cement particles, that increase in dispersion of cement is a function of adsorbed dispersing agent.

Studies of concrete with entrained air

DELMAR L. BLOEM and STANTON WALKER, Technical Information Letter 22, 1945, of National Ready Mixed Concrete Association. Reviewed by AUTHORS

Progress report on preliminary group of concrete tests carried out in Research Laboratory of National Ready Mixed Concrete Association at University of Maryland. Principal purpose to obtain information on effect of entrained air on mixing water requirements for concrete. To magnify results large quantities of air (about 10 percent) entrained. Strength and "dynamic E" tests of 6 by 12 -in. cylinders also made. Amount of air measured for hardened concrete as well as fresh concrete. Data indicate: (1) Mixing water can be reduced from about 45 percent (lean mixes) to 20 percent (rich mixes) of volume total air entrained; (2) Approximately 30 percent more air in fresh concrete than in hardened concrete; (3) Less reduction in strength due to air than to equivalent volume of mixing water.

Effect of length on the strength of compression test specimens

J. TUCKER, JR.; ASTM 1945 Preprint. Synopsis by AUTHOR

When compression test specimens of concrete are shorter than twice their least width it is customary to adjust the test values by the use of a single set of correction factors, such as those in Standard Methods of Securing, Preparing, and Testing Specimens from Hardened Concrete for Compressive and Flexural Strengths (C 42-44), the corrections depending only upon the ratio of length to least width. This paper demonstrates that the extent of the variation in strength with length is dependent upon several distinct factors. For example, the relation between strength and slenderness of specimen varies with the strength of the concrete and therefore the use of a single set of correction factors is not warranted. Much more reliable estimates of strength may be obtained by limiting the range of slenderness values to those for which the correction is small, even though this may require diameters less than those usually considered acceptable. Although the numerical values which are given apply to concrete, the principles are applicable to other brittle materials.

Equations for computing elastic constants from flexural and torsional resonant frequencies of vibration of prisms and cylinders

GERALD PICKETT; ASTM 1945 Preprint Synopsis by AUTHOR

The equations of Mason, Love, etc., that do not take into account the effects of shear are not so satisfactory as Timoshenko's differential equation for flexural (transverse) vibration of prismatic bars. Goens' solution of Timoshenko's differential equation for bars with free ends is in very close agreement with results obtained by means of the mathematical theory of elasticity if the shear constant K' is given the proper value. The proper value depends on Poisson's ratio, being about $\frac{5}{6}$ for $\mu = 0$, $\frac{8}{9}$ for $\mu = \frac{1}{6}$, and 0.85 for $\mu = \frac{1}{3}$. The radius of gyration r adequately describes the cross-section of a prism for the computation of resonant flexural vibration. The approximate equation given by Goens for the correction factor T_1 can be made accurate to at least three figures by subtracting the quantity $125(r/l)^4$.

Equations are given for computing Young's modulus and the shear modulus from flexural and torsional resonant frequencies, respectively.

Graphs are given from which the size and shape factor C in the equation for Young's modulus and Goens' correction factor T are readily determined.

An equation is given for obtaining Poisson's ratio from the flexural and torsional resonant frequencies, and the limitations of the equation are discussed.

Bridge foundations

W. A. Fairhurst, *Concrete and Constructional Engineering*, V. 40, No. 3, (Mar. 1945) pp. 37–43
Reviewed by Glenn Murphy

This is the concluding installment of a series dealing with design and construction features of bridge foundations. The use of counterweighted abutments is shown in the description of the Dinnet Bridge over the Dee River. The bridge over Grange Burn, Sterlingshire, illustrates the use of deep cellular foundations so proportioned that the structure develops the same load on the subgrade as did the excavated material. This construction is advantageous where the allowable bearing pressure is abnormally low.

Preliminary boring for the foundations of the Kerse bridge showed 5 ft. of mud and stone, 3 ft. of sand and gravel, and at least 83 ft. of soft silty clay. To meet this condition a collar pile was devised. A 3½ ft. square collar, 18 in. thick, was laid in the mud and the lower end of a 35-ft. tapered reinforced concrete pile started through the hole in the collar. Each pile was constructed with a bulge near the top. The pile was driven until the bulge contacted the collar, then both the pile and the collar were driven down. A proof load of 90 tons was placed on one of the piles, resulting in an total settlement of less than ½ in. in 2 weeks. No further deflection occurred when the load was reduced to 75 tons. The design load was 22 tons. The author reports the bridge to be in perfect condition after 12 years service.

Rings in cement kilns, part I: why they form and how to prevent them

Harold R. Gingerich, *Rock Products*, V. 48, No. 6, pp. 89-90, June 1945. Reviewed by R. N. Young

Articles deal with formation of slurry rings and clinker rings in cement kilns, effects of kiln design and operation, methods of removing and preventing. Part I deals with influence of materials and "Laboratory Control" on clinker ring formation. Physical and chemical properties of raw materials are said to influence kiln deposits. Refractory lining is important. Fusion of clinker with kiln lining promotes ring formation. Uneven surface of refractory provides a convenient point for start of ring formation. Viscosity of the molten cement materials is a factor. A mobile liquid with high alkali content will usually develop ring formation readily. High ash coal, with low melting point of the ash promotes rings in area where coal impinges on load, apt to occur when composition of ash varies. Content of *Fe, Al, S* in ash is a leading factor, as is also fineness of coal, coarse ground fuel tending to aid in ring formation. Under laboratory controls, some factors listed as promoting formation of rings are: uneven composition of raw mix, low *Si-Fe, Al* ratio, high *Al* (by agglomeration of clinker), high *Fe*, reducing atmosphere in kiln, coarse raw fineness (except when kiln burning temperatures are high). Range in clinkering temperature depends on change in liquid content of clinker with change in temperature. Range may be wide or narrow. It is thought that ring formation is less apt to occur when the range is wide.

Theory of elastically restrained beams, applied to statically indeterminate reinforced concrete beams

M. Hilal (Lecturer, Faculty of Engineering, Giza). Costa Tsoumas & Co. Press, 1945.
Reviewed by F. E. Richart

In this booklet the author presents solutions of a number of problems in continuous beams and frames, using well known methods of analysis and notation commonly used in Europe. While the title implies that the treatment applies particularly to reinforced concrete, the analysis applies to elastic members of any material, and little attention is given to details of design or analysis peculiar to reinforced concrete.

The method of "fixed points" is used throughout, and analytical and graphical determinations of the position of such points are given for members of constant and varying moments of inertia.

Solutions are given for continuous beams on free supports, including tabulated values for influence lines for moments and shears in beams of 2, 3 and 4 equal spans. Graphical solutions are also demonstrated. Maximum and minimum moments and shears due to live loads, and the effect of settlement of supports are treated.

For certain continuous frames analytical and graphical determinations of the fixed points are shown. Applications are made to beams supported on girders which provide torsional restraint; also to the determination of moments due to temperature and shrinkage and to movements of the supports. For continuous multiple frames, the "equation of four moments" and the equation of deformation angles (essentially the "slope—deflection" equations) are stated.

While it probably contains little that is new to students of the theory of continuous frames, this booklet forms a convenient reference to the "fixed point" method of analysis, particularly to American engineers since it is written in English.

The proportioning of concrete (dosagem de concretos)

F. L. L. Carneiro, Instituto Nacional de Tecnologia, Rio de Janeiro, 1943. *Road Abstracts*, V. XII, No. 5, May 1, 1945. Highway Research Abstracts

The formulas of D. Abrams, J. Bolomey, O. Graf, I. Lyse, and other workers, correlating the compressive strength of concrete with the water-cemet ratio, are critically reviewed, and the method of proportioning is discussed that has been used since 1937, by the National Institute of Technology of Brazil. The formula employed for proportioning relates the water-cement ratio with the total weight of aggregate per unit weight of cement and the weight of water required expressed as a percentage of the total weight of dry materials. The properties and functions of cement, aggregates and water are reviewed in their relation to the quality of concrete, and Brazilian standard specifications for different types of cement and for the sampling, grading and testing of aggregates are quoted in full. Methods are discussed of pre-determining properties of concrete such as compressive and tensile strengths and impermeability and durability, and the relevant clauses are quoted of the Brazilian standards for the strength of structural reinforced concrete and for preparing and curing cylindrical test specimens. Grading and proportioning of aggregates, and workability of concrete, are examined. The grading and proportioning charts used by the National Institute of Technology are reproduced, and the percentages of water required with different sizes of coarse aggregate are tabulated for concrete to be compacted (a) by manual methods, and (b) by vibration. Recommendations are made regarding control and sampling on the site, and permissible tolerances are indicated. An appendix describes briefly a method of determining the tensile strength of test cylinders.

Experiments with admixtures for reducing the mixing water content and improving the workability of concrete

W. Kronsbein, *Zement*, 1943, 32 (19/20), 209-17. *Building Science Abstracts*, V. XVII (New Series) No. 10, Oct. 1944. Highway Research Abstracts

A brief review is made of the action of admixtures to improve the workability and strength of concrete and of published results of tests of various proprietary products. This is followed by an account of experiments with two proprietary admixtures, "Betonplast" and "Murasit" in concrete made with two portland cements and one portland-blastfurnace cement. The reduction of the water content for the same workability of the mix varied, e.g., it was in some cases about 12 per cent, in others less than 8 per cent. An increase of compressive strength of 30 per cent or more, representing a cement economy of about 70 per cent, was obtained in one case, in others the effect in strength was slight. The degree of water-tightness obtained by the admixtures also varied. In general, the experiments showed the effects of the admixtures on the concrete properties to be by no means the same for all concretes. The reduction of the quantity of mixing water made possible differed greatly for different cements and did not always mean an economy of cement. In some cases, the concrete from the mix with a reduced water content had not a denser structure but was more or less porous and there was no increase but even a reduction of strength. Where, in spite of its porous structure, the concrete showed an improvement in water-tightness, this was attributable to the formation of closed pores due to the admixture. The differing results obtained with the admixtures would seem to be largely due to difference in the cement properties; with the few cements used for the experiments the results obtained were very marked. It is to be assumed that the particle structure of the mix, the aggregate shape and grading and the absolute water content also influence the result. In general, the use of admixtures to obtain a definite effect would not seem possible. In every case it would be necessary to carry out preliminary experiments with an admixture in order to determine whether advantages are gained by its use, e.g., a reduction of the mixing water content allowing an economy of cement, an improvement in workability, the prevention of segregation.

Stress conditions for the failure of saturated concrete and rock

K. Terzaghi, ASTM 1945 Preprint — Reviewed by Gerald Pickett

An analysis is made of the stress conditions leading to failure of porous bodies that are under compression. Previous data are examined and conclusions are drawn. Three types of failure are described: (a) splitting, which is regarded as a failure in tension; (b) pseudo-shear, which is regarded as a failure partly in tension and partly in shear; (c) shear. The equation given by Ros for increase in strength with increase in confining pressure is considered to be applicable to materials that fail by splitting. The equation is

$$q_c = q_u + p_c \frac{q_u}{q_t}$$

where q_c is the compressive strength when the confining pressure is p_c

q_u and q_t are the compressive and tensile strengths, respectively, under uniaxial loading.

Terzaghi gives a more complicated equation containing functions of an unknown angle for materials like concrete that fail in pseudo-shear. For such materials, he explains the non-linear increase in strength with increase in confining pressure as being due to a greater tendency to fail in shear when a confining pressure is applied, the greater the confining pressure, the greater the tendency to fail in shear. The equations just mentioned apply when the pores are either empty or, if filled with liquid, are under no hydrostatic pressure. Terzaghi explains that if the liquid penetrates the pores, the confining hydrostatic pressure is not fully effective. He next describes a test in which the pores are completely filled with liquid, the liquid being under a hydrostatic pressure equal to the confining pressure. He mentions previously conducted tests of this kind on concrete in which there was very little increase in strength with increase in confining pressure. From these results Terzaghi reasons that the individual constituents of saturated concrete are almost entirely surrounded by liquid. He further reasons that the voids must therefore consist of narrow slits.

Although it appears that Terzaghi is rather firmly convinced of the correctness of his analysis, he mentions that the basis for his analysis is semi-empirical. Hence, final answers must await the results of more conclusive tests. He describes the essential conditions for such tests.

Evaluating the mechanical strength of coarse aggregates

A. H. D. Markwick and F. A. Shergold. (Abstract of paper No. 5447, *Journal of the Institution of Civil Engineers*, Apr. 1945). *The Surveyor and Municipal and County Engineer*, V. CIV, No. 2782, May 18, 1945, London. Highway Research Abstracts

This paper relates to a test developed at the Road Research Laboratory to meet the need for a single test that can be quickly and simply carried out on material as crushed and supplied for use in construction, in which condition it is termed "aggregate".

In the standard aggregate crushing test a measured quantity of aggregate (about 3 kilogrammes) that has been sifted between ½-in. and ⅜-in. British Standard sieves and oven-dried for 24 hrs. at 100-110 deg. C. (212-230 deg. D.) is placed in a 6-in. diameter cylindrical steel mould with a close-fitting plunger, and subjected to a load increasing uniformly to 40 tons, in 10 min. The aggregate crushing value is determined by the percentage of fines passing a No. 7 British Standard sieve after test, and is an inverse measure of the mechanical strength of the aggregate. Values range from 5.3 for an exceptionally hard hornfels to 72.8 for foamed slag or soft brick. The values for hard igneous rocks, quartzites, flints and sandstones range from 10 to 20, for limestones from 20 to 30, and for slags from 25 upwards. In developing the test, factors affecting the results, such as size of sample, total load, rate of application, drying conditions and method of compaction were standardized. Particle shape was not standardized and the authors state that flaky and elongated particles gave somewhat poor test values.

To obtain geometrical similarity, when aggregates of various sizes are tested, the ratio of the aggregate size to the aperture size of the sieve used to separate the fines should, the authors explain, be kept constant, and the size of the test cylinder should also, in theory, be varied proportionately to the aggregate size, though in practice it has been found that three sizes of cylinder suffice to cover the usual range of aggregate sizes. Details of the conditions for testing various sizes of aggregates are shown in the following table:

RECOMMENDED TEST CONDITIONS FOR AGGREGATE CRUSHING TESTS ON VARIOUS SIZES OF AGGREGATES

	*Size of aggregate to be tested (B.S. sieves: inches)								
Passing Retained on	2 1½	1½ 1	1 ¾	¾ ½	½ ⅜	⅜ ¼	¼ 3/16	3/16 ⅛	⅛ No 7
Size of test sieve for separating fines: inches	⅜	¼	3/16	⅛	No. 7	No. 10	No. 14	No. 18	No. 25
Diameter of test cylinder: inches	12	12	12	6	6	6	3	3	3
Load applied: tons	160	160	160	40	40	40	10	10	10
Relative volume of test sample	8	8	8	1	1	1	⅛	⅛	⅛

*The larger size of sieve defines the nominal size of the aggregate.

The effect of size on crushing strength was illustrated by the result of a test on cylindrical specimens of a limestone the strength falling from 29,000 psi. for 1-in. (diameter and height) specimens to 20,000 psi. for 2-in. specimens.

As regards correlation with other tests, on the same materials, the crushing values of a large number of ½-in. aggregates showed: (1) Almost numerical agreement with the Los Angeles values; (2) fair correlation with crushing strength and impact values; (3) poor correlation with dry, and wet attrition values; (4) little correlation with water absorption and abrasion values.

Disintegration of bridge concrete in the west

F. H. Jackson, *Public Roads*, V. 24, No. 4 — Reviewed by the Author

This report gives the results of an inspection of approximately 200 concrete structures in areas of Wyoming, Oregon, Washington and California, in connection with a study of the causes of premature failure of concrete in certain portions of these states. The report also includes discussion of possible reasons for the failures and suggestions for revisions in specifications to govern future work.

Many of the older projects, particularly those built prior to 1930, were in surprisingly good condition. However, the concrete in many of the bridges, including a rather disturbingly large proportion of the newer structures, showed sufficient evidence of distress to warrant considerable concern.

During the course of these inspections, four distinct types of concrete deterioration were observed. These were classified in the report as follows:

Type 1.—Deterioration due to gradual or normal weathering. This is usually indicated by slight surface erosion, pitting, rounded corners, etc. Many old structures also show cracks due to settlement and impact from colliding vehicles. This condition is to be expected and is evidence of durable concrete, especially when found on older structures subject to severe weathering.

Type 2.—Deterioration due to accelerated weathering. This type of failure is unfortunately rather common, particularly in areas subjected to severe frost action. It is usually evidenced first by the formation of fine cracks or "D-lines" on the surfaces of exposed members. Concrete so affected has little strength and the matrix has a dull, chalky appearance in sharp contrast to the dense, compact bluish-gray matrix usually found in good concrete. Rapid and progressive disintegration of the concrete frequently follows the appearance of "D-lines."

Type 3.—Deterioration due to salt scaling. This type is the result of using calcium or sodium chloride for ice removal and will almost invariably occur on surfaces so treated, particularly if the concrete is new.

Type 4.—Cracking due to abnormal expansion. This type of failure is due to internal expansion of the concrete resulting in a formation of relatively wide, open cracks of appreciable depth in the surface of the member. The principal example of this type of failure is the expansion due to reaction between the alkalies in portland cement and the siliceous constituents of certain aggregates.

Evidence of type 2 deterioration was found in many structures in those areas of Wyoming, Oregon and Washington which are subject to severe weathering. In some cases there was evidence that weathering of the type 2 variety had been preceded by internal expansion (type 4), with final failure as the result of a combination of the two. However, in many places there was no evidence that alkali reaction had been a factor in the disintegration of the concrete; failure in these instances being clearly the result of accelerated natural weathering.

The report discusses the possibility of using air entrainment to increase the durability of concrete, as well as certain restrictions on the alkali content of portland cement and the necessity for examination of the alkali-reactive characteristics of aggregates in areas which seem to be subject to this action. The report also discusses the apparently superior performance of old-fashioned cements as compared to the portland cement now being manufactured. It concludes with the following recommendations:

That, where concrete is to be exposed to severe frost action, specifications be revised, where necessary, to require, in addition to a suitable cement content, that the free water content of the mix shall, in no case, exceed 6.0 gallons per sack of cement.

That, where concrete structures or portions of structures (including pavements) will be exposed to severe frost action, provision be made to entrain sufficient air in the fresh concrete so as to produce a total air content of from 3 to 5 percent, based on the theoretical weight of the air-free concrete.

That the desired air entrainment be obtained preferably by adding an approved air-entraining agent to the concrete at the time of mixing, in such quantity as will maintain the percentage of air within the limits specified.

That approval of any material proposed for use as an air-entraining agent be based on data, obtained from either research or field use, or both, that are sufficiently comprehensive to demonstrate to the satisfaction of the contracting agency that the proposed material, when used as required, will not seriously affect the strength or other essential properties of the concrete.

That, when the desired air entrainment is to be obtained by the use of an air-entraining cement: (1) The cement meet the requirements for air entrainment given in A.S.T.M. Specification C 175-44 T; and (2) the specifications authorize the engineer to require such changes in materials, proportions, or methods of mixing as may be necessary from time to time to maintain the percentage of air within the limits specified.

That, in all areas where tests or previous experience indicate that alkali-reactive aggregates may be encountered, specifications for portland cement be modified by requiring that the total percentage of sodium oxide plus 0.658 times the percentage of potassium oxide shall not exceed 0.60.

Tests on square twisted steel bars and their application as reinforcement of concrete

K. Hajnal-Konyi. Reprint from *The Structural Engineer*, V. XXI, No. 9. (Sept. 1943) pp. 327-368; Discussion Vol. XXII, No. 2 and 3 (Feb., Mar. 1944) pp. 66-92, 114-118. Reviewed by Author

The tests comprised 35 beams reinforced with small size square bars (5g and ½ in.) both twisted and untwisted. All beams were 6 in. wide and 7 ft.0 in. long, the depths varying between 4 and 10 in. Some were made of ordinary grade concrete (1:2:4 mix), some in high grade concrete (1:1:2 mix) and the amount of reinforcement varied between .059 percent and 1.785 percent for untwisted steel, and between .214 percent and 1.19 percent for twisted steel. The beams were tested on a span of 6 ft. 0 in. with two knife-edge loads at the third points. All beams had sufficient shear reinforcement to exclude failure by diagonal tension.

In the majority of the beams reinforced with twisted steel, up to .9 percent failure occurred by fracture of the reinforcement. This did not happen with mild steel even at $p = .214$ percent and, for this reason, two beams were made with .059 and .137 percent respectively, in both of which the mild steel bars could be fractured. In both beams the maximum load was reached at the first crack after which the load suddenly dropped and then increased again to a second maximum at which fracture of the reinforcement occurred. By this stage a very great deflection had occurred. The second maximum was lower than the first.

In all beams reinforced with twisted bars the maximum bending moment was substantially in excess of the value calculated from the ultimate strength of the reinforcement on the basis of the plastic theory. On the basis of the standard method the excess was even greater in most of the beams. To explore this phenomenon special tensile tests were arranged. 5g bars both twisted and untwisted were fractured in concrete specimens of 3x3 in. at the critical section. Three tests with twisted bars agreed very well with each other, their average ultimate strength was 16 percent in excess of the ultimate strength of specimens cut from the same rods, when tested in the air. The untwisted bars showed no excess since the bond was destroyed over the full embedded length of the bars long before failure.

It appears that steel embedded in concrete shows an ultimate strength higher than that obtained in the usual test if the bond in the neighborhood of the crack is maintained on each side until failure. This phenomenon is similar to the well known notch effect and does not occur with steel having a definite yield point. It can be achieved with work hardened steel which does not yield.

Nothing in the behavior of the beams suggested that the conventional "yield" point of twisted steel (which is the stress corresponding to an elongation of .5 percent) was in any way significant. The tests have shown that the ultimate strength of twisted bars is a safe basis for calculating the ultimate bending moments.

In all tests the development of cracks was carefully observed. According to continental tests, a maximum width of .01 in. can be considered harmless from the point of view of corrosion. In no beam was this limit reached at a calculated stress lower than 40,000 psi.

The article points out deficiencies in the standard method of design and demonstrates that a far better agreement with test results can be obtained by the plastic theory (Whitney's method). It deals particularly with the reduction of the design factor M/bd^2 at the so called "economic" percentage, if the permissible steel stress is increased. This is a consequence of the standard method and handicaps the economic use of high tensile steel. The tests have proved that square twisted steel can safely be substituted for mild steel in the inverse ratio of its ultimate strength to the yield point of mild steel without increasing the cross section of the concrete. If the permissible stress for square twisted steel is fixed at 50 percent above that of mild steel, the factor of safety of beams reinforced with square twisted steel is higher than if mild steel of 1.5-timed the sectional area of the twisted steel had been used.

The permissible steel stress is limited by the width of cracks. On the assumption of a permissible width of .01 in. a permissible stress of 30,000 psi. would be safe for square twisted steel having an ultimate stress of not less than 75,000 psi. This applies to slabs and rectangular beams up to .9 percent in 1:2:4 concrete (min. cube strength 2850 psi.) and up to 1.2 percent in 1:1:2 concrete (min. cube strength 3750 psi.), reinforced with square twisted bars of a size not exceeding ½ in.

The article also deals with the loss of the high tensile properties of cold worked steel by the influence of heat. Tests have proved that permanent loss of high tensile properties does not occur if the temperature of the steel has not exceeded 400 C and is insignificant even if the steel has been heated to 500 C. On the other hand, once the temperature of mild steel has exceeded 500 C, the building is likely to collapse or at least to be beyond repair.

In the discussion the author, referring to an article by A. C. Vivian*, drew attention to the gross errors which were introduced in the "nominal" stress-strain diagram of a metal which underwent much change in area before fracture. Work hardening affects the volume of metal involved in the fracture, since the reduction of the sectional area

*Squadron-Leader A. C. Vivian: Mechanical Properties of Metals. *Engineering*. July 23 and Aug. 8, 1943, pp. 78-81 and 118-120. Paper entitled "A Renaissance of Mechanical Properties" presented to the Institution of Mechanical Engineers.

was considerably less than in the case of mild steel.* When the stress of both is mentioned it is to the nominal area and not to the actual area at fracture that is referred to. It may also explain the further increase in the ultimate strength if a bar is fractured while firmly gripped in concrete and thus the loss of the sectional area at fracture is further reduced.

Structural efficiency of transverse weakened-plane joints

EARL C. SUTHERLAND AND HARRY D. CASHELL, *Public Roads*, April-May-June 1945, V. 24, No. 4
Reviewed by the AUTHORS

Early in 1940 the Public Roads Administration sponsored a comprehensive investigation, in cooperation with several State Highway Departments, for the primary purpose of studying the effects of varying the spacing of expansion joints in pavements with closely spaced weakened-plane contraction joints. As a part of this investigation the Public Roads Administration carried out a program of special tests to determine the ability of joints of the weakened-plane type to reduce the critical stresses caused by loads acting near the joints.

In this study information was obtained on the structural behavior of weakened-plane joints as affected by: (1) type of coarse aggregate, (2) maximum size of coarse aggregate, (3) presence or absence of dowel bars, (4) method of producing fracture at the joint, (5) compressive forces acting to close the joint, and (6) width of joint opening.

The pavement on which these tests were made was divided into 6 sections, each 30 ft. long, 20 ft. wide and of 8-in. uniform thickness. Each section was definitely separated from those adjoining it and was divided longitudinally by a deformed metal center joint having ⅝-in. diameter tie bars at 60-in. intervals and divided transversely by a weakened-plane joint. Provision was made for opening and closing the joints, by mechanical means, so that they could be tested under compression and at various controlled openings.

To investigate the influence of the type and size of coarse aggregate, on the structural efficiency of weakened-plane joints, different concrete mixtures were used in the various sections. Potomac river gravel was used as the coarse aggregate in 4 sections, the maximum size being 1 in. in 2 sections and 2½ in., in the other two. The coarse aggregate in the two remaining sections was a crushed limestone obtained from near Frederick, Md. The maximum size of the aggregate was 1 in., in one of the limestone sections and 2½ in. in the other.

Load transfer devices were installed in one of the joints for comparing the relative efficiency of weakened-plane joints with and without load transfer devices. These consisted of ¾- by 24-in. plain round dowels spaced at intervals of 12 in.

The testing schedule was essentially the same for each of the joints. Briefly, this schedule consisted of:

1. Measurement of the critical strains at the interiors of the four panels of a given section for a load of selected magnitude.

2. With the joints under compression, caused by a horizontal force of 120,000 lb. acting longitudinally at one end of the section, the measurement of the critical strains caused by loads acting at the joint edges and corners.

3. Release of the compressive force and measurement of the critical strains caused by loads acting at the joint edges and corners for successive controlled joint openings of 0.037, 0.055, 0.073 and 0.092 inch.

*This may explain why the ultimate strength of work hardened steel is higher than that of mild steel.

4. Opening of the joint to a width of ½ in. and measurement of the critical strains caused by loads acting at the joint edges and corners. For this opening all effects of aggregate interlocking had disappeared and the slab edges at the joint were acting as free ends. In the case of the doweled joint all dowels were cut before testing at this opening.

5. Remeasurement of the critical strains caused by loads acting at the interior of the panels.

From the data obtained in these tests it was possible to compute the efficiency of the various joints in reducing the critical stresses caused by loads acting at the joint edges and corners. It was found that aggregate interlock was effective in stress control, both at the joint edges and joint corners, when the joints were under compression. At the 0.037-in. opening aggregate interlock was very effective in controlling the critical corner stresses, but it was found to be an uncertain means of stress control at the joint edges with this opening. At the 0.073-in. opening aggregate interlock was ineffective in stress control both at the joint edges and corners. The dowels definitely increased the effectiveness of the joint in reducing the critical corner stresses and improved the uniformity of stress reduction between the different corners and from point to point along the joint edge. However, the doweled weakened-plane joint was not highly effective in reducing the critical edge stresses probably because the diameter of the dowels was not sufficient to be effective in slabs of 8 in. or more in thickness.

The weakened-plane joints in the slabs with gravel aggregates were more effective than those in slabs with crushed stone aggregates. Also large gravel aggregate gave more effective stress control than small gravel aggregate, but the size of the aggregate had little influence on the effectiveness of the joints in the slabs with crushed-stone aggregate.

To determine the cause of this difference in the effectiveness of the joints with the different types of coarse aggregates, a portion of one of the panels forming the joint was removed from each section so that the faces of the joints could be examined. The examination showed that in the sections having gravel coarse aggregate the joint faces were rougher than those of sections containing crushed-stone coarse aggregate due to the fact that when the crack occurred at the joints, the crushed-stone aggregate fractured whereas the gravel aggregate, in most instances, pulled out through loss of bond. It should be recognized that different aggregates might act differently in this respect. That is, some gravel aggregates might fracture at the joints, giving a smooth face, while some crushed-stone aggregates might pull out.

It is concluded from this investigation that aggregate interlock is not a dependable method of stress control at weakened-plane joints. However, it has been found from field observations that aggregate interlock is helpful in preventing or reducing the amount of faulting at weakened-plane joints and cracks when they are not allowed to open too widely. In pavements laid with expansion joints there is a tendency for the contraction joints and cracks to open progressively until all available expansion space is dissipated and if the available expansion space is sufficient the joints and cracks may open so widely that the effectiveness of aggregate interlock, in this respect, will be destroyed. Thus, if the effectiveness of aggregate interlock for preventing or reducing faulting at weakened-plane joints and cracks is to be maintained it appears to be necessary to limit the amount of available expansion space in the pavement. This may be done either by actually eliminating expansion joints or by using an expansion joint filler that offers considerable resistance to compression.

ACI NEWS LETTER

Vol. 17 No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE September 1945

REPORT OF 1945 NOMINATING COMMITTEE

The 1945 ACI Nominating Committee, Roderick B. Young, chairman, Roy W. Crum, Raymond E. Davis, Frank H. Jackson, J. C. Pearson, T. C. Powers, T. E. Stanton, Morton O. Withey report the following nominations for offices whose terms expire at the 1946 convention:

President

Harrison F. Gonnerman, Manager Research Laboratory, Portland Cement Association, Chicago, an ACI Member since 1918; Wason Research Medalist, Member Board of Direction as Director Fifth District since February 1943; elected Vice-President February 1945. A further record of his Institute activity will be found under his name in the ACI Directory. He is a member of the executive group of ACI Committee 115, Research; a member of the Publications Committee and in papers and otherwise has contributed much to the Institute's technical work.

Vice-President (to succeed himself)

Stanton Walker, Engineering Director, National Sand & Gravel Association and of the National Ready Mixed Concrete Association, Washington, D. C.; an ACI Member since 1921; member of Standards Committee since 1937, and of the Advisory Committee, as Chairman Department 200, General Properties, since 1938 and became Chairman of the Advisory Committee, February 1945. Member Board of Direction since 1940 when he became Director Fourth District by appointment to fill a vacancy and twice elected; elected Director-at-Large for a three-year term, 1943 and Vice-President, 1945.

Vice-President

Robert F. Blanks, Chief, Engineering and Geological Control and Research, Bureau of Reclamation, Denver, Colo.; ACI Member since 1932; member Publications Committee since 1941; Chairman since February 1945; Member Board of Direction as Director Sixth District, 1941-42; appointed by Board March 1944 to be Director-at-Large to fill a vacancy and re-elected February 1945; Chairman Committee 613, which reported "Recommended Practice for the Design of Concrete Mixes," now become a Standard of the Institute. Has written and participated in the writing of many technical contributions to ACI work.

Director-at-Large (to succeed himself)

Henry L. Kennedy, Manager Cement Section, Dewey & Almy Chemical Co., Cambridge, Mass. ACI Member since 1934; elected to Board of Direction 1944 and re-elected 1945, and at once appointed to fill a vacancy as Director-at-Large caused by the election of Stanton Walker to a vice-presidency.

Regional Director, First District (to succeed himself)

Paul W. Norton, Consulting Engineer, Boston, ACI Member since 1931; appointed First-District Director to fill vacancy caused by Mr. Kennedy's appointment as Director-at-Large.

Regional Director, Second District

Roy R. Zipprodt, Research and Consulting Engineer, Committee on Reinforced Concrete Research, American Iron & Steel Institute, ACI Member since 1920 and Secretary ACI Committee 318, Standard Building Code.

Regional Director, Third District (to succeed himself)

Alexander Foster, Jr., Vice-President, Warner Co., Philadelphia; ACI Member since 1912; elected to the Board as Director Third District, 1945.

Regional Director, Fourth District (to succeed himself)

Frank H. Jackson, Senior Engineer of Tests, Public Roads Administration, Washington, D. C.; ACI Member since 1924; a member Advisory Committee as Chairman Department 900, Joint Efforts, since 1929; member Program Committee 1938-40; member Publications Committee since 1941; member Board of Direction as Director Fourth District, 1937-38 and elected to the same position, 1945.

Regional Director, Fifth District (to succeed himself)

Charles S. Whitney, Consulting Structural Engineer, Milwaukee, Wis., ACI Member since 1920; Wason Medalist for Most Meritorious Paper; elected to the Board as Director Fifth District 1945.

Regional Director, Sixth District (to succeed himself)

Herbert J. Gilkey, Head Department Theoretical and Applied Mechanics, Iowa State College, Ames, Ia.; ACI Member since 1924; Wason Medalist for Most Meritorious Paper; Member Standards Committee and chairman since March 1944; member Publications Committee since 1939; member Board of Direction, as Director Sixth District, 1937-38 and elected to the same position 1945.

Nominating Committee

The Nominating Committee also presented 20 candidates from among whom five are to be chosen to serve with the three latest past-presidents as the 1946 Nominating Committee—the candidate receiving the most votes on the letter ballot to be Chairman. The 20 candidates:

Arthur J. Boase	F. V. Reagel
S. J. Chamberlin	Frank R. Richart
Miles N. Clair	J. R. Shank
P. J. Freeman	B. W. Steele
Fred Hubbard	E. O. Sweetser
J. W. Kelly	Louis H. Tuthill
William F. Kellerman	I. L. Tyler
George A. Maney	C. A. Willson
H. S. Meissner	Benjamin Wilk
John R. Nichols	Roy N. Young

Who's Who in this ACI Journal

Clifford A. Betts

is engineer U. S. Forest Service, and not a member of this Institute, but he writes interestingly of forest service concrete work (p. 1). He is a Ph. B. of the Sheffield Scientific School of Yale University and was a scholarship student in hydraulics at the University of Wisconsin.

As a student he did surveying and city map work for South Norwalk, Conn. where he was later inspector on sewer construction, then draftsman on waterworks and sewage disposal plans with James H. Fuertes, New York; in charge of building construction in South Norwalk; he did research work in the hydraulic laboratory, University of Wisconsin, 1914; was engineer for the City of Norwalk, Conn. in 1915; assistant city engineer in Norwalk in 1916; assistant engineer, Bridgeport, Conn. 1917; did design work and drafting for the Commissioner of Public Docks, Portland, Oregon; then was chief engineer for the Cummings-Moberly Lumber Co. cooperating with the U. S. Government in aircraft spruce production, 1918; with United States Forest Service, Denver, Denver Municipal Water works, 1919 to '23. He was office engineer, Moffatt Tunnel Commission 1923 to '28; engineer, Bureau of Reclamation on Owyhee Dam, Oregon and on 3½ and 4½ mile long tunnels 1928 to '34. On the technical staff of the Mississippi Valley Commission and National Resources Commission for comprehensive planning for utilization of water resources, 1934; engineer, U. S. Forest Service, Washington, D. C. 1935 to date. He has written papers for the engineering press and won the Thomas Fitch Roland prize 1932, American Society Civil Engineers.

Ralph W. Kluge and Edward C. Tuma

collaborated in the paper on lapped-bar splices (p. 13).

Mr. Kluge, an ACI Member since 1938 is not only not new to these pages, but is a Wason medalist by reason of his work with Wilbur M. Wilson in "Tests of Rigid Frame Bridges." He left the University of Illinois where he was Special Assistant Professor of Theoretical and Applied Mechanics, in 1941 to go to the Bureau of Standards, as structural engineer in the Masonry Construction Section.

Mr. Tuma was graduated from the Nebraska State Teachers College with the degree of B. S. in 1930. After graduation, he was employed for short periods with the Rocky Mountain Drilling Co. and the Kansas City Bridge Construction Co.; taught mathematics at the Big Horn County High School in Lovel, and later at the Hot Springs County High School at Thermopolis, Gotham, Wyoming. In January 1943, he was appointed Jr. Materials Engineer in the Masonry Construction Section of the National Bureau of Standards.

Culbertson W. Ross

(see p. 37) received his B. S. degree from Alma College in 1926 and in 1931 his M.S. from the University of Michigan. He has been at the National Bureau of Standards since 1927, where he was first employed in the Cement Section, and since 1930 in the Masonry Construction Section where he has been engaged in the study of the deflections of structures with temperature changes; in connection with the Arlington Memorial Bridge, and the Reinforced Concrete Arches of the Navy Model Testing Basin at Carderock, Md.

Charles E. Wuerpel

who reports extensive field use of cement containing Vinsol resin (p. 49) is not new to these pages. He is Principal Engineer, Central Concrete Laboratory, North Atlantic Division Corps of Engineers U. S. Army and has been an ACI Member since 1937.

New Members

The Board of Direction approved 78 applications for Membership (66 Individual, 6 Corporation, 3 Junior, 3 Student) received in May, June and July as follows:

J. Francisco Aguilar V., 1845 Franklin St., San Francisco 9, Calif.

Felix Garcia Alonso, 22 Nro. 154 Vedado, Havana, Cuba

Edwin C. Anderson, 986 Dexter Horton Bldg., Seattle 4, Wash.

Jose Pernz Benitoa, Ave 5a No. 8 Rpto, Miramar, Havana, Cuba

Emanuel Ben-Zvi, Kiryath Shmuel, House Gruber, Jerusalem (Palestine)

Ross D. Billings, United States Bureau of Reclamation, Anderson Dam, Idaho

Donald Blair, 173 Daly Ave., Ottawa, Ontario, Canada

H. C. Boyd, c/o Virginia Steel Co., P. O. Box 595, Richmond, Va.

Servando Pita Camacho, Patrocinio 59 Vibora, Havana, Cuba

Nicanor del Campo, 14 y 19 Rpto, Almendares, Marianao, Havana, Cuba

James T. Caprio, 1221 N. Fremont Ave., Tucson, Ariz.

Fco. Martin y Ruiz del Castillo, Bentre 6 y F. Rpti Beniteg, Marianas, Havana, Cuba

Ceco Steel Products Corp., 5701 West 26th St., Cicero 50, Ill. (H. D. Jolley)

Cement & Concrete Assn., 52 Grosvenor Gardens, London, S. W. 1, England

Thomas F. Chace, 268 Market Street, San Francisco 11, Calif.

R. S. Chew, 936 Mills Bldg., San Francisco, Calif.

Walter L. Couse, 12740 Lyndon Ave., Detroit 27, Mich.

Walter A. Crossman, 216 South Lee, Altus, Okla.

G. L. Cummings, c/o C. D. Howe Co. Ltd., Port Arthur, Ontario, Canada

Stuart B. Dickens, 13 Kingsway Road, Leicester, England

Gamil Boulos Fanous, 4 Harass St., Garden City, Cairo, Egypt

Martin E. Flaherty, 9205 Whitney Ave., Elmhurst, N. Y.

W. H. G. Flay, 386 Sunnyside Ave., Ottawa, Canada

Herman Frauenfelder, P. O. Box 606, 166 Chapel St., New Haven 3, Conn.

G. W. Froggatt, Borough Engineers Office, Town Hall St., Blackpool, England

R. H. Gagle, Montana Highway Dept., Helena, Mont.

Alex Graham Garvock, c/o A. I. Garvock Ltd., Regent Theatre Bldg., Ottawa, Ontario

Ray C. Giddings, 4669 East 49th St., Los Angeles, Calif.

Great Lakes Concrete Pipe Co., Inc., 9 Austin St., Buffalo 7, N. Y. (M. C. Kelly)

Robert H. Griffin, 5909 Contra Costa Rd., Oakland 11, Calif.

Joseph R. Guptill, 5936 McAndrew Drive, Oakland 11, Calif.

Ronald W. Hadley, Ceco Steel Products Corp., Foot of New York Ave., Jersey City, N. J.

Lester T. Hagadorn, P. O. Box 518, Salina, Kansas

Leo Hirsh, c/o Pacific Concrete Products Co., 11330 Tuxford, Roscoe, Calif.

Lyman G. Horton, 191 S. El Molino Ave., Pasadena 5, Calif.

Joyce & McGregor Ltd., P. O. Box 1424, Cape Town, South Africa

Jose Lecuona, Aguiar No. 361 Dpto. 202, Havana Cuba

William Lerch, Portland Cement Assn., 33 W. Grand Ave., Chicago 10, Ill.

Walter Lohrey, 788 Central Ave., Scarsdale, N. Y.

W. T. MacDonald, 16 Gould St., Toronto, Ontario

Thomas C. McPoyle, Birdsboro, Pa.

L. E. McQuillin, Dewey Portland Cement Co., Dewey, Okla.

H. L. Mahaffy, 355 St. James St., Montreal, Canada

S. I. Mahbub, K. B., c/o The Registrar Irrigation Secretariat, Lahore, India

Cesar E. Maso, San Lazaro 682, Havana, Cuba

H. B. Miller, c/o Christiani & Nielsen, P. O. Box 2827, Cape Town, S. Africa

Sylvester Morabito, Hq. Det. 2233 Engr. Const. Ser. Section APO 244, c/o P.M. San Francisco, Calif.

F. Hunter Neison, 1423 Dufossat St., New Orleans 15, La.

Charles I. Orr, 222 Noble Ave., Akron 2, Ohio

Pan American Commerce, Inc., 205 W. Wacker Dr., Chicago, 6, Ill. (Thomas Mabry)

Mario Romanach Paniagua, 19 y 6 Apt F Vedado, Havana, Cuba

Adrian del Paso Jr., Iturrigaray 160, Mexico, D. F.

Frederick Payne, Portland House, Tothill St., London, S. W. 1, England

Frederick James Pearson, 12 Torkington Road, Gatley, Cheshire, England

Frank W. Pelsue, 207½ N. El Molino St., Alhambra, Calif.

Gerald Pickett, 33 W. Grand Ave., Chicago 10, Ill.

Francisco A. Pividal, la No. 105 Vedado, Havana, Cuba

Ben Poisner, 518 Dalaware, Kansas City, Mo.

James J. Pollard, Dept. of Architecture, Georgia School of Technology, Atlanta, Ga.

William M. Robb, Abbey House, Castlegate, Grantham Lincs, England

Simeon Ross, 6544 Saunders Street, Forest Hills, L. I., N. Y.

Tilton E. Shelburne, Va. Dept. of Highways, Richmond 19, Va.

Antonio Quintana Simonetti, Cerro 1852, Havana, Cuba

J. N. Sparling, 2630 Chester Ave., Cleveland 14, Ohio

Frederick E. Springate, 3333 Ridgewood Drive, Parma 9, Ohio

Hugh William Stephenson, CCM, CBMU No. 620, C/O F. P. O., San Francisco, Calif.

L. J. Sullivan, Box 6, San Pedro, Calif.

Frank Tanaka, 602 Greenlawn, Fort Wayne 7, Ind.

Alderman Library, University of Virginia, Charlottesville, Va.

Manuel Febles Valdes, 9 No. 659 Vedado, Havana, Cuba

Manuel Angel Gonazlez del Valle, Oficios No. 104, Apt. 304, Havana, Cuba

Albert Weiner, 1133 Mile Square Road, Yonkers 4, N. Y.

J. S. Whitney, Ens. USNR, USS McNair, (DD 679), c/o F. P. O., San Francisco, Calif.

J. Williamson, Office of the Dist. Roads Engr., Provincial Roads Dept., Ceres, Cape Province, Union of S. Africa

George Winter, Lincoln Hall, Cornell University, Ithaca, New York

Julian H. Wulbern, c/o H. E. Beister Corp., 3-135 General Motors Bldg., Detroit 2, Mich.

J. W. Wynn, c/o Engstrom & Wynn, 1117 Chapline St., Wheeling, W. Va.

Chia Shiang Yen, 1105 W. Clark St., Urbana, Ill.

New ACI Charter

ACI has been re-incorporated with a new statement of objects. The Institute's original charter, granted to its predecessor organization, the National Association of Cement Users, by articles of incorporation, recorded in the District of Columbia, Dec. 20, 1906, (first convention 1905) was amended for change of name to American Concrete Institute July 2, 1913. Steps to a new charter, better to set forth the developed purposes of the organization, were completed with re-incorporation Aug. 8, 1945.

A certified copy of an excerpt from minutes, Board of Direction, Feb. 16, 1945 is the basis of and embodies the new articles. From the minutes:

11) WHEREAS, a referendum ballot by Members of the American Concrete Institute on charter revision showed 576 for and 2 votes against the proposition with 107 members not voting on this proposition,

AND WHEREAS, the proposed revision of the Articles of Incorporation of the National Association of Cement Users, under the Incorporation Laws of the District of Columbia, recorded December 20, 1906 and as amended with change of name to American Concrete Institute July 2, 1913, was further revised by the affirmative vote of more than two-thirds of the members of the Board of Direction, February 29, 1944,

THEREFORE, Past Presidents Roy W. Crum, P. H. Bates and Ben Moreell, all of whom are citizens of the United States and Mr. Bates and Vice Admiral Moreell, who are residents of the District of Columbia, are now authorized to act for the American Concrete Institute in filing the approved Certificate of Reincorporation as follows:

Know all men by these presents:

That we the undersigned, of whom all are citizens of the United States and a majority are citizens of the District of Columbia, have been duly authorized by affirmative vote of more than two thirds of the members of the Board of Direction of the American Concrete Institute, a corporation existing under the laws of the District of Columbia, to file this Certificate of Re-incorporation, under Section 29-604 of Chapter VI, of Title 29 of the Code of the District of Columbia (1940).

1. The name of the corporation shall continue to be the American Concrete Institute.
2. The existence of said corporation shall be perpetual.
3. The objects of the said corporation shall be to organize the efforts of its members for a non-profit public service in gathering, correlating and dissemi-

nating information for the improvement of the design, construction, manufacture, use and maintenance of concrete products and structures. To this field of engineering education its resources shall forever be devoted.

4. **The management of the corporation for the ensuing year shall be vested in a Board of Direction, consisting of eighteen members.**

I, the undersigned, do hereby certify that I am the duly appointed Secretary-Treasurer of the American Concrete Institute, and that the foregoing is a true and correct excerpt from the minutes of a meeting of the Board of Direction of the said corporation held on the 16th of February 1945, at which meeting a quorum was present and that more than two-thirds of the eighteen members of the Board voted in favor of the resolution.

IN WITNESS WHEREOF, I have hereunto set my hand and have caused the seal of the corporation to be affixed this 17th day of July, 1945 AD.

HARVEY WHIPPLE, Secretary-Treasurer

Signed by Past Presidents—Ben Moreell, P. H. Bates and R. W. Crum, July 26, 1945, the new Articles of Incorporation were recorded, District of Columbia, Aug. 8, 1945.

Honor Roll

February 1 to August 31, 1945

Rene Pulido y Morales, in Havana, Cuba heads the list with 16 new Members proposed since Feb. 1.

Rene Pulido y Morales	**16**
Roy Zipprodt	**5**
Harry B. Dickens	**4**
H. F. Gonnerman	**4**
A. Amirikian	**3**
J. H. Spilkin	**3**
Charles S. Whitney	**3**
Ernst Gruenwald	**2½**
Charles E. Wuerpel	**2½**
C. Blaschitz	**2**
Francis MacLeay	**2**
D. E. Parsons	**2**
J. M. Wells	**2**
J. W. Kelly	1½
W. Fisher Cassie	1
A. R. Collins	1
H. W. Cormack	1
R. F. Dierking	1
G. H. Hodgson	1
V. P. Jensen	1
L. I. Johnstone	1
William G. McFarland	1
Denis Matthews	1
Calvin C. Oleson	1
A. F. Penny	1
Kenneth Powers	1
John A. Ruhling	1
C. H. Scholer	1
Byram Steel	1
G. W. Stokes	1
Wm. Summers, Jr.	1
M. A. Timlin	1
J. W. Tinkler	1
Maxwell Upson	1
Stanton Walker	1

The following credits are, in each instance, "50-50" with another Member:

Birger Arneberg	R. E. McLaughlin
E. E. Bauer	Adolph Meyer
E. W. Bauman	M. D. Olver
R. R. Coghlan	F. E. Richart
R. B. Crepps	Kanwar Sain
Harmer E. Davis	J. L. Savage
W. C. Hanna	Oskar Schreier
Carl W. Hunt	A. L. Strong
Paul A. Jones	A. R. Waters
H. J. McGillivray	K. B. Woods

Lloyd P. Lumpkin

a member of the Institute from 1937; died at his home in Little Rock, Ark., Jan. 11, 1945—word of the death having only recently came to the Institute. He was a native of Dixon, Missouri; was graduated from the University of Missouri and became engineer of materials and tests for the Arkansas State Highway Department; later technical engineer for the Marquette Cement Mfg., Co. Chicago; also associated with Ben M.

Hogan Co. general contractors, Little Rock.

Frederick H. Richardson

who had been a member of the Institute since 1932 died November, 1944 according to word reaching the Institute only recently. Mr. Richardson was born in Detroit in 1882; he was educated in Seattle Public Schools and the University of Washington (Sigma Nu). He was in general engineering practice from 1906 to 1916, was captain of Engineers AEF in France 1918 to '19; senior examiner, U. S. Shipping Board, Emergency Fleet Corp.; Assistant Engineer in Washington State Highway Department; District Engineer, Portland Cement Association; Consulting Engineer Portland Cement Association of Utah some time prior to his death.

Fred A. Davis

who had been a member of the Institute since 1929 and Materials Engineer and head of the Department of Tests of the State Road Commission of West Virginia, died December 17, 1944, in a Morgantown hospital after a long illness. Word of his death only recently came to the Institute. Mr. Davis had joined the road commission as assistant engineer in 1921 and after serving several terms, as City Engineer of Morgantown which was his birth place. He was graduated C. E., from West Virginia University in 1898; started his engineering work in the employ of the Baltimore & Ohio Railroad; he was consulting engineer during the building of the "short line" from Clarksburg to New Martinsville. At one time he was connected with Clark and Krebs, Charleston engineering firm, developing coal lands; was later with the Pittsburgh engineering concern of Heyl & Patterson. He was also a member of the American Society for Testing Materials.

Lith-I-Bar Associates

manufacturers of concrete joists of the Lith-I-Bar type have organized to develop and promote their concrete joist construction for floors and roofs. Members of the organization include: The Formigli Corp. Philadelphia, Pa., The Dextone Co., New Haven 3, Conn., Gravel Products Corp., Buffalo, N. Y., Otto Buehner & Co., Salt Lake City 5, Utah, Lith-I-Bar Co. of Calif., Los Angeles, Calif., Economy Cast Stone Co., Richmond, Va., Arnold Stone Co., Greensboro, N. C., Cambridge Cement Stone Co., Boston, Mass. Office headquarters are in New Haven, 3, Conn. P. O. Box 606 with Herman Frauenfelder, Secretary.

The 42nd Annual

ACI CONVENTION

New Yorker Hotel

New York City

February 18 to 21

1946

Have You a Volume X ACI Proceedings to Spare?

The civil engineering department of a university needs Vol. X, 1914, Proceedings of the American Concrete Institute to complete its set. Somebody may have an extra copy, and if so, the Institute would be glad to know it and if it's in good condition, to pay the full membership price of the volume—$5.00.

This Journal is very late due to acute man power shortage at our printers. We are sorry. The releases from the armed services and the war plants apparently have not included many printers.

ACI publications in large current demand

ACI Standard—1945

148 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); three recommended practices: Use of Metal Supports for Reinforcement (ACI–319-42); Measuring, Mixing and Placing Concrete (ACI 614-42), Design of Concrete Mixes (ACI 613-44), and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—all between two covers, $1.50 per copy—to ACI Members, $1.00.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-entraining Portland Cements or Air-entraining Materials Added to Batch at Mixer" and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944. $1.25 per copy; 75 cents to Members.

ACI Manual of Concrete Inspection (July 1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI members 75 cents.

"The Joint Committee Report" (June 1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. Published June 15, 1940; 140 pages. Price $1.50—to ACI members $1.00.

Reinforced Concrete Design Handbook (Dec. 1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI members.

Concrete Primer (Feb. 1928)

Prepared for ACI by F. R. McMillan, it had five separate printings by the Institute alone (totalling nearly 70,000 copies). By special arrangement it has been translated and published abroad in many different languages. It is still going strong. In the foreword the author said "This primer is an attempt to develop in simple terms the principles governing concrete mixtures and to show how a knowledge of these principles and of the properties of cement can be applied to the production of permanent structures in concrete." 46 pages, 25 cents (cheaper in quantity)

For further information about ACI Membership and Publications (including pamphlets presenting Synopsis of recent ACI papers and reports) address:

AMERICAN CONCRETE INSTITUTE 742 New Center Building Detroit 2, Michigan

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fourth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1945 issue and (when it is completed) in V. 41, ACI Proceedings. **Watch for the 5th Annual Technical Progress Section in the February 1946 JOURNAL.**

Concrete Products Plant Equipment

page

Besser Manufacturing Co., 800 45th St., Alpena, Mich. 428
—Concrete products plant equipment, production

Stearns Manufacturing Co., Inc., Adrian, Mich. 387
—Vibration and tamp type block machines, mixers and skip loaders

Construction Equipment

Baily Vibrator Co., 1526 Wood St., Philadelphia 2, Pa. 407
—Concrete vibrators

Blaw-Knox Division of Blaw-Knox Co., Farmers Bank Bldg., Pittsburgh, Pa. 394-5
—Truck mixer loading and bulk cement plants, road building equipment, buckets, batching plants, steel forms

Butler Bin Co., Waukesha, Wis. 421
—Central mix, ready-mix, bulk cement and batching plants, cement handling equipment

Chain Belt Co. of Milwaukee, Milwaukee, Wis. 418-9
—Mixers, pavers, pumps

Electric Tamper & Equipment Co., Ludington, Mich. 410-11
—Concrete vibrators

Flexible Road Joint Machine Co., Warren, Ohio 389
—Pavement joint and joint installers

Fuller Co., Catasauqua, Pa. 377
—Unloading and conveying pulverized materials

Heltzel Steel Form & Iron Co., Warren, Ohio 378-9
—Pavement expansion joint beams

Jaeger Machine Co., The, Columbus, Ohio 392-3
—Concrete paving equipment

C. S. Johnson Co., The, Champaign, Ill. 429
—Mixing and batching plants, buckets, elevators

Kelley Electric Machine Co., 287 Hinman Ave., Buffalo 17, N. Y. 422-3
—Floor finishing equipment

Koehring Co., Milwaukee, Wis. 380
—Tilting and non-tilting construction mixers

Mall Tool Co., 7703 So. Chicago Ave., Chicago 19, Ill. 427
—Concrete vibrators

Master Vibrator Co., Dayton 1, Ohio 430-1
—Concrete vibrators

Ransome Machinery Co., Dunellen, N. J. 433
—Mixers—paving, truck

Viber Co., 726 So. Flower St., Burbank, Calif. 381
—Concrete vibrators

Contractors, Engineers and Special Services

Kalman Floor Co., Inc., 110 E. 42nd St., New York 17, N. Y. 390-1
—Floor finishing methods

Prepakt Concrete Co., The, and Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio 397-400
—Pressure filled concrete

Raymond Concrete Pile Co., 140 Cedar St., New York 6, N. Y. 396
—Pile foundations

Roberts and Schaefer Co., 307 No. Michigan Ave., Chicago 1, Ill. 426
—Thin shell concrete roofs

Scientific Concrete Service Corp., McLachlen Bldg., Washington, D. C. 416
—Mix controls and records

Vacuum Concrete, Inc., 4210 Sansom St., Philadelphia 4, Pa. 409
—Suction control of water in the concrete

Materials and Accessories

Calcium Chloride Assn., The, 4145 Penobscot Bldg., Detroit 26, Mich. 420
—Calcium chloride

Concrete Masonry Products Co., 140 W. 65th St., Chicago, Ill. 385
—Non-shrink metallic aggregate

Dewey and Almy Chemical Co., Cambridge, Mass. 412-5
—Air-entraining and plasticising agents

Horn Co., A. C., Long Island City 1, N. Y. 417
—Waterproofing

Hunt Process Co., 7012 Stanford Ave., Los Angeles 1, Calif. 425
—Curing compound

Inland Steel Co., The, 38 So. Dearborn St., Chicago 3, Ill. 384
—Reinforcing bars

Lone Star Cement Corp., 342 Madison Ave., N. Y. 382-3
—Portland cements

Master Builders Co., The, Cleveland, Ohio, Toronto, Ont. 402-5
—Cement dispersing and air-entraining agents

Rail Steel Bar Association 386
—Reinforcement bars

Richmond Screw Anchor Co., Inc., 816 Liberty Ave., Brooklyn 8, N. Y. 408
—Form tying devices

Sika Chemical Corp., 37 Gregory Ave., Passaic, N. J. 434-5
—Waterproofings, plasticizer, and densifier

United States Rubber Co., Rockefeller Center, New York 20, N. Y. 401
—Form lining

Testing Equipment

American Machine & Metals, Inc., East Moline, Ill. 388
—Riehle hydraulic testing machines

Why one husband kissed his wife four times!

"Here's a kiss for the money you're saving...while it's coming in faster through the the war years. I know in my bones jobs like mine may not last forever. Who can tell what's going to happen day-after-tomorrow? Thank God you've got sense enough to see that today's the time to get a little money tucked away.

"Here's a kiss for the War Bonds you're making me hold on to! I'd never do it without you, honey; it's too easy to find reasons for cashing 'em in—but when it comes time to put the children through school or pay for an emergency operation, we'll be thankful.

"Here's a kiss for the insurance you talked me into buying. I've felt a lot easier ever since I've known our future is protected—you and the kids would be safe if anything happened to me—you and I won't have to spend our old age living on someone's charity. And every cent we put in insurance or War Bonds or other savings helps keep prices down.

and

here's a kiss for being *you*—a woman with brains enough in your pretty head to make sure we don't buy a single thing we *don't* need in times like these—because you know a crazy wave of spending in wartime would march America straight into inflation. Baby, I sure knew how to pick 'em the day I married you!

ONE PERSON CAN START IT!

You give inflation a boost

- —when you buy anything you can do without
- —when you buy above ceiling or without giving up stamps (Black Market!)
- —when you ask more money for your services or the goods you sell.

Save your money. Buy and hold all the War Bonds you can afford—to pay for the war and protect your own future. Keep up your insurance.

HELP US KEEP PRICES DOWN

A United States War message prepared by the War Advertising Council; approved by the Office of War Information; and contributed by this magazine in cooperation with the Magazine Publishers of America.