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Resuming, with this volume year, the former JOURNAL publication schedule of 10 issues instead of 6 for the year, the Supplement, issued in recent years with the November issue, is mailed with this December JOURNAL as Part 2. It contains Title Page, Table of Contents, Closing Discussion and Indexes, concluding the volume otherwise completed in the issue of last June.

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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

● **Note proposed revisions to the "ACI CODE" as reported by ACI COMMITTEE 318 on p. 401.**

● With the September 1946 issue of the Journal, ACI resumed its pre-depression publication schedule of ten issues a year—monthly, September to June—instead of six issues as in recent years.

● The annual Supplement, usually issued with the November Journal, is mailed this year with this number as Part 2. It contains Title Page, Table of Contents, Closing Discussion and Indexes for Proceedings volume 42, otherwise completed with the June 1946 Journal.

● Discussion of the papers and reports in this issue is invited. As with all contributions submitted for the consideration of the Publications Committee with a view to publication, triplicate copies of discussion should be addressed to Secretary, ACI Publications Committee, New Center Building, Detroit 2, Mich. Closing date for discussion of December contents is April 1, 1947.

● 43rd Annual ACI Convention, Cincinnati, Ohio, February 24, 25, 26, 1947. Circle those dates on your calendar.

[To facilitate selective distribution, **separate prints** of this title (43-12) 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 April 1, 1947]

Title 43-12 —a part of PROCEEDINGS, AMERICAN CONCRETE INSTITUTE **Vol. 43**

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Vol. 18 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **December 1946**

The Structural Effectiveness of Protective Shells on Reinforced Concrete Columns*

By F. E. RICHART†
Member Amercain Concrete Institute

SYNOPSIS

This paper presents a study of 108 plain, tied or spirally reinforced concrete columns. The columns were 7, 8 and 9 in. round or square, 45 in. long, and the ties and spirals were 6 in. in diameter.

The columns were loaded axially, with "flat" ends. Strains were measured and close observations were made of the initial failure of the protective shell.

Analyses of the test results were made to see if the column shells were fully effective. This was the case with the shells of spirally reinforced columns, but the tied columns showed a slight deficiency in the strength expected on the basis of previous tests of the 1930 ACI column investigation.

The test results lend support to the design methods prescribed in the current ACI Building Regulations for Reinforced Concrete.

INTRODUCTION

The structural effectiveness of the concrete shell enclosing the reinforcing unit of a concrete column has long been a controversial subject. For many years the design of spirally reinforced columns was based on consideration of only the area of the concrete core, probably because in tests the shell concrete cracks and spalls off before the spiral comes into action. The 1924 Joint Committee Report (1)‡ recommended use of the core area for spirally reinforced columns but specified the gross area for tied columns. This does not seem consistent with any idea of fire protection, since if the shell area is reserved for fire protection purposes for spiral columns, it certainly should be likewise reserved for tied columns, which are likely to be more vulnerable to fire damage than those containing spirals.

*Received by the Institute, September 20, 1946.
†Research Professor of Engineering Materials, University of Illinois.
‡See references end of text.

The 1940 Joint Committee Report [2] and the current ACI Building Regulations [3] introduced a concept of spiral column behavior which makes it logical to use the gross area for all column sections. It is well known that both the shell and the spiral reinforcement contribute in a definite, calculable way to the strength of a column, but never act simultaneously, so that only one of the two should appear in a column formula. By specifying that enough spiral reinforcement be used to slightly exceed the strength contributed by the shell, the toughening effect of the spiral is provided as assurance against a sudden failure, but the spiral is not designed to provide a large contribution of strength after the shell fails. This design envisions the spiral as an insurance factor, a second line of defense, but does not call on it as an element of added strength, which would require a huge shortening of the column for its development. Such a spiral column develops its maximum strength by the action of the vertical steel and the overall concrete section; it is relatively stiff, the concrete section remains intact and the deformation is small (.0015 to .002) right up to the maximum load. This may be compared with spirally reinforced columns of what may be termed the European style of design, which may require a shortening of 1 percent or more at the maximum load, a complete spalling of the shell concrete and a tangent modulus of elasticity at high loads less than 5 per cent of the original value.

Three questions have been raised regarding the effectiveness of shells: (1) Can sound, homogenous concrete be placed in the shell? (2) Will the spiral unit form a surface of cleavage between shell and core? (3) In tied columns, may the buckling of highly stressed vertical bars cause a premature splitting and failure of the shell?

DESCRIPTION OF TESTS

To secure information on some of the questions outlined, a series of tests was made in 1938 at the Talbot Laboratory, University of Illinois. The test columns used were both round and square, 7, 8 and 9 in. in diameter or width, and 45 in. long. Ties and spirals were circular, 6 in. in outside diameter. A few plain columns were 8 in. round or square.

The columns were made with three grades of concrete, having average compressive strengths of 2880, 4900 and 6280 psi. The vertical bars used in all reinforced columns were four ½-in. plain rounds, of hard grade steel. They were milled to exact length and were placed with the ends flush with the plane ends of the column.

The spiral reinforcement was of drawn wire of nine different sizes, having useful limit values ranging from 61,600 to 97,000 psi. In all cases the pitch of spiral was 1 in. Three designs of spiral were used: design A

(spiral equivalent to shell), complying very closely to the ACI Building Regulations, Section 1103; design B (spiral stronger than shell), with roughly 40 percent more spiral than design A; and design C (spiral weaker than shell) with roughly 40 percent less spiral than design A. Due to variations in the actual strengths of spiral wire and concrete, and the relatively few wire sizes available, it was not possible to produce the columns with the exact relations between spiral and shell strengths just specified.

The concrete, made with torpedo sand and a gravel of 1-in. maximum size, was machine mixed, and was placed with the aid of a Viber internal vibrator. No difficulty was found in securing sound concrete, with smooth surfaces.

All of the columns were cured 28 days in a standard moist room. They were tested with flat ends, following the usual procedure of applying loads in ten or more increments, taking strain measurements at each increment. Because the failure of the shells destroyed gage lines, measurements were taken with attached extensometers and were discontinued after spalling of the column shell began.

It is of interest that tests of 8-in. round and square columns, two of a kind, gave ratios of strength of column to that of 6 by 12-in. cylinder as follows: lean concrete, 0.83; medium concrete, 0.84; rich concrete, 0.87; average, 0.85.

RESULTS OF TESTS OF SPIRALLY REINFORCED COLUMNS

Shell failures

Nearly all of the columns of normal design A and all of design C, deficient in spiral, failed when the protective shell began to spall. While with further compression and shortening some of these columns developed a second "maximum" load due to the action of the spiral, this load never exceeded the load at first spalling.

Two columns of design A, and all of design B with an excess of spiral, developed considerable additional load after the shell failed. They furnished useful information, however, on the effectiveness of shells at the spalling load.

For the columns which developed shell failures, the column strength may be expected to be the sum of the strength of the vertical steel, at its yield point stress, and the gross section of the concrete at its ultimate capacity. From the results of many previous tests[4], this may be written

$$P = 0.85\,(A_g - A_s)\,f'_c + A_s f_y \qquad (1)$$

The notation used is defined in Table 1. The factor 0.85 represents an experimental determination of the ratio of the strength of concrete in a

column 7½ diameters in length to that in a 6- by 12-in. cylinder, and is based on a large number of tests. It is proposed to insert the results of the present series of tests in eq. (1) and solve for the numerical factor. If the result is 0.85 or more, it would seem to indicate complete effectiveness of the shell concrete, or, that with the varying length-diameter ratios used in the tests, the factor should be slightly greater than 0.85.

The results of this analysis are given in Table 1. From the total load at shell failure, P, the contribution of the vertical steel has been deducted; the remainder, divided by the product of concrete area $(A_g - A_s)$ and cylinder strength, f'_c, gives the experimental factor, C. The values of C are seen to range from 0.75 to 0.94, with a grand average of 0.83. The average value is 0.84 for columns with square shells and 0.82 for those with circular shells.

From the last column in Table 1, it is seen that the shell area for these tests columns represent an excessively large proportion of the total concrete area, 27 to 66 percent, with an average for the group of 49 percent. This is considerably greater than would generally be used in large building columns and constitutes a rather severe test of the shell areas. In view of this condition, the average value of the constant C of 0.83 for the total concrete area may be considered in good agreement with the expected value of 0.85. There is nothing to show that the slight deficiency is due to a shortcoming of the shell concrete any more than of the core concrete; if the shell concrete is to be charged with all of it, the deficiency is less than 5 percent. For all practical purposes this is negligible.

A brief study indicates that there is no consistent effect of class of concrete, shell thickness or design of spiral, as indicated by the following tabulation of average values of C from Table 1.

Class of concrete	Value of C	Minimum shell thickness	Value of C	Design of spiral	Value
A	0.831	½ in.	0.845	Normal—A	0.825
B	0.798	1 in.	0.821	Excessive—B	0.835
C	0.862	1½ in.	0.842	Deficient—C	0.833

The record of the tests shows no indication of lack of soundness or reliability of shell concrete and no indication of a surface of separation coincident with the position of the spiral. Failure of the shell occurred at strains corresponding to those at failure of the plain concrete. At these strains in the reinforced columns there was the usual plastic bulging of the core and the development of stress in the spirals.

TABLE 1—RESULTS OF SPIRAL COLUMN TESTS, SHELL FAILURES

The maximum load listed is the load at which there was definite failure of the shell, even though the column later developed additional strength. All columns were 45 in. long, with 6 in. core diameter; vertical bars, 4 ½-in. round; area, A_s = 0.76 sq. in.; yield point, f_y = 50850 psi.; $A_s f_y$ = 38.5 kips. A_g is the gross or overall area of the column. Spiral design A has spiral roughly equivalent to shell; design, B, spiral stronger than shell, design C, spiral weaker than shell. Maximum load, as defined above,
$P = C\,(A_g - A_s)\,f'_c + A_s f_y$, whence

$$C = \frac{P - A_s f_y}{(A_g - A_s) f'_c}$$

Value of C, from previous tests, was 0.85. Two columns of a kind.

Column		Spiral design	Concrete		Total concrete area $A_g - A_s$ sq. in.	Max. load, P kips	Total load on concrete P-$A_s f_y$, kips	Factor, C for concrete	Av. ratio, shell area / $A_g - A_s$
Size, in.	Shape		Class	Comp. strength f'_c, psi.					
8	square	A	A	3240	63.86	194.0	155.5	.750	
		B	A	2360	63.32	165.0[1]	126.5	.845	
		C	A	2405	63.38	173.5	135.0	.885	.565
8	square	A	B	4900	63.40	293.0	254.5	.819	
		B	B	5000	64.04	294.5[1]	256.0	.800	
		C	B	4455	63.32	262.0	223.5	.810	.566
8	square	A	C	6145	63.16	376.8[1]	338.3	.872	
		B	C	6235	62.77	379.0[1]	340.5	.870	
		C	C	5520	63.56	362.0	323.5	.920	.564
7	square	A	A	2555	48.31	137.5	199.0	.801	
		A	B	4650	49.30	219.5	181.0	.788	
		A	C	5605	48.52	275.0	236.5	.867	.434
9	square	A	A	3265	81.50	271.5	233.0	.875	
		A	B	4985	82.50	374.5	336.0	.815	
		A	C	6465	81.96	510.0	471.5	.891	.665
Av.	square	ABC	ABC					.840	.559
8	round	A	A	3240	48.69	165.5	127.0	.804	
		B	A	2360	48.57	138.5[1]	100.0	.887	
		C	A	2405	48.94	135.0	96.5	.820	.436
8	round	A	B	4900	48.81	222.5	184.0	.779	
		B	B	5000	49.25	219.5[1]	181.0	.735	
		C	B	4455	48.70	208.0	169.5	.780	.438
8	round	A	C	6145	48.81	265.0[1]	226.5	.750	
		B	C	6235	48.82	304.0[1]	265.5	.871	
		C	C	5520	48.69	248.5	210.0	.781	.436
7	round	A	A	2555	37.24	117.8	79.3	.833	
		A	B	4650	37.56	185.5	147.0	.840	
		A	C	5605	37.72	238.0	199.5	.942	.267
9	round	A	A	3265	62.86	204.8	166.3	.810	
		A	B	4985	62.91	291.0	252.5	.840	
		A	C	6465	62.93	386.5	348.0	.856	.563
Av.	round	ABC	ABC					.820	.428
Grand Av.	Square & round	ABC	ABC					.830	

[1]These columns carried additional load due to spiral, after shell failed.

Spiral failures

Table 1 indicates that after the shell had failed, certain columns of spiral designs A and B took further load, and after a large amount of shortening developed a maximum load. The significant results of these tests are given in Table 2, together with an analysis based on the conventional equation[4] for the strength of a spirally reinforced column. The maximum load, P, is as follows:

$$P = 0.85\, A_c f'_c\, (1\text{-}p) + A_s f_y + K\, p' f'_s\, A_c \qquad (2)$$

The meaning of the notation is given in Table 2. The three terms in the right hand member of the equation represent the contributions of the concrete core, the vertical bars, and the restraining effect of the spiral, respectively. By substituting the known areas of sections, the percentages of reinforcement and the experimental values of cylinder strength f'_c, the yield point stress of vertical bars, f_y, and the useful limit of the spiral steel, values of the factor K are obtained. The useful limit of the spirals is an arbitrary measure of the spiral strength at a strain of 0.005 and was proposed by the writer in 1931 [5] after a study of measured spiral deformations at or near maximum load in a large number of test columns.

TABLE 2—RESULTS OF SPIRAL COLUMN TESTS, SPIRAL FAILURES

Maximum Load, P, is load at complete failure of column. Columns 45 in. long, two of a kind. Core diam., 6 in. Core area, A_c, = 28.3 sq. in. Vertical bars, 4 ½-in round, $A_s f_y$ = 38.5 kips. Per cent spiral p' by volume of core; f'_s = useful limit stress in spiral. Vertical steel ratio, p = .0268, on core area. Spiral design A, spiral roughly equivalent to shell, design B, spiral stronger than shell. Spiral effectiveness factor, K, is computed from equation

$$K\, p' f'_s A_c = P - 0.85\, A_c f'_c\, (1\text{-}p) - A_s f_y$$

Concrete		Steel reinforcement					Max. load, P kips	$.85 A_c f'_c (1\text{-}p) + A_s f_y$, kips	$K p' f'_s A_c$ kips	Spiral effectiveness K
Class	f'_c psi.	Design	A_s used $K p' f'_s A_c$ in column	p'	f'_s kips per sq.in.	$p' f'_s$ psi.				
C	6145	A	8 in. sq.	.0494	72.4	3575	408.5	182.1	226.4	2.24
C	6145	A	rd.	.0324	76.2	2470	323.0	182.1	140.9	2.02
A	2360	B	8 in. sq.	.0324	76.2	2470	199.7	93.7	106.0	1.52
A	2360	B	rd.	.0191	90.1	1720	159.0	93.7	65.3	1.34
B	5000	B	sq.	.0494	72.4	3575	323.0	155.5	167.5	1.66
B	5000	B	rd.	.0324	76.2	2470	275.0	155.5	119.5	1.71
C	6235	B	sq.	.0696	61.6	4285	423.0	184.3	238.7	1.97
C	6235	B	rd.	.0494	72.4	3575	377.0	184.3	192.7	1.91
Average										1.80

It is seen that the average value of K found from these tests is 1.80. The value which has been found in previous tests at this laboratory, particularly in the ACI Column investigation in 1930 [4], was about 2.0, though corresponding tests made at Lehigh University gave a considerably lower value. The current tests have rather unusual features; very high values of the useful limit stress in the spirals (61 to 90 kips per sq. in.) combined with spiral percentages of 1.9 to 7.0. The results indicate very satisfactory effectiveness of spiral even in the highest percentages employed. The fact that the spiral factor K is a little lower than the value of 2 assumed in the design probably explains why most of the columns having the normal spiral design A developed their maximum load through the shell rather than through the spiral contribution.

Results of tests of tied columns

The results of the tests of thirty six round and square tied columns are given in Table 3 and the results have been analyzed in the same way as those of the spirally reinforced columns of Table 1. As before, the factor C for the concrete is slightly greater for the square columns than for the round ones, though the difference is hardly enough to be significant.

The feature that is surprising, and somewhat disturbing, is that the factor C averages only about 0.75 instead of the value 0.85 expected. These columns were designed to be identical, except for the use of ties instead of spirals, with a part of the columns of Table 1. The vertical bars were identical in amount, quality and position, and the concrete was of the same proportions. There is little variation in C due to size of column or kind of concrete, though the smaller columns and the stronger concretes show a slight superiority.

In other tests, the value of C equal to 0.85 has generally been found to apply equally well to plain, tied or spiral columns. The reason for the low value of 0.75, about 11 percent lower than expected, is not apparent. It might be surmised that the ties and the concrete shell were not sufficient to prevent premature buckling of the vertical bars as the yield point was approached, but nothing about the manner of failure indicated this. Failure of all elements of the columns apparently took place at the same instant, suddenly and without warning. Furthermore, many of the previous tests had virtually no shells at all to restrain the buckling of the bars.

It should not be implied that the results of this group of tests should nullify or supersede the results of many equally reliable previous tests on which the conventional value of 0.85 was based. The results, however, combined with the sudden type of failure characteristic of tied columns, do furnish a good argument for requiring a higher factor of safety for this type of column. The current ACI Building Regulations,

TABLE 3—RESULTS OF TIED COLUMN TESTS

All columns 45 in. long, 7, 8 and 9 in. round or square. Vertical bars, 4 ½-in. round; yield point 50,850 psi. $A_s = 0.76$ sq. in., $A_s f_y = 38.5$ kips. Ties, 6 in. diameter, No. 9 wire, 6 in. spacing. Two columns of a kind.

$$C = \frac{P - 38.5}{(A_g - A_s) f'_c}$$

Column		Concrete		Total concrete area sq. in. $(A_g - A_s)$	Maximum load P, kips	Factor, C, for concrete	
Size, in.	Shape	Class	Comp. strength f'_c, psi.				
7	square	A	2520	48.31	129.8	.750	.788
		B	4190	48.38	212.0	.857	
		C	5820	47.05	245.0	.755	
8	square	A	2955	63.96	173.0	.711	.791
		B	5075	64.52	305.2	.815	
		C	6665	62.92	394.0	.848	
9	square	A	3050	81.51	219.2	.727	.750
		B	5640	82.51	346.2	.664	
		C	6420	82.13	491.8	.860	
Average						.776	
7	round	A	2520	37.67	109.3	.745	.743
		B	4190	37.46	160.0	.774	
		C	5820	37.45	193.5	.711	
8	round	A	2955	48.56	137.5	.691	.735
		B	5075	48.88	226.0	.762	
		C	6665	48.81	283.3	.753	
9	round	A	3050	61.66	177.0	.738	.729
		B	5640	63.14	286.3	.697	
		C	6420	62.20	339.5	.753	
Average						.736	
Grand Average						.756	

based on tests in which the factor 0.85 was found, require a factor of safety 25 percent greater for tied columns than for those with spiral reinforcement. For the usual range of materials the ACI Regulations provide a factor of safety against failure ranging from 3.7 to 4.4. A little study shows that if the factor 0.75 is substituted in the ultimate load formula, the factor of safety under the ACI Regulations is still 3.5 to 4.0. Hence, even if these tests were the only source of information on the subject, it is seen that the ACI design procedure would still provide ample safety for such columns. In fact, compared to some European codes, it may be considered distinctly conservative.

GENERAL COMMENTS

The foregoing information seems to controvert two opinions frequently held among engineers. One was that the shells of tied columns, because of better conditions for placing the concrete, would surely be more reliable than those of spirally reinforced ones. The other, as stated in the 1928 ACI Building Code, was that while the full gross section of tied columns might be used without question in design, the shells of spiral columns should be neglected as a load carrying element. As used in the current ACI Building Regulations, the use of the gross area formulas for spiral columns appears fully justified.

In addition to the information from these tests, there are two groups of fairly recent European tests which contain contributions to the subject.

Austrian tests

A series of column tests, made by a subcommittee of the Austrian Committee on Reinforced Concrete, was reported in 1931 by the late Dr. F. Emperger[6]. The tests included forty types of plain, tied and spirally reinforced columns. The columns were generally 20 or 24 cm. square or 22.5 cm. in diameter and were 150 cm. in length.

From tests of a relatively small group of columns, Emperger concluded that for columns with heavy ties or light spirals the full gross area of the column could be relied upon, but for columns with light ties the shell might fail at strains in the neighborhood of 0.001 before the vertical steel had attained its full yield point stress. This does not agree with the writer's tests, in which the plain and tied columns developed strains of 0.0015 to 0.0020 and more before failure, but may have some bearing on the low concrete effectiveness in the present tests of tied columns.

Dutch tests

Column tests by the Dutch Concrete Association, Amsterdam, were reported briefly by N. J. Rengers in 1932 [7]. About 35 of the test columns were spirally reinforced, with 22.5 cm. core diameters, with 24 cm. square shells or without shells. The author gives the unqualified conclusion, and with relatively little analysis of the test results to support it, that with the weaker grade of concrete used (cube strength, 188 kg per sq. cm.) 80 percent of the shell area was effective, while with a stronger concrete (cube strength, 389 kg per sq. cm.) 100 percent of the shell area was effective. Since the yield point of the vertical bars is not known, it is difficult to make any accurate analysis of the results, but unless a steel of high strength was used, it appears that the effectiveness of the shells made with the weaker concrete must have been considerably more than the 80 percent quoted.

CONCLUSION

The tests described in this paper indicate that the shell concrete of spirally reinforced columns can be counted on for full effectiveness as a load-carrying element, if the concrete is properly placed and compacted.

The effectiveness of the concrete section of tied columns in these tests was only about 90 percent of the value expected on the basis of previous tests of tied columns.

The effectiveness of the spiral reinforcement in those columns in which the strength produced by the spirals was greater than that contributed by the column shells was 1.80, as compared with the value of 2.0, which is usually found from such tests. However, this is a factor which may be expected to vary considerably, and considering the high strengths and percentages of spiral reinforcement employed, this value may be considered as satisfactory.

The results of the studies of shell effectiveness would seem to support the present ACI design method in which the gross area of spirally reinforced columns is employed. Spiral columns designed on this basis have two very desirable physical characteristics, the relatively high stiffness of a tied column right up to the maximum load and a slow manner of failure, marked by the spalling of the shell, at the maximum load.

The rather small group of tied column tests indicates a little less effectiveness of the shell concrete than was expected on the basis of other tests; this, together with the well-known sudden and violent type of failure observed in the tied columns, is consistent with the ACI design provisions, which call for a factor of safety for tied columns 25 percent greater than that for spirally reinforced columns.

ACKNOWLEDGMENT

The foregoing tests were conducted as a part of the work of the Engineering Experiment Station of the University of Illinois, under the administrative direction of Dean M. L. Enger, Director, and Professor F. B. Seely, Head of the Department of Theoretical and Applied Mechanics. Special acknowledgment is made to Richard H. Heitman, former Research Graduate Assistant, who carried out the testing program.

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Vol. 18 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **December 1946**

Precast Concrete Structures*

By A. AMIRIKIAN†
Member American Concrete Institute

SYNOPSIS

Precasting is becoming a major factor in the choice of reinforced concrete as a construction material because of ever-rising cost of labor and materials. The advantages of precasting are not however confined to savings in cost and materials. Since it is a planned method of construction, comparable to factory production, its use also assures a better control of quality and speedier completion of the project. This article is an attempt to show how precasting can be utilized to provide the framing of a great variety of structures. The first part deals with bent type of framing as used in buildings, the second describes a novel type of framing consisting of precast cells, particularly suitable for floating structures.

GENERAL CONSIDERATIONS

Weight and strength are primary considerations in precasting. A successful design is one which utilizes the smallest number of assembly elements possessing the least weight during erection and the greatest strength per unit weight of framing.

Since precasting is a controlled operation, performed under ideal conditions of forming, pouring, vibration, curing and inspection, the requirements of strength and the dimensional restrictions necessary for the casting of slender sections are generally met with but little difficulty. Obviously, the technique of casting will differ in many details from that of a conventional poured-in-place job. The basic features include the following:

Position of pours

All casting is accomplished in the flat or horizontal position of the element, thus assuring ease of pours, control of quality and thinness of sections unobtainable in vertical pours.

*Presented 42nd Annual Convention, Buffalo, N. Y., February 21, 1946.
†Head Designing Engineer, Bureau of Yards and Docks, U. S. Navy Dept., Washington, D. C.

Form work

Container forms of cast elements are of rigid construction for a close maintenance of dimensions and may have rather elaborate details and outline. These forms usually are built of steel or concrete, suitable for multiple and repeated use. In accordance with the shape of the elements, they may be either stationary pan type or rotary box type.

Minimum thickness for casting

With the aid of vibration and positioned pouring, it is possible to cast structural elements as thin as ½ in.

Grade of concrete

Thin sections and lighter weight elements will require the use of high strength concrete, with a nominal 28-day strength of 4000 psi to 5000 psi.

Size of aggregate

In most cases the size of the coarse aggregate will be limited to ¼ in.

Reinforcing

The reinforcing of plaques, webbing and cellular elements will consist mainly of high-tensile wire fabric of rather close mesh, with a minimum yield value of 70,000 psi. In some cases wire fabric may be supplemented with ordinary concrete reinforcing bars.

Cover and spacing of reinforcing

The minimum cover for wire mesh may vary from ¼ in. to ⅜ in., and the clear spacing between reinforcing rods may be reduced to 1 in.

Casting tolerance

Elements may be cast with a tolerance of $\frac{1}{32}$ in. as regards the thickness and to an accuracy of $\frac{1}{16}$ in. in overall dimensions.

Shapes of cast elements

The cross section of the element may generally conform to one of the following outlines: (a) solid rectangular with or without ribs, as in slabs and plaques; (b) channel, tee- or ell-shaped; and (c) hollow rectangular, as in a cell or box.

Types of framing

According to framing arrangement, types of precasting may be grouped under two general headings: (a) bent framing and (b) cellular framing. The arrangement of the first type is in conformity with the so-called conventional framing as generally utilized in building construction. The second type is rather novel as regards arrangement and presents a new medium of framing for structures afloat and on shore. A brief description of each type, covering general characteristics, basic details and the extent of application, follows.

1. BENT FRAMING

General arrangement

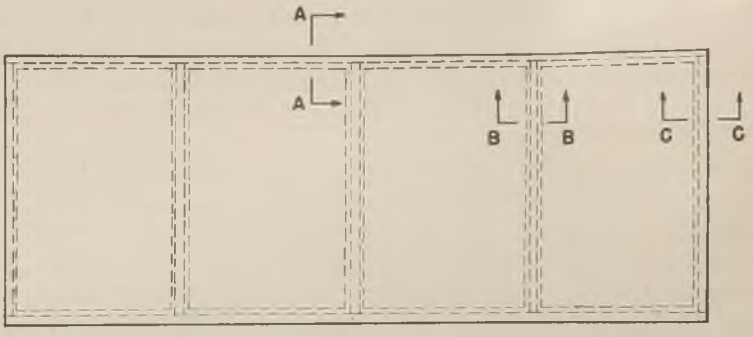
The main components of a building frame consist of the floor and roof members and the supporting bents in the form of arch ribs, rigid frames and continuous or simply-supported girders and beams.

Floor, roof and wall panels

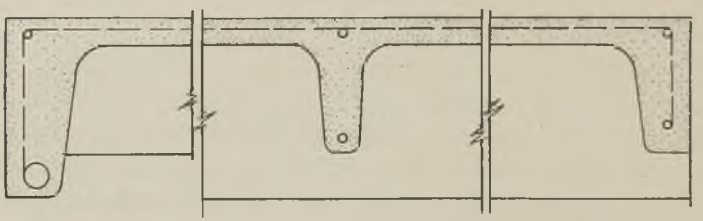
In devising a precast element to span the main supports or frames, one is obviously influenced by the framing arrangement of a conventionally designed poured-in-place job. In that type of construction, the floor or roof slab is supported either directly by the main framing, as a one-way or two-way slab, or by stringers which in turn frame into the bents. Except for conditions of unusual loading, the latter arrangement will generally result in a framing of the lightest weight. In addition to the type of framing, the choice of precast elements is affected by such other considerations as limitations in over-all dimensions and weight for assuring ease in handling, a minimum amount of jointing and simplicity in connections.

Fig. 1 (a) illustrates a precast panel having a framing arrangement in conformity with conventional type of construction. As will be noted, the unit consists of a slab, two longitudinal edge beams or girders and a series of transverse beams which divide the panel into sub-panels of square or rectangular outline. The length of the panel may vary from 16 ft. to 30 ft. and the width from 4 ft. to 8 ft., in accordance with facilities of erection. For a live load of up to 75 psf, the corresponding weight of the panel will vary between a minimum of about ¾ ton to a maximum of about 6 tons. A suggested detail for connecting the panels together and anchoring them to the supports is shown in Fig. 1 (b).

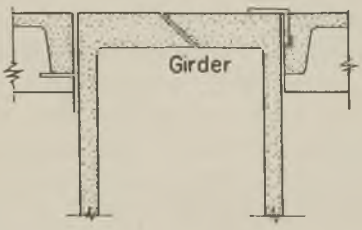
As an alternative to the flat-panel framing arrangement, the unit may be cast in the form of a half-barrel shell, stiffened with a series of diaphragms. Three typical panels illustrating this method of framing are shown in Fig. 2. The panel in (a) represents the simplest type, and is suitable for roof or wall framing. For floor construction, the panel may be somewhat modified by the addition of exterior diaphragms to form supports or chairs for the flooring. While in general the unit will consist of a single half-barrel, the section may also be made of multiple half-barrels, as shown in (b) and (c), according to the requirements of framing and facilities of erection. Due to curved shell action, the weight of a half-barrel panel will be considerably less than that of a comparable flat panel. As regards casting, no serious difficulties are anticipated, since all diaphragms may be precast and set in the form prior to pouring of the shell.

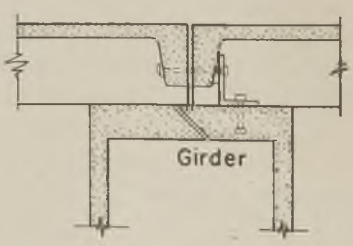


PLAN VIEW OF PANEL



Sect. A-A Sect. B-B Sect. C-C





Connection Details of Panels to Girder

Fig. 1—Flat-type roof or wall panel

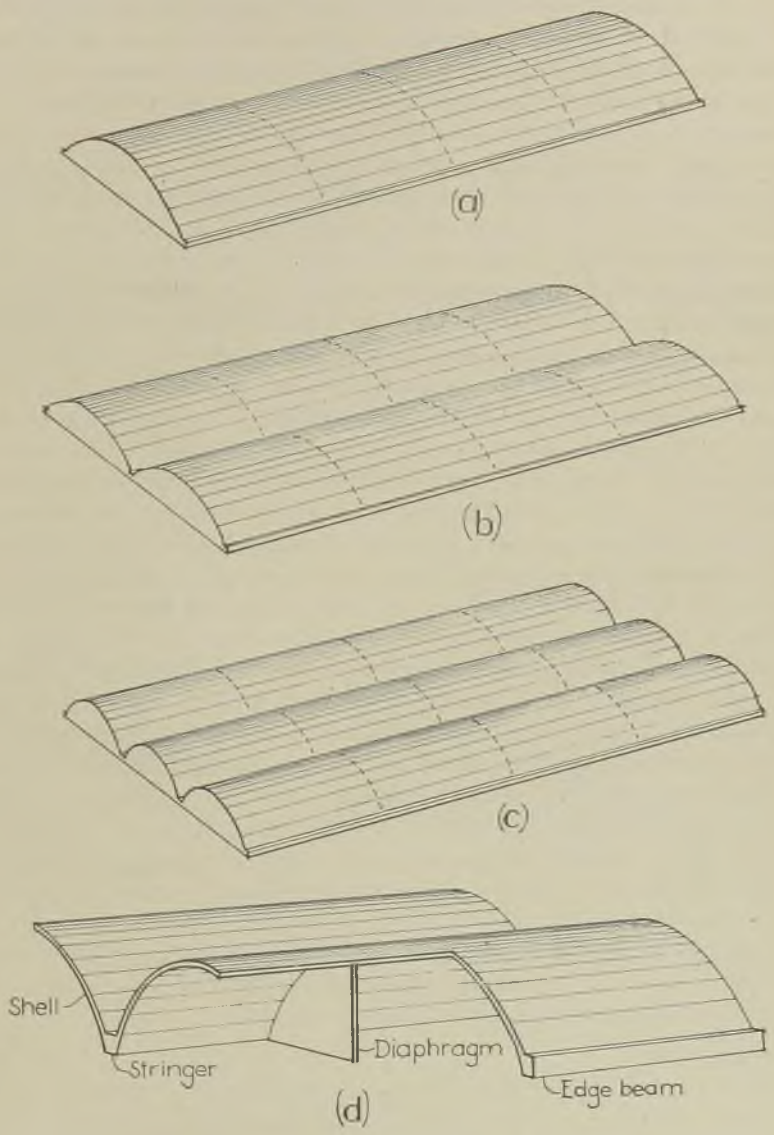


Fig. 2—Arch-type roof or wall panel

Girders

In devising a most favorable cross-sectional form for a precast beam, one is obviously guided by the fundamental principle of placing material at locations of optimum efficiency. The problem is one of furnishing sufficient section for bending and shear with the least cross-sectional area. Since the parts of a rectangular cross section which most effectively resist bending are those located farthest from the neutral axis, it is evident that a large reduction in the area of a solid section, as generally utilized in conventional concrete construction, can be made by removal of less effective parts of the core to form a hollow section. While the use of hollow sections may in general be found impracticable in poured-in-place construction, the positioned casting of such sections on the basis of planned production involves but little difficulties. In some cases the section may be cast as one unit; in most cases, however, it will be more convenient to cast it in two pieces in the form of channels.* Fig. 3 illustrates the latter method of casting as applied to a continuous beam or girder. The general arrangement of framing is shown in the sectional elevation, Fig. 3 (a), and a typical cross-section is given in Fig. 4. The channels have thin webs, reinforced with wire-mesh, and flanges of sufficient thickness to accommodate the main longitudinal reinforcing and to provide the necessary compression area. The same section also illustrates a suggested method for interconnecting the two channels. The arrangement consists of a pipe-separator assembly, Fig. 3 (d), cast either in the flanges of each channel or in embossed lips under the flanges, and held together by through bolts. Two types of beam splices are sketched in (b) and (c) of Fig. 3. In the former method the continuity of the member is maintained by lapping or welding some of the longitudinal rods of the abutting segments. For this purpose, a wedge-shaped opening is provided at the ends of the cast segments. After completion of the welding, the openings are closed by either guniting to the normal thickness of the cast element or by filling concrete in the splice compartment. In the other method of splicing, shown in Fig. 3 (c), the longitudinal reinforcing is spliced by looping and anchoring into the concrete fill of the splice compartment. For clarity of presentation, the wire mesh reinforcing is omitted from the details.

Rigid-frame bents

The basic features of framing described above for continuous beams are also applicable to rigid frames. The simple bent shown in Fig. 4 will serve for illustration. As will be noted, the assembly is composed of five main segments: a girder, two columns and two connecting knees. In some cases it may be possible to cast the column sections with the knees and thus reduce the number of the involved splices from four to two.

*Patent applied for by U. S. Navy in the name of A. Amirikian.

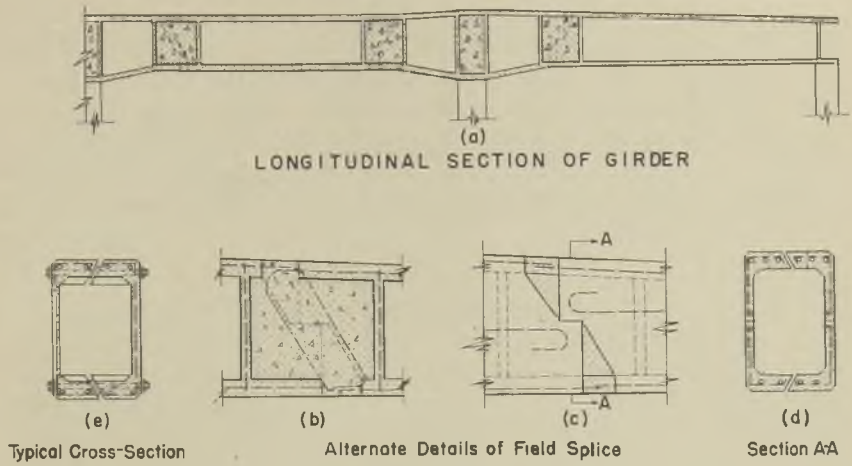


Fig. 3—Continuous girder

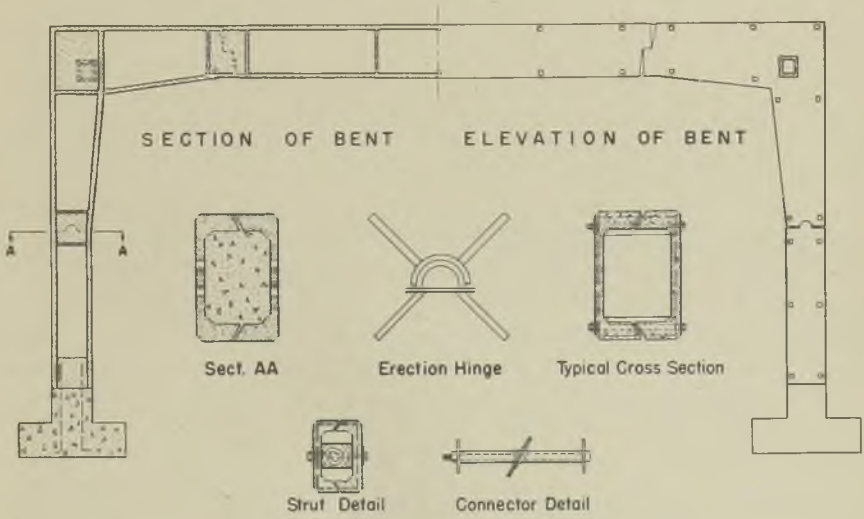


Fig. 4—Rigid-frame bent

Here again the component elements of each segment or member consist of two channels, as shown in the typical section in Fig. 4, joined together by a bolt-and-sleeve arrangement (connector detail). As an alternative to the latter detail, loose timber block separators may be substituted for the cast-in pipe-sleeves (see strut detail). To obviate possible over-strain on the concrete when tightening the bolts, welded washers are provided at the two ends of each sleeve, cast in the flanges with slight projections and serving as bearing plates.

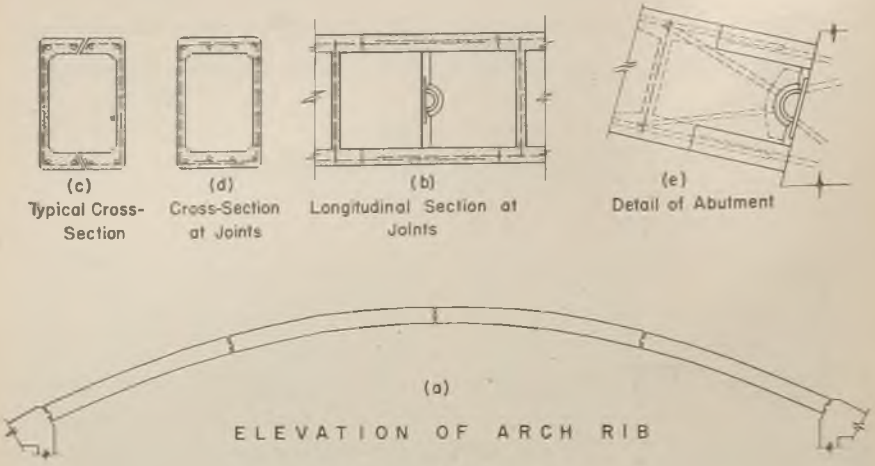


Fig. 5—Arch bent

Arch bents

The precasting of an arch bent, as shown in Fig. 5 (a), presents no different problems than those encountered in the construction of rigid-frame bents. The main assembly will consist of segments of either straight or curved outline and of hollow cross section. These segments may be cast either in one piece, as tubular units, or in two pieces in the form of channels as previously described. Hollow sections being particularly adaptable for the pattern of arch stresses, the use of these tubular segments of relatively lighter weight makes it possible to construct precast concrete building or hangar arches up to 500 ft. of span, with erection ease comparable to steel rib construction. The lengths of the erection segments, and the corresponding number of the field splices, will obviously vary in accordance with the handling and erection facilities available for each project. In general, arch ribs up to 200 ft. in span may be erected in three segments, consisting of a center and two abutment pieces. For shorter spans, say up to 120 ft., the latter two segments may be eliminated and the rib erected in one piece. A suggested detail for interconnection of the erection segments is shown in Fig. 5 (b).

2. CELLULAR FRAMING

Cellular slab

By definition, a cellular slab is an egg-crate framing arrangement in which interconnecting vertical web panels are anchored between two horizontal flange plates to form a series of cells. The evident advantage of a slab so composed over a solid slab lies in its greatly reduced weight

for comparable strength. While the basic idea of cellular framing is not new, since it is used to a certain extent in some types of steel structures, inherent difficulties and limitations in poured-in-place concrete work have precluded hitherto its utilization in concrete construction. However, precasting, with the aid of a new structural element, in the form of a precast cell, eliminates the involved difficulties and provides unlimited opportunities of application of the principle to a great variety of structures.

Concept of cell

As in the case of many inventions, the concept of the precast cell is traceable to the exigencies of war. Due to the unavailability of steel plating, it was necessary to use reinforced concrete as a substitute material for the construction of certain types of auxiliary floating craft for the Navy. The resulting structures obtained by the conventional poured-in-place construction technique, while structurally adequate, were however much too heavy in comparison with the steel prototypes. To reduce weight, consideration was given to the use of precast panels. However, the task of assembly and jointing of a great number of loose plates to form a given framing appeared too difficult for a satisfactory solution. It was in laboring with this practical phase of the problem that the idea of a cell or box-shaped structural element was conceived by the author.*

Detail of cell

Fig. 6 (a) illustrates the basic features of the element. Essentially, it is a rectangular box with two open ends and four faces which join to

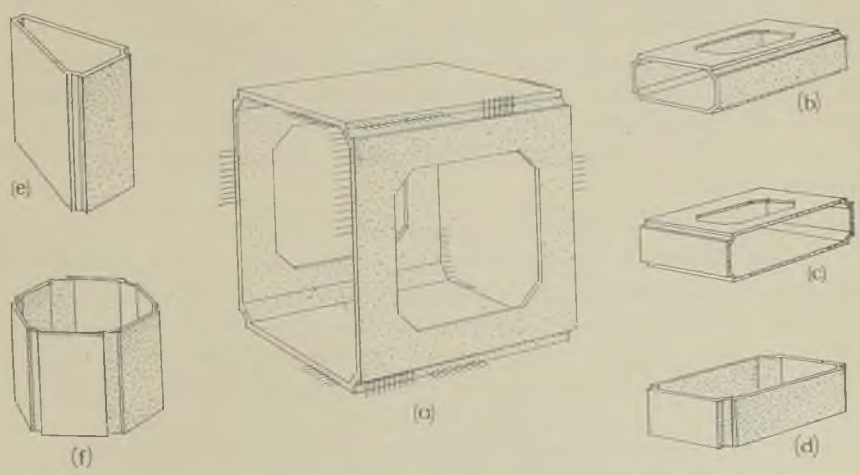


Fig. 6.—Typical precast cells

*Patent applied for by U. S. Navy in the name of A. Amirikian.

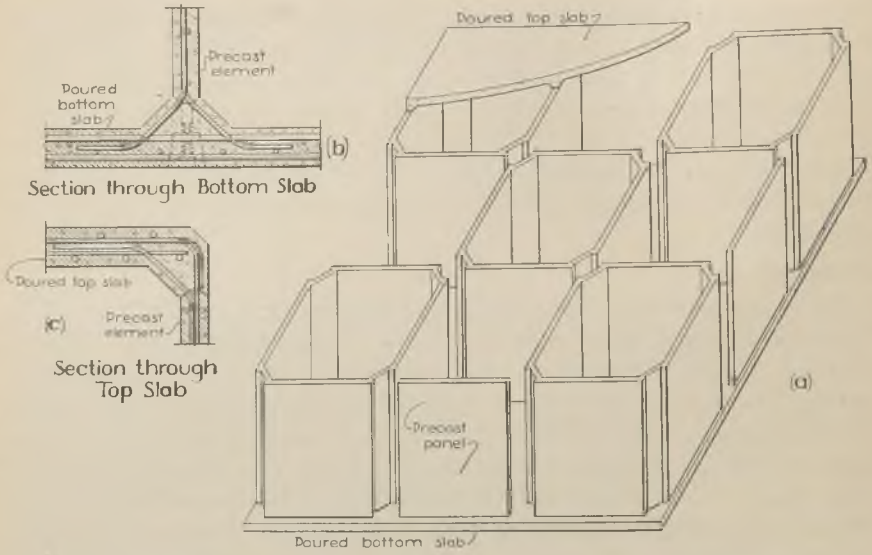


Fig. 7—Single-layer cell assembly with poured-in-place top and bottom slabs

form a channel-shaped groove at the corners. The faces may be solid or have openings as required. The wire-mesh reinforcing protrudes from all four edges of each face to serve as a means of connection splice between adjoining boxes of an assembly.

The box is cast on a rotary collapsible form, enabling each of the sides or walls to be poured continuously, in succession, in a horizontal position. This positioned pouring assures ease and control of operation, and enables the casting of very thin sections of great strength.

By varying the three over-all dimensions of the box, it may be shaped as a cube or as a prism, in accordance with the required framing arrangement. In cross-section, the box may have an outline other than rectangular, such as, triangular or hexagonal. Some of these forms are shown in Fig. 6.

Framing arrangement

Having described the characteristic of the box, it now remains to show how it can be utilized in a cellular framing. As stated above, the framing comprises a series of vertical bulkheads which intersect each other at right angles to form an egg-crate webbing, and two horizontal plates which serve as the top and bottom flanges of the slab. In accordance with two basic arrangements, a set of boxes may provide either a part of the framing, such as the egg-crate webbing, or the entire framing.

To obtain the first arrangement, the boxes are set on their ends, abutting cornerwise on a checker-board pattern, as shown in Fig. 7. With

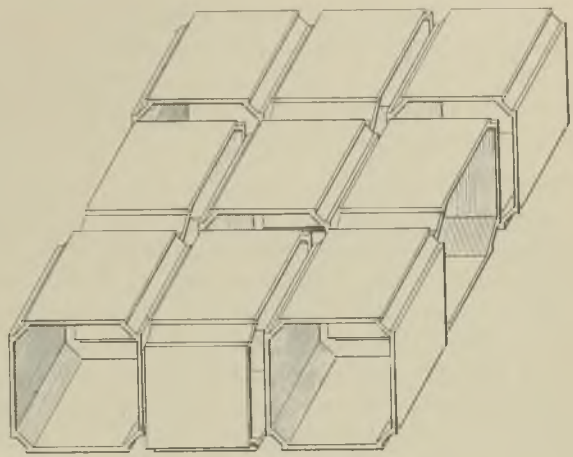


Fig. 8—Single-layer all-precast assembly

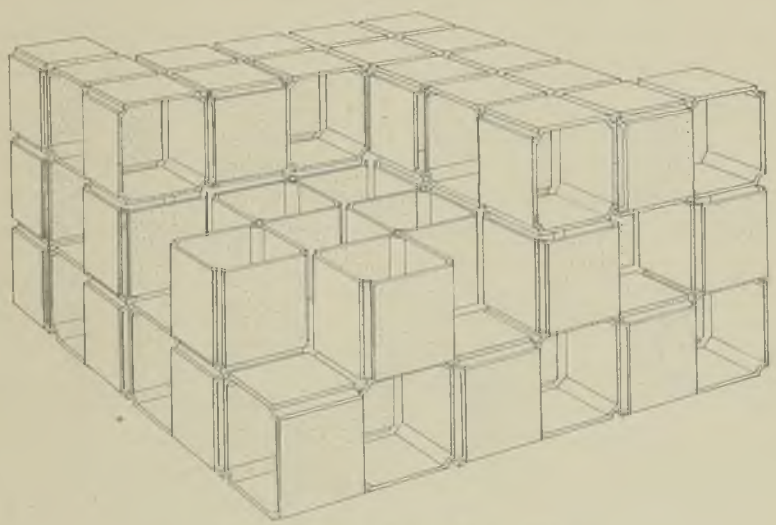


Fig. 9—Multiple-layer cell assembly

precast webbing thus furnished by the boxes, the framing is then completed by pouring in place the top and bottom flange slabs.

Fig. 8 illustrates the second arrangement. As will be noted, here the boxes are set on their sides and so oriented that the open end of one box faces the closed side of the adjacent box. With this arrangement, and after jointing, the sides of the boxes form continuous planes of single-wall thickness extending in three directions and thus providing precast elements for the entire framing.

These two basic single-layer arrangements may also be used to form a multiple-layer assembly. For this purpose, the boxes in the successive layers are set in an alternating sequence, using one arrangement in the first layer and the other pattern in the next. Fig. 9 illustrates a multiple-layer assembly thus obtained.

Method of assembly

The erection procedure of a cellular assembly is relatively simple. Since the boxes are inherently stable, their setting is accomplished without the aid of shoring, guying or other devices of support generally used in conventional construction. For example, in erecting the single-layer solid pattern shown in Fig. 8, the boxes are merely set side by side on an erection platform which is chalk-marked to maintain the required joint gaps between the boxes and to insure alignment of the walls in each direction. The erection of the single layer checkerboard pattern shown in Fig. 7 is equally simple. In this arrangement proper spacing and alignment of the boxes may be obtained by means of tapered plug inserts set in the bottom slab which engage threaded studs cast in the bottom edges of the boxes, as detailed in Fig. 7 (b).

In the case of the multiple-layer assembly illustrated in Fig. 9, after setting up the bottom layer as described above, the erection of the boxes in the succeeding layers may be accomplished by means of temporary supporting and spacer devices. Two suggested devices for this purpose are shown in Fig. 10. The detail in (a) is a simple jacking arrangement,

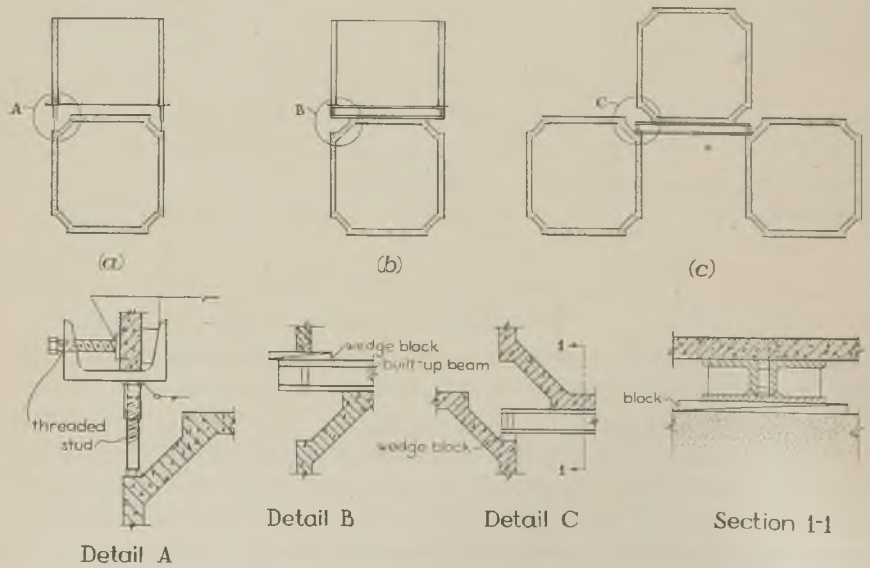


Fig. 10—Erection and assembly devices

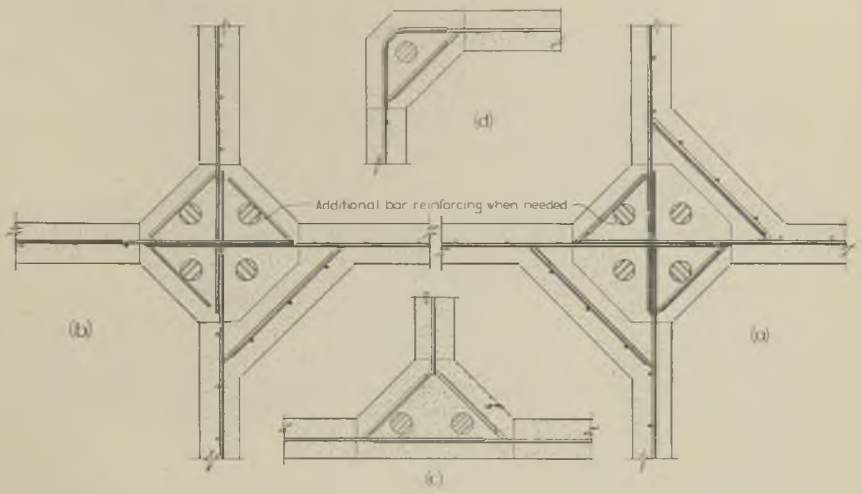


Fig. 11—Typical joint details of cells

particularly suitable for supporting the edge of one box on the corner ledge of the other, with the bottom bolt serving as support as well as spacer. The other arrangement shown in (b) and (c) consists of a built-up beam with wedge-block ends and can be used as a support either for the edges of a box, as in (b), or the bottom side of a supported box as indicated in (c) and detail "C."

Method of jointing

Since relatively small joint gaps are desirable, the interconnections of the boxes in an assembly are most conveniently accomplished by use of pressure grout. Fig. 11 illustrates typical joint details between elements having a single-layer mesh reinforcing. The section in (a) is for two boxes abutting cornerwise, as in the case of vertical joints of the single-layer assembly shown in Fig. 7; (b) is a section of a joint between the corner of a box and the edges of two abutting boxes, as in the case of joints between two layers of the multiple-layer assembly shown in Fig. 9; (c) is a section of a joint between three edges, as will result when the end faces of an assembly are closed with precast plaques; and (d) is a corner section between two edges. Fig. 12 illustrates similar joint details for boxes having two layers of mesh reinforcing. In addition, a suggested detail is indicated in (c) for retaining the grout in the joint. The arrangement, drawn in this case for a joint between four edges, consists of two form strips and anchor blocks which are secured in place by wire ties. As will be noted from the reinforcing details in Fig. 11 and 12, some of the mesh wires projecting from the corners or edges of the precast elements are lapped straight in the joint gap and others bent into the poured

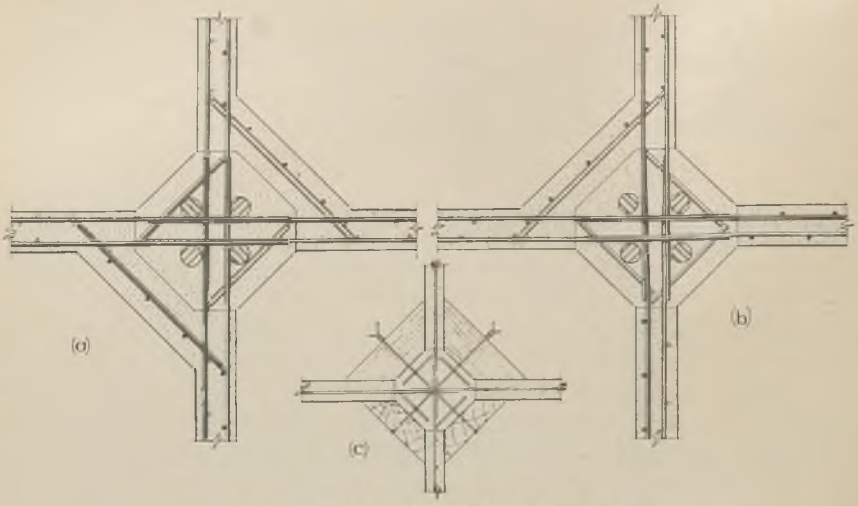


Fig. 12—Typical joint details of cells

fillets. The joint grooves also provide space for additional reinforcing bars.

Range of application

The precast cell constitutes an ideal framing element for floating structures, such as barges, gate and breakwater caissons, drydocks, tankers, floating piers, landing strips and many other water craft in the designs of which strength and lighter displacement are primary considerations. Results of comparative designs prepared for some of these structures indicate that the weight of a conventionally-poured job may be reduced as much as 60 percent by the use of precast cellular framing. Other advantages favoring precast cellular framing are the resulting economy, simplicity of erection and speed of construction. Still another advantage is the fact that no basin or special facilities of erection and launching are required. For example, in erecting a cellular barge such as shown in Fig. 13, a construction platform is all that is needed for the assembly of the cells in either arrangement. Opportunities of application are not, however, confined to floating structures only, since these elements can also be utilized to provide the framing of various shore buildings. Cellular framing is particularly adaptable to long-span floor construction as well as suitable for walls. Extensive studies will no doubt disclose possibilities of a wide range of applications.

CONCLUSION

In the huge construction program planned for the postwar era, concrete will undoubtedly play an important role. This role can, however,

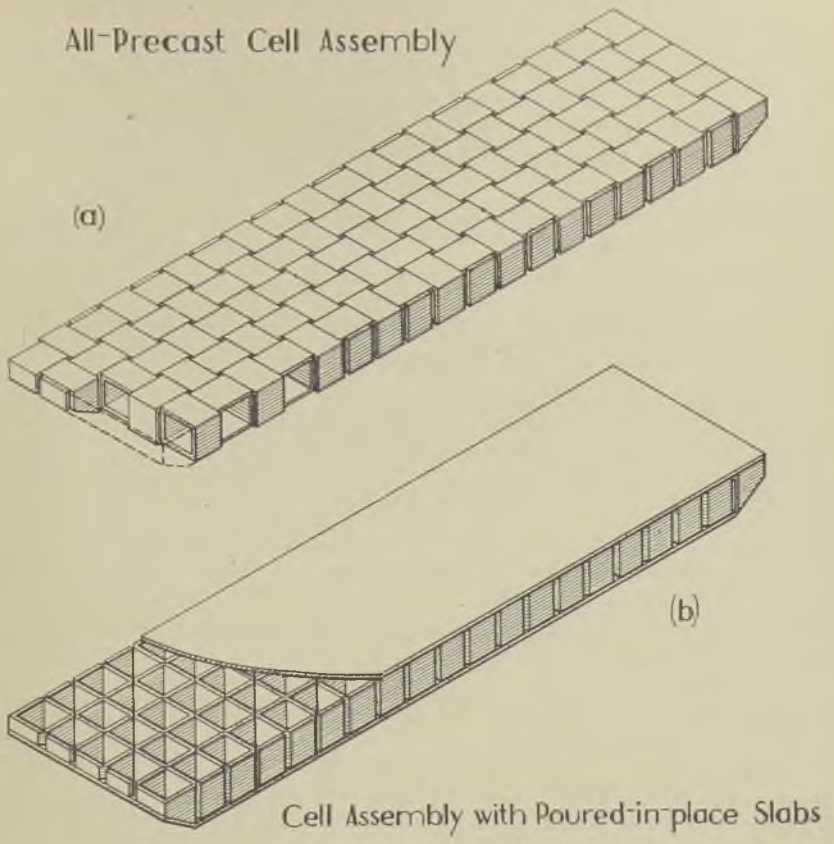


Fig. 13—Cellular assembly of a barge

be made more significant if precasting possibilities are fully explored and utilized. The scope of precasting is presented here in the interest of and as a challenge to the profession and the industry. Certain experimental projects of precast concrete construction, embracing both cellular and bent framing, have already been completed for the Navy. The results of these initial undertakings have been very encouraging, but a great deal of work still remains to be done for the improvement of various phases of precasting. There are many problems which need further investigation and experimentation in laboratories. A committee appointed by the Institute for this purpose could render valuable service in coordinating all such efforts.* Much of the success of precasting depends on the ingenuity and resourcefulness of the contractor, since the actual construction technique employed involves skilful planning, fabrication and handling.

*Since this paper was presented, ACI Committee 324, "Precast Reinforced Concrete Structures", has been organized with Mr. Amirikian as chairman.

[To facilitate selective distribution, **separate prints** of this title (43-14) 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 April 1, 1946]

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Vol. 18 No. 4 7400 SECOND BOULEVARD, DETROIT 2, MICHIGAN **December 1946**

Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars*

By ARTHUR P. CLARK†
Member American Concrete Institute

SYNOPSIS

The purpose of the tests described was to determine the resistance to slip in concrete of 17 different designs of deformed reinforcing bars.

The tests were of the pull-out type in which the bars were cast in a horizontal position; the depth of concrete under the bars and the length of embedment were varied. The slip of the bar was measured at the loaded and free end.

Three tests were made of each variable for each design of deformation.

It was established that a certain group of the bars was definitely superior to the others, in the sense that their average rating was significantly higher than the average of the others. Bars cast in the top position were much less effective than those cast in the bottom position.

INTRODUCTION

The tests of pull-out specimens reported herein constitute the first phase of a comprehensive investigation on the bond efficiency of deformed concrete reinforcing bars. The investigation was initiated by the Committee on Reinforced Concrete Research of the American Iron and Steel Institute, and was conducted through the medium of a Research Fellowship established at the National Bureau of Standards in Washington.

MATERIALS

Reinforcing bars

The bars, all nominally ⅞-in. diameter, are illustrated in Fig. 1a and 1b by two views of each bar to show the pattern of the deformations as produced by the upper and lower rolls and also the relation of the two.

*Received by the Institute, September 21, 1946.
†Research Associate, American Iron and Steel Institute, National Bureau of Standards, Washington, D. C.

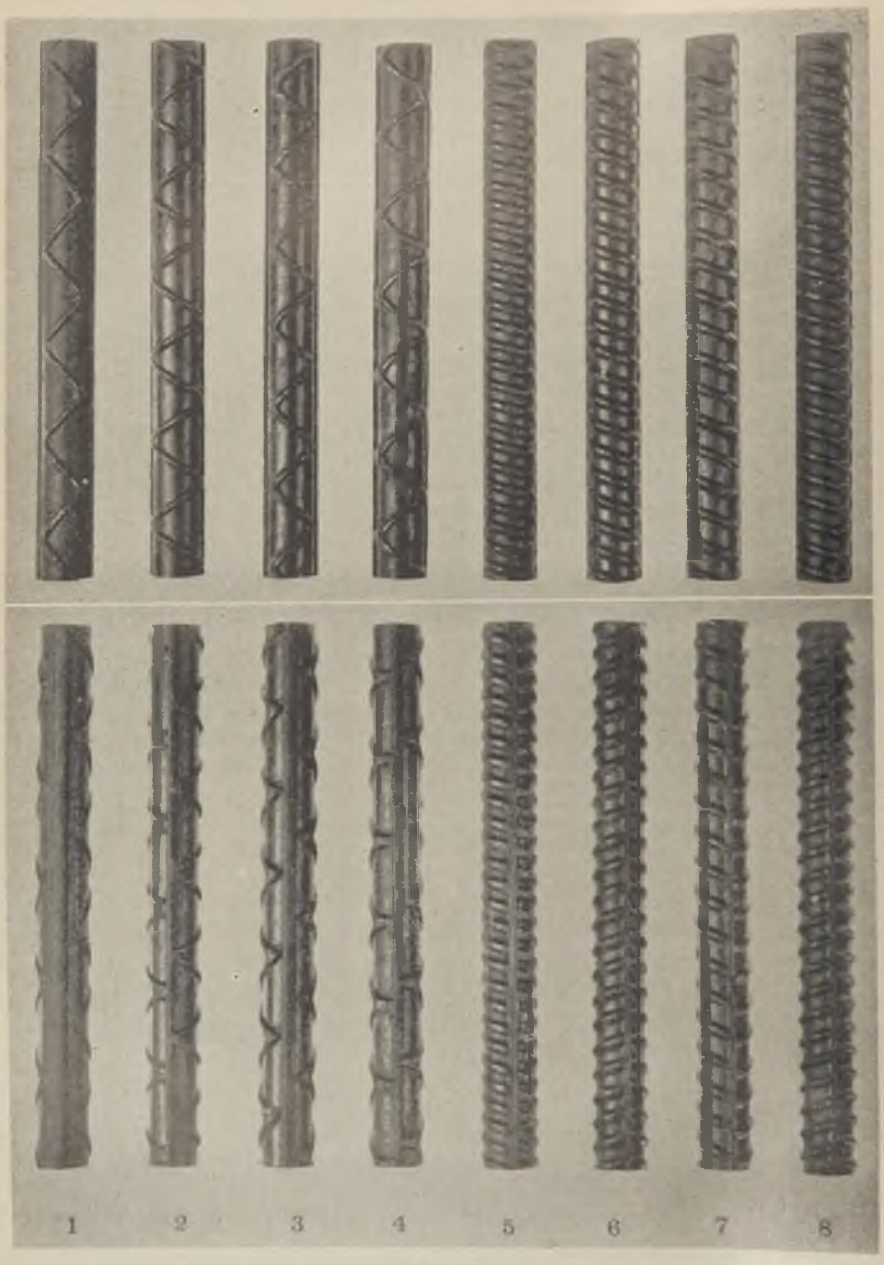


Fig. 1a

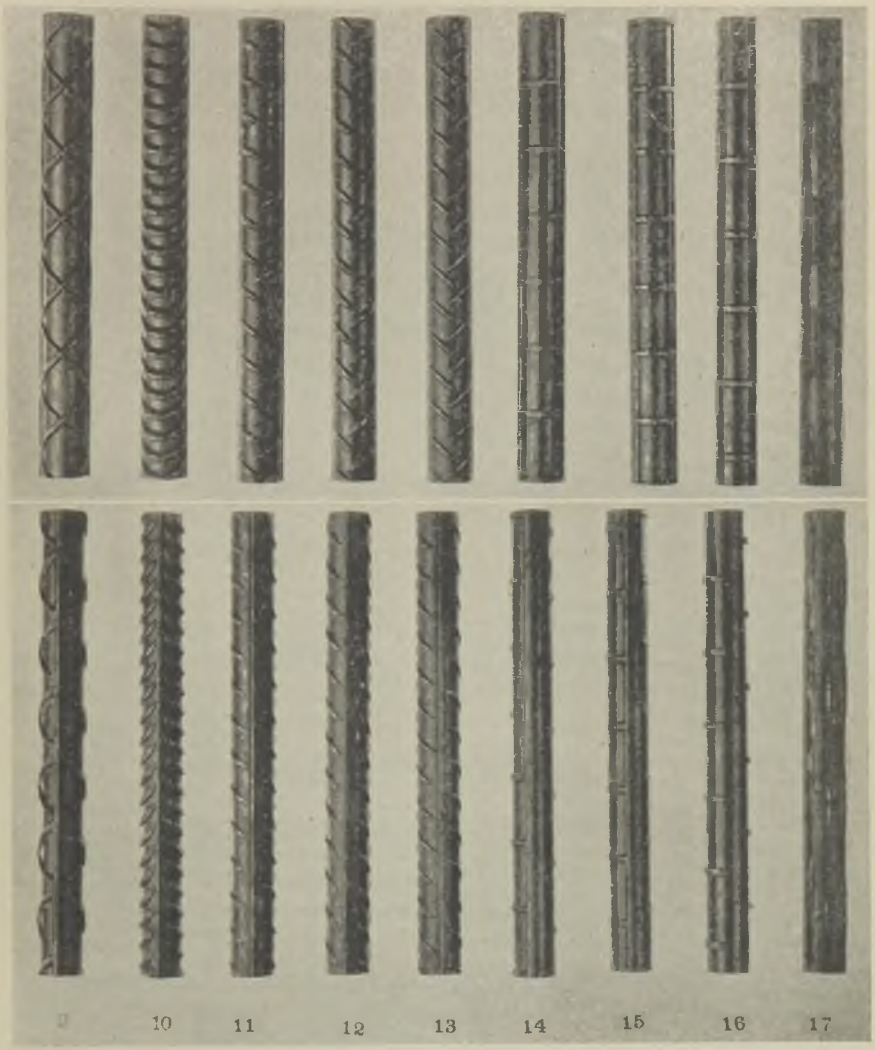


Fig. 1b

The yield point was determined in a 100,000-lb capacity testing machine of the beam and poise type by observing the drop of the beam. The height of the deformations was measured with a dial gage reading to 1/1000 in. The bar was held in a jig (Fig. 2) and readings taken at three or more points around the bar at each of several positions along the length of the bar. A total of 92 measurements of deformation heights were made by three observers to determine the average heights of the deformations of each bar. The bearing area of the

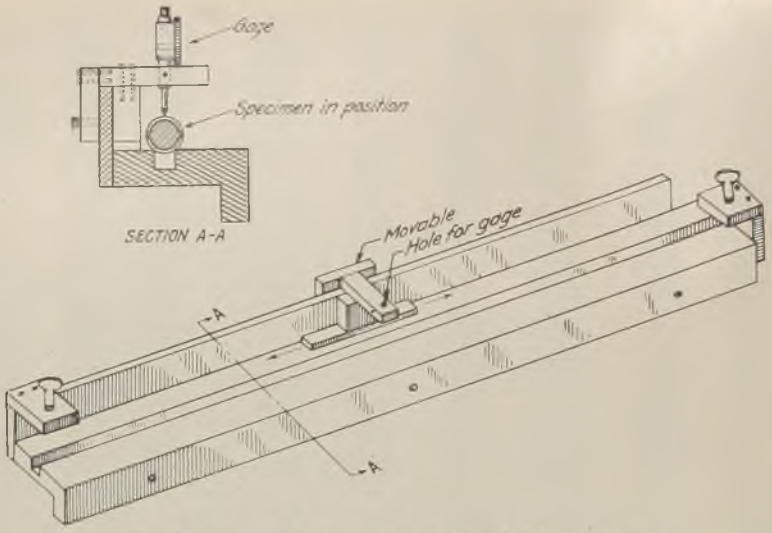


Fig. 2—Jig used in measuring height of deformations.

deformations per lin. in. of bar was determined from the average height, projected length and spacing. The two latter dimensions were measured from an impression of the deformations projected on a plane surface made by rolling the bar in a thin layer of quick-setting plaster.

The physical properties of the bars are given in Table 1.

TABLE 1—PHYSICAL PROPERTIES OF THE DEFORMED REINFORCING BARS

Bar No.	Weight per ft., lb.	Area,* sq. in.	Yield point		Tensile strength		Elongation in 8 in., in.	Deformations	
			lb.	psi	lb.	psi		Average height, in.	Bearing area, sq. in. per in.
1	2.059	0.61	26,890	44,100	47,430	77,700	2.125	0.045	0.082
2	1.966	.58	25,100	43,200	43,020	74,100	2.053	.046	.149
3	1.915	.56	22,970	41,000	34,490	63,300	2.719	.063	.134
4	2.011	.59	22,160	37,500	34,750	58,900	2.688	.067	.192
5	2.211	.65	24,870	38,300	45,790	70,400	2.094	.040	.322
6	2.042	.60	26,360	43,900	48,680	81,100	1.750	.055	.354
7	2.010	.59	27,110	45,900	47,870	81,100	1.719	.050	.257
8	2.058	.61	32,300	53,300	47,430	77,800	2.031	.055	.390
9	2.058	.61	25,050	41,100	42,870	70,300	2.000	.054	.224
10	2.030	.60	25,860	43,100	42,580	71,000	2.031	.050	.332
11	2.066	.61	33,880	55,500	60,020	98,400	1.500	.027	.101
12	2.049	.60	36,620	61,000	61,880	103,100	1.563	.028	.100
13	1.973	.58	36,780	63,400	62,090	107,100	1.375	.030	.103
14	2.020	.59	27,700	46,900	47,850	81,100	1.625	.067	.102
15	2.026	.60	27,640	46,100	45,220	75,400	2.031	.033	.052
16	2.006	.59	21,900	35,750	33,060	56,000	2.938	.033	.041
17	2.042	.60	----	----	77,240	128,700	0.906	----	----

*These areas and the calculations of yield points and tensile strengths therefrom are based on length and weight measurements in accordance with Sec. 10 of A.S.T.M. Designation A15-39. All calculations of stresses in the bars in pullout specimens are based on the nominal area, (0.60 sq. in.) of a ⅞-in. plain round bar.

Concrete

Concrete was machine-mixed and proportioned by weight in the ratio of 1:1.74:2.23. The water-cement ratio was 5.75 gal. per sack. Portland cement meeting the current standard specifications of the A.S.T.M. for Type I cement was used.

The coarse aggregate was Potomac River gravel ranging in size from No. 4 to 1 in. The fine aggregate was Potomac River sand graded as shown in Table 2. The dry rodded weight of the gravel was 106 lb. per cu. ft. and the sand 109 lb. per cu. ft.

TABLE 2—SIEVE ANALYSIS OF SAND

U. S. Standard Sieve No.	Percentage passing by weight
4	98
8	86
16	71
30	44
50	12
100	3

The slump of the concrete averaged 4¼ in. and the compressive strength of standard 6- by 12-in. cylinders, cured in moist air for 28 days, averaged 5,600 psi. The modulus of elasticity at 37 days was 4,140,000 psi.

DESCRIPTION OF TEST SPECIMENS

The pull-out specimens were 8 by 9 in. in cross section and of two lengths—8 in. and 16 in.; the bars in these specimens were held rigidly in the molds during casting with the longitudinal ribs in a horizontal plane. Two specimens were cast in a mold 18 in. in depth with one bar near the top and one near the bottom; the depth of concrete under the bar in the top position was 15 in., and 2 in. for the bar in the bottom position. Triangular steel strips welded horizontally to the interior sides of the molds at mid-height scored the concrete blocks to permit their subsequent separation into two specimens.

The molds were made of heavy steel sections fabricated to permit stripping of the specimens without damaging the concrete and the molds were sealed prior to casting to prevent leakage. The concrete was deposited in the mold in three layers and rodded in place.

The blocks were kept in moist storage in the molds after casting and at the end of three days were removed and divided into two specimens by loading as a beam in a testing machine, as illustrated in Fig. 3. All blocks broke in a plane where scored and the surface departed from a

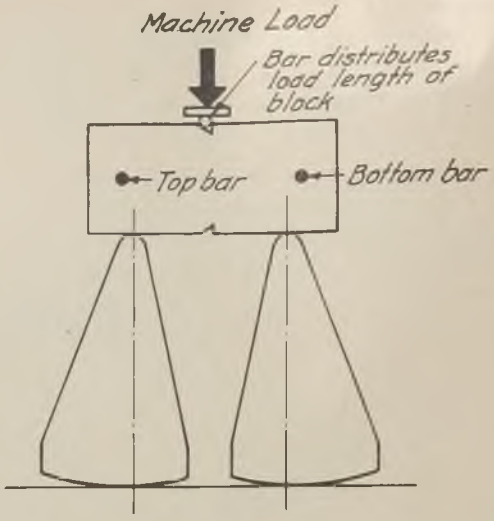


Fig. 3.—Method of breaking blocks to produce two pull-out specimens.

plane surface generally only to about the extent of the size of the coarse aggregate particles. The average breaking load was 1,660 lb. per in. of length of specimen. The specimens were then placed in a moist room until tested at the age of 29 to 31 days.

TESTING PROCEDURE

The pull-out specimens were tested in a 60,000-lb. capacity fluid-support, Bourdon-tube hydraulic machine and the load was applied at the rate of about 2,000 lb. per min., the dial gages being read without stopping the machine. The specimen was seated on a rubber cushion on two segments of a 2-in. base plate attached to the face of a spherical bearing block (Fig. 4). Slip of the bar was measured with 0.0001-in. micrometer dial gages and the readings were estimated to 0.00005 in. At the loaded end, two dials were held by a steel bar firmly attached to the bottom face of the specimen by bolts screwed into a series of nuts cast in the specimen. The dial gages were in contact with the smooth surface of a steel yoke fastened to the reinforcing bar by three set screws with cupped ends 1 in. below the surface of the concrete. The support bar for the dial gages and the yoke were free to move in a recess in the base plate. As load was applied, the average of the two dial gage readings indicated the amount of movement of the point on the reinforcing bar at which the yoke was attached with reference to the lower face of the concrete. To give the slip at the face of the concrete (slip at the loaded end), the dial gage readings were corrected for the elongation of the reinforcing bar in the 1-in. distance between the point of attachment of the yoke and the face of the concrete.

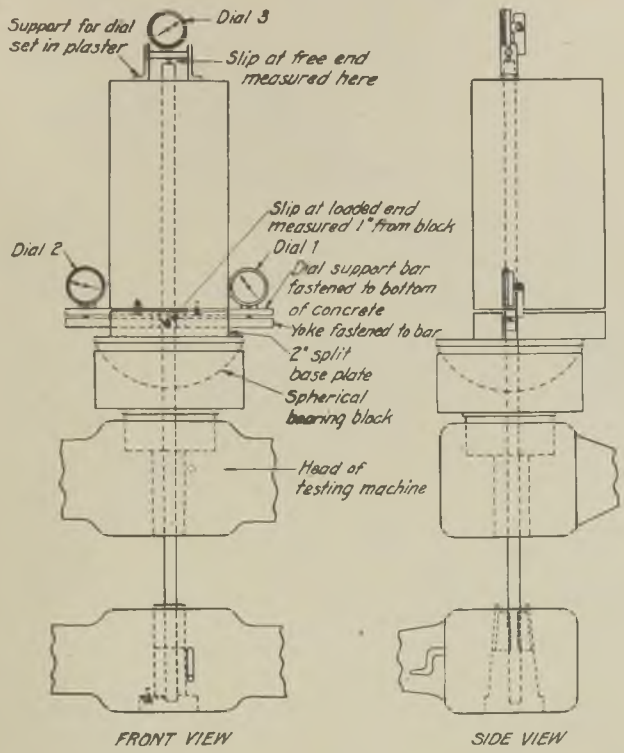


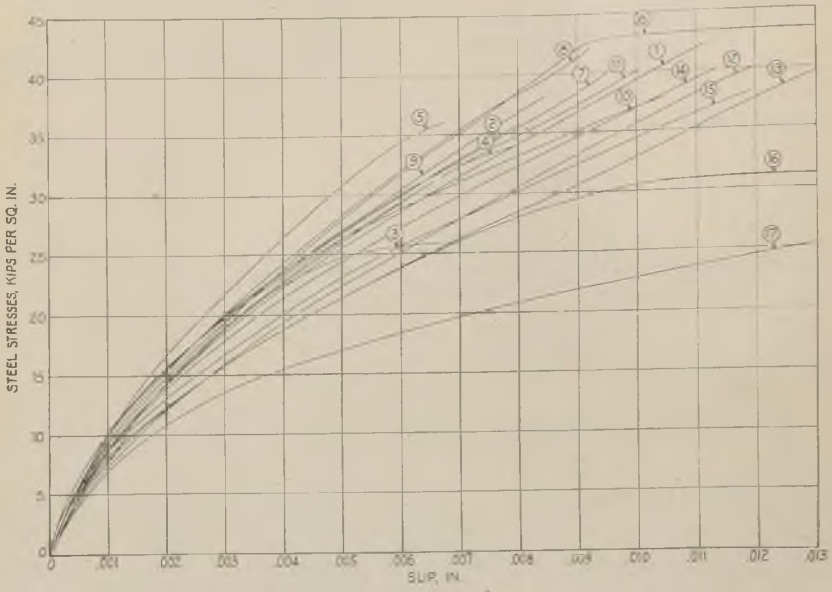
Fig. 4.—Specimen in testing machine.

The slip at the free end was read directly from the dial gage mounted on a frame, seated in plaster on the top of the specimen with the point of the dial resting on the planed end of the bar.

Initial dial gage readings were taken at a machine load of 300 lb. then 600 lb. and readings at increments of 600 lb. were taken until the machine registered 7,200 lb, and then at increments of about 1,200 lb. until failure. The dial gages at the loaded end were removed as the stress in the bar approached the yield point or when a loosening of the support bar or yoke, due to the stretch in the bar or initial cracking of the concrete, made the readings of questionable value. Readings at the free end were continued until just before failure.

RESULTS AND DISCUSSION

Tests in all cases were continued until failure of the specimen occurred, either by splitting of the concrete or by the bar pulling through the concrete. The typical splitting (79 percent of all specimens) was in a plane through the longitudinal ribs of the bar. Other specimens (13 percent) failed by splitting in a plane approximately at right angles



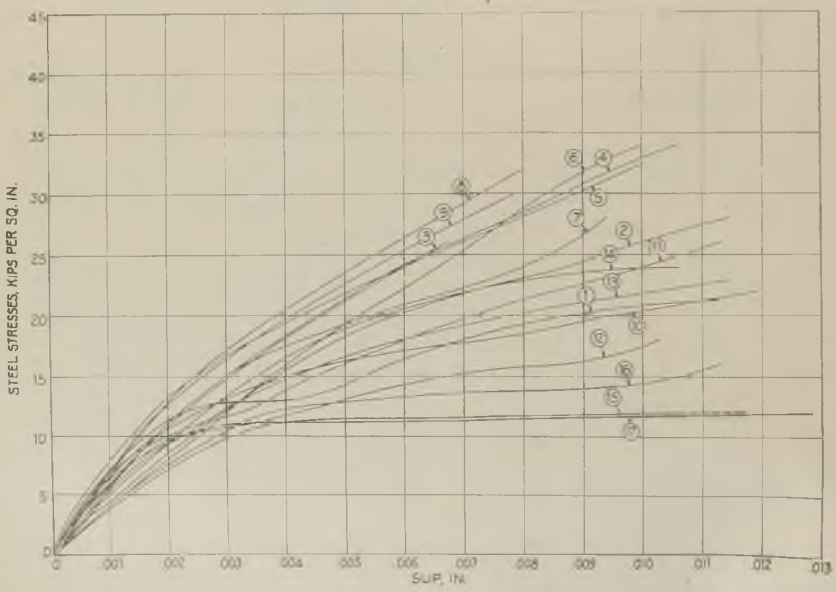
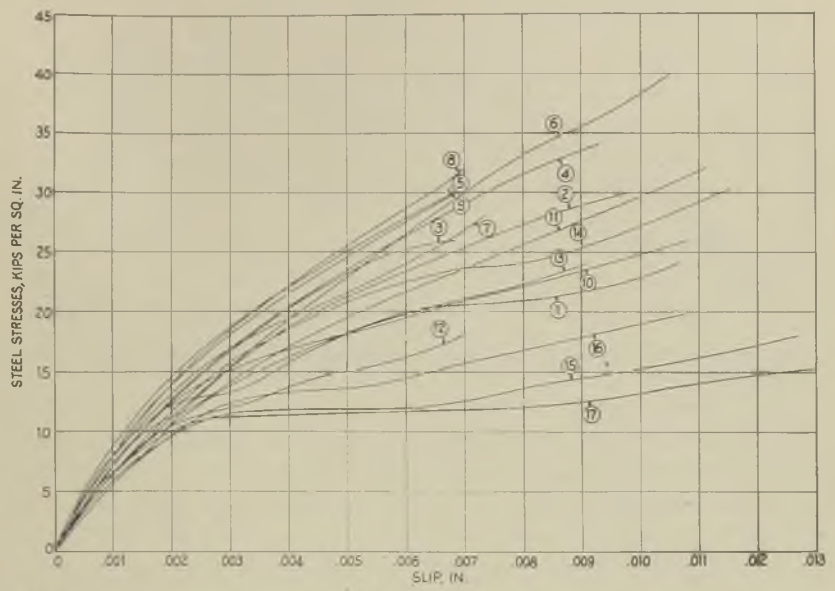


Fig. 5. (top)—Load-slip curves for bottom bars embedded 16 in. Slip measured at the loaded end.

Fig. 6 (bottom)—Load-slip curves for top bars embedded 16 in. Slip measured at the loaded end.



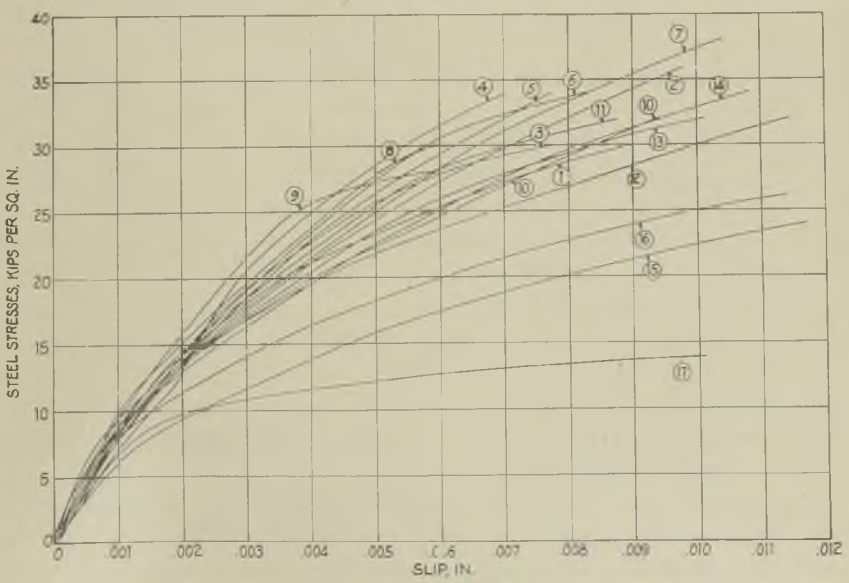


Fig. 7 (top)—Load-slip curves for the average of bottom and top bars embedded 16 in. Slip measured at the loaded end.

Fig. 8 (bottom)—Load-slip curves for bottom bars embedded 8 in. Slip measured at the loaded end.

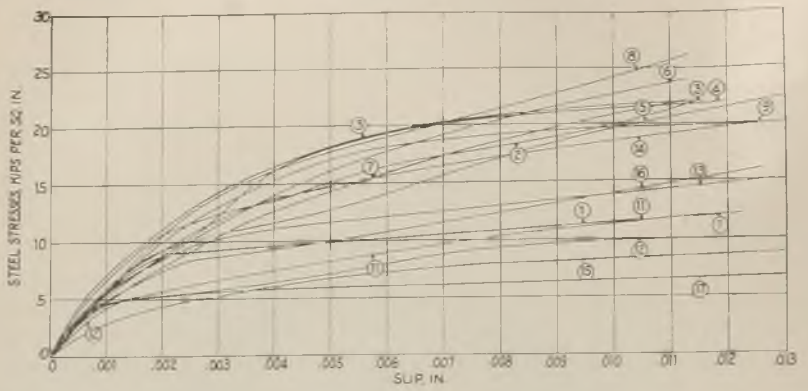


Fig. 9—Load-slip curves for top bars embedded 8 in. Slip measured at the loaded end.

to the plane through the longitudinal ribs. Seventeen specimens (8 percent) failed by the bar pulling through the concrete; these consisted of specimens containing bars 17 (with one exception), two each of 15 and 16 and one each of 11 and 13.

Twelve tests were made with each of the 17 bars with different patterns of deformations—three each with the bar in the top and bottom positions and with 8-in. and 16-in. embedments, the slip being measured at the loaded and free ends. All of the test results for bars of each pattern were combined to arrive at a grand average which thus represents the equivalent of 24 determinations of the stress-slip relationship.

The load-slip relation, as measured at the loaded end for both bottom and top bars and for the 8- and 16-in. embedments, is shown in Fig. 5, 6, 7, 8 and 9. Each curve represents, in general, the average data obtained from three specimens.

In the extensive literature dealing with the bond strength of reinforcing bars, the test bars have usually been rated by considering the stresses developed by the bars at some arbitrarily selected value of slip. In this investigation, an attempt was made to develop a method of rating bars which would take into account the stress-slip relationship for a considerable range of values of slip.

Load-slip curves of the bars of different designs show that some curves cross others. This means that one bar may have the highest stress for given slips along part of the curve, while the reverse may be true along another part of the curve.

Reinforced concrete has a wide variety of uses ranging from members where the prime consideration is to keep cracks as narrow as possible, such as structures intended to confine liquids and avoid leakage or

members exposed to attack by fumes or liquids which tend to hasten corrosion of steel, to those members where such conditions do not prevail and where deflection may not be a matter of particular concern. In the first case, a low value of design stress in the reinforcing steel is generally used, and in the other a high value. Between the extremes, other considerations may dictate the use of intermediate design stresses. A desirable concrete reinforcing bar would be one having a good bond resistance for all stresses within the range of probable use and up to the point of failure.

In this investigation, to determine the bars which could be expected to give the best bond resistance, it seemed logical to adopt a method for comparison which would rate the bars on the basis of average performance through a wide range of slip values. The method selected was to record the stress developed at several values of slip from the smallest at which readings were reliable up to the maximum slip developed by the majority of the bars. These stresses for each bar were totaled and the sum divided by the number of readings to obtain a figure indicative of the rating of the bar. These values may be used to indicate the bonding efficiency of the bars.

The value of slip at which the stresses were recorded are shown in the following table:

Embedment, in.	Position of bar	Measured at	Slip, in.
16	Bottom	Loaded end	0.0005, 0.001, 0.002, 0.003, 0.004, 0.005, 0.0075, and 0.01
16	Top	" "	" " " " " "
8	Bottom	" "	" " " " " "
8	Top	" "	" " " " " "
16	Bottom	Free end	0.00005 and 0.0001
16	Top	" "	0.00005, 0.0001, 0.0005 and 0.001
8	Bottom	" "	" " " " "
8	Top	" "	0.00005, 0.0001, 0.0005, 0..001 0.005, and 0.01

The writer is of the opinion that this method, which is original insofar as he knows, is a reasonable one and gives a better basis for comparing bond performance than other methods heretofore employed.

The diagram in Fig. 10 represents, for slip at the loaded end, the rating of the bars for 16-in. embedment. The ordinates "steel stress" are the ratings used. In the vertical column for each type of bar, the maximum height of the column indicates the rating of the bottom bars. The top of the vertical hatched portion indicates the rating for the top bars and the top of the diagonal hatched portion indicates the average rating for both top and bottom bars.

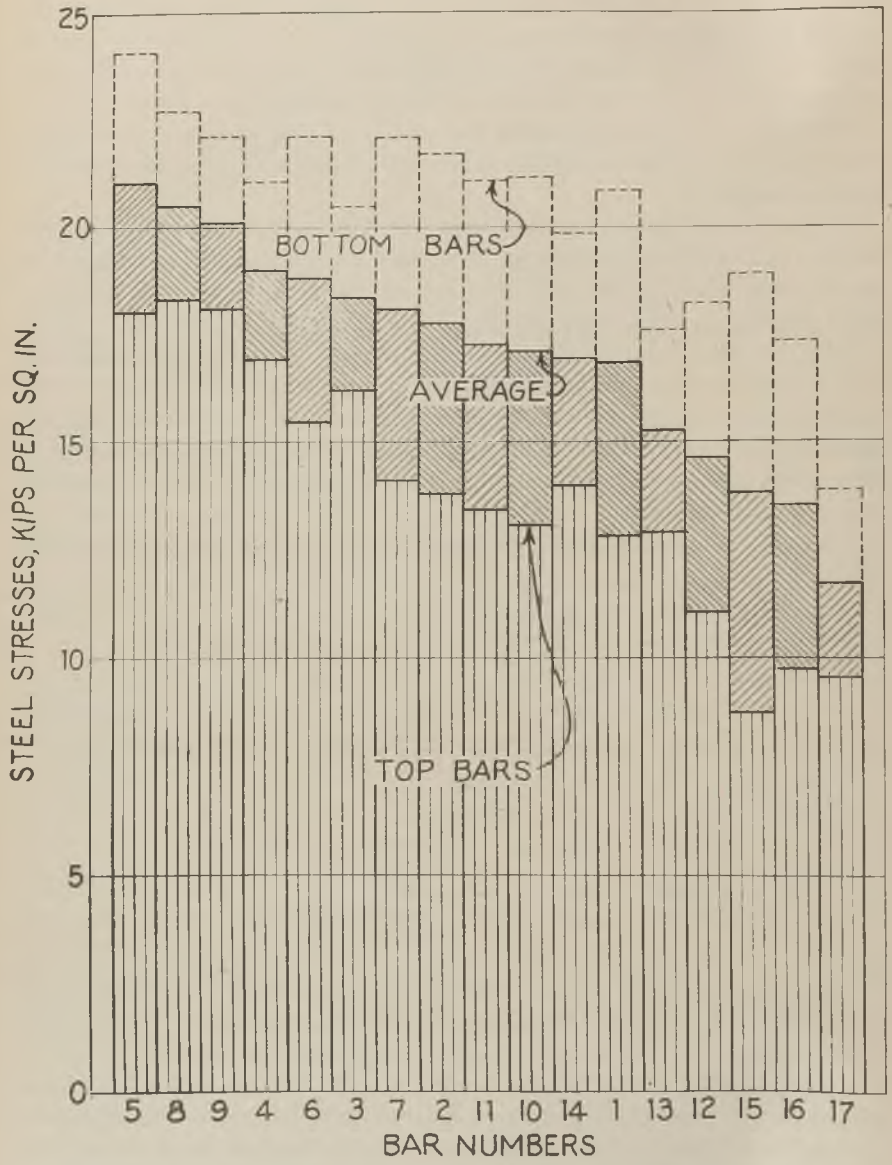


Fig. 10—Rating for each bar corresponding to slips of 0.0005, 0.001, 0.002, 0.003, 0.004, 0.005, 0.0075 and 0.01 in. at the loaded end for 16-in. embedment

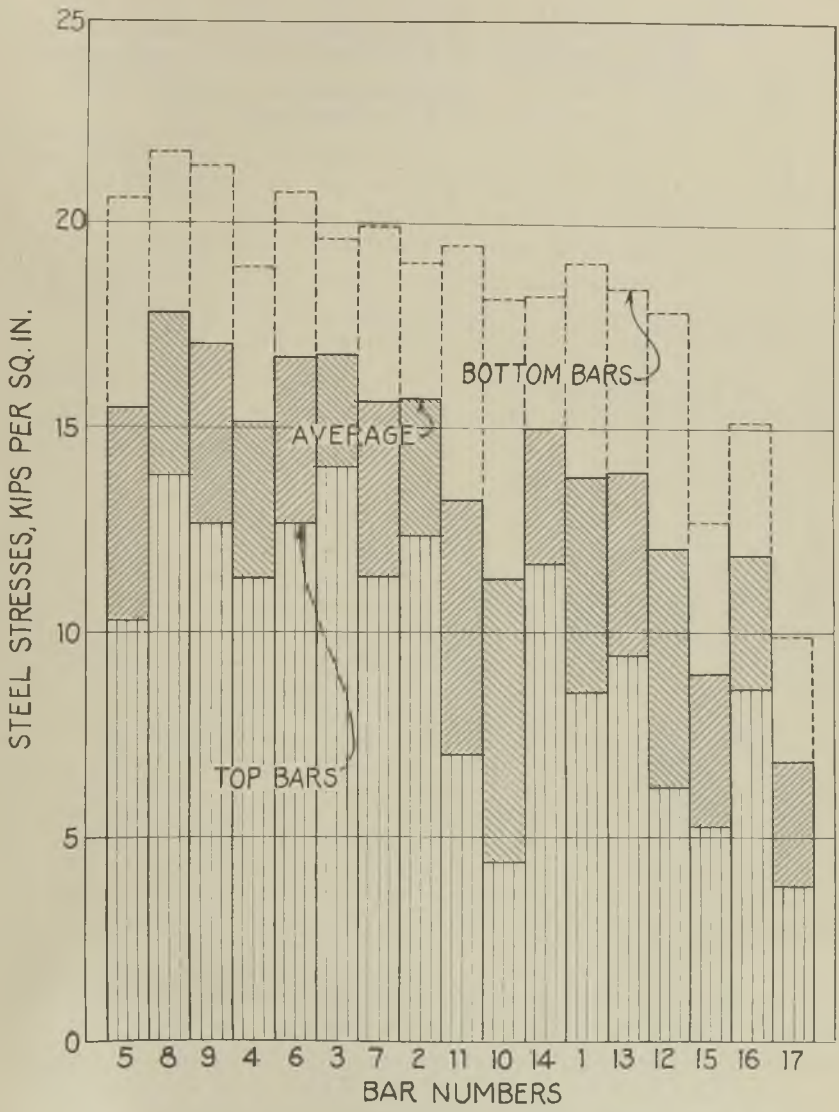


Fig. 11—Rating for each bar corresponding to slips of 0.0005, 0.001, 0.002, 0.003, 0.004, 0.005, 0.0075 and 0.01 in. at the loaded end for 8-in. embedment.

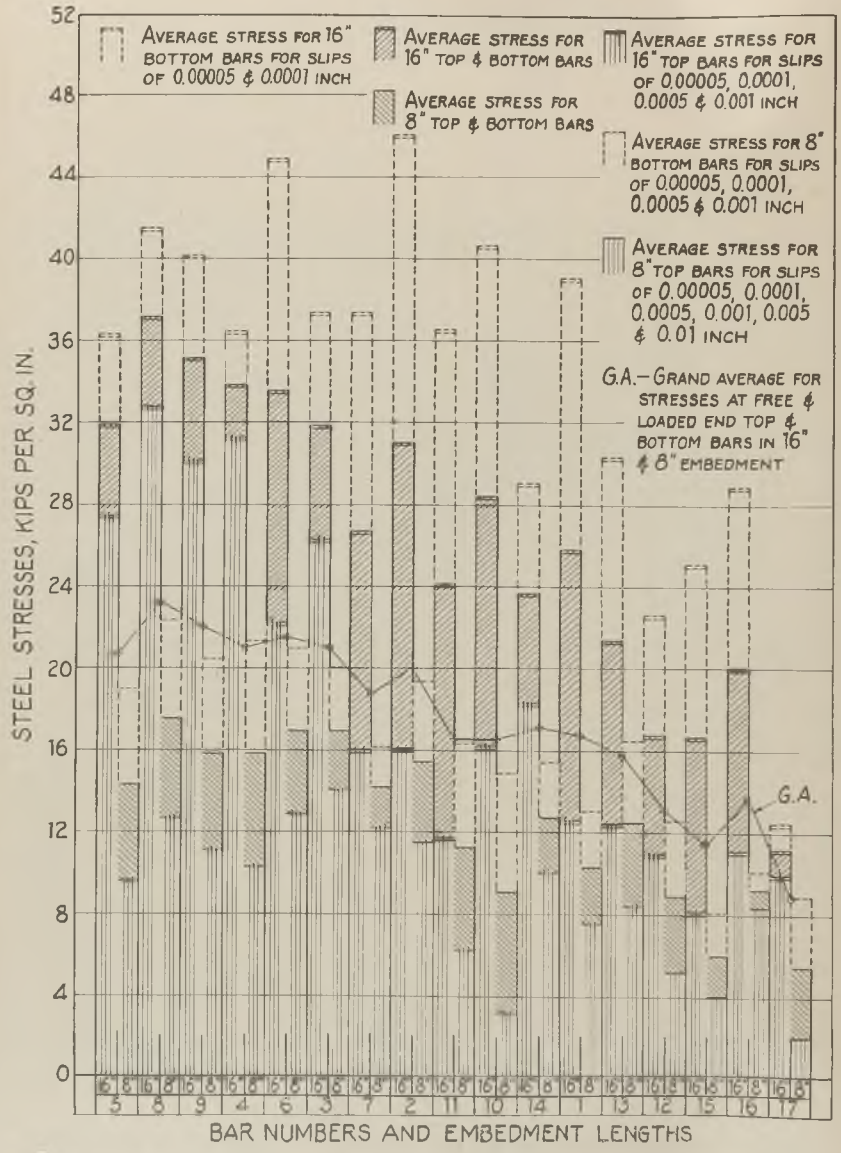


Fig. 12—Rating for each bar corresponding to slips, as given in the notes, at the free end for 16-in. and 8-in. embedments.

Fig. 11 represents for slip at the loaded end the rating of the bar for 8-in. embedment, in a manner similar to Fig. 10.

Fig. 12 represents the rating of the bars for slips at the free end. The ratings for 8-in. and 16-in. embedments are shown on one diagram in a manner similar to the diagrams for the loaded end. The solid line marked "G.A." represents the grand average of all the ratings for 8-in. embedment, 16-in. embedment, top bars and bottom bars for slips at both the loaded and the free ends.

In Fig. 10, 11 and 12, the order in which the bars were arranged corresponds to the sequence of the average ratings of the bars with 16-in. embedment. Bars 8, 9, 6, 3, 4 and 5 showed the highest grand average ratings, and in the order named the difference between the extreme values of the grand averages were only about 11 percent. The average of these six exceeds the average of the remaining bars by 31 percent.

Considering the slip at both ends and for both embedments, the rating for all bars in the top position was 56.8 percent of the rating for the bars in the bottom position. For bars 8, 9, 6, 3, 4 and 5, this ratio was 67.9 percent.

For all bars of the same type of design, 5, 6, 7 and 8, the ratio of the rating of the top bars to that of the bottom bars was 62.9 percent, and for all bars of the same general type, 1, 2, 3, 4 and 9, this ratio was 61.8 percent. The data from these tests indicate that, for bars cast in a horizontal position, the pattern of the deformations on the bar has little influence on the loss of bond strength due to settlement of the plastic concrete under the bar.

Although a detailed statistical analysis of the dispersion of the data was not made, an examination of the ranges of values of stress for slips greater than 0.003 in. at the loaded ends of bars indicated that a difference of less than 10 percent between the ordinates of any two of the curves of Fig. 5, 6, 8 and 9 is not positively significant. Similarly, differences of less than 1500 psi in the averages for the top and bottom bars shown in Fig. 10 and 11 or of less than 750 psi in the grand averages shown in Fig. 12 may not be significant.

Bond resistance is influenced by several factors, including the average height of the deformations, the bearing area of the deformations, the shearing area of the concrete between the deformations, the inclination of the bearing face of the deformations, and the angle the deformations make with the longitudinal axis of the bar. While in this series of tests, it was not possible to compare bars in which only one of these factors was a variable, the graphs in Fig. 13 show the relation of these factors to the grand average stress developed in the bars. As was to be expected, the presence of other factors obscured, to some extent, the

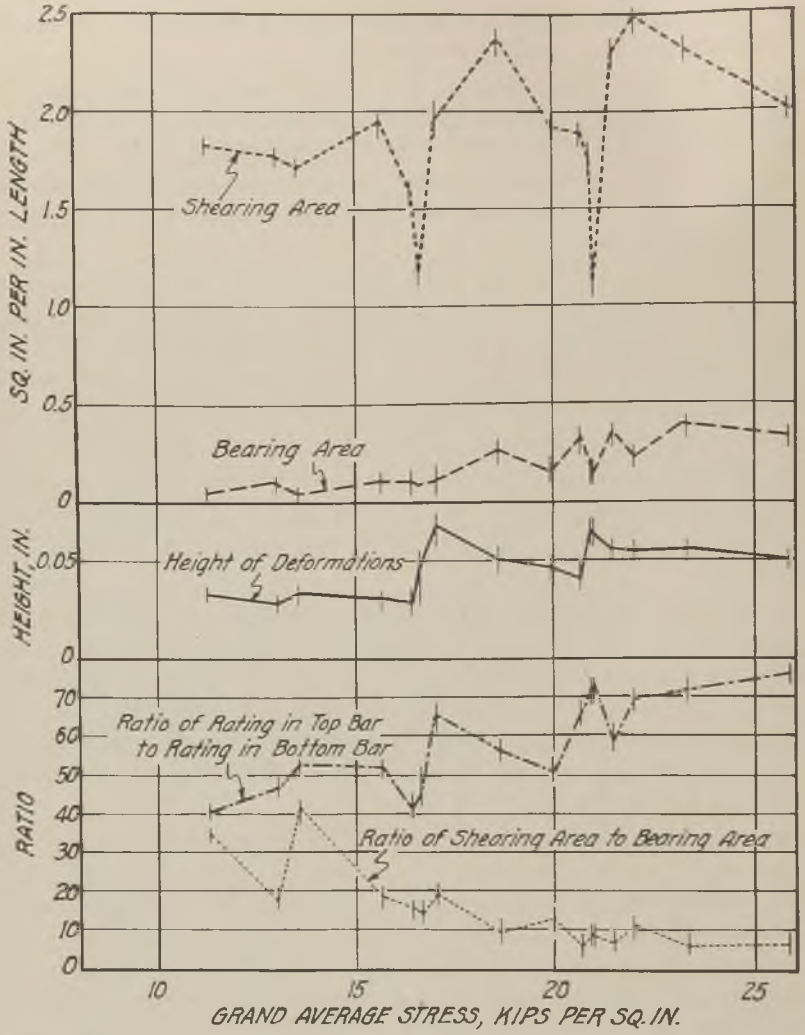


Fig. 13—Effect of height and bearing area of deformations and shearing area of concrete between deformations and the ratio of stress in top bars to the stress in bottom bars on the grand average stress in bars.

relation between the grand average stress and the variables shown in Fig. 13; nevertheless, the graphs illustrate an unmistakable trend in all cases except the graph for the shearing area.

It is apparant that, in general, the grand average stress increases with the bearing area of the deformations, their height, and the ratio of rating in the top bar to the rating in the bottom bar. It is of interest to note the similarity between the last two graphs named. The simi-

larity of these graphs indicates that the height of deformations has an influence on the loss of bond strength due to the settlement of the plastic concrete under the bars. The graph illustrating the effect of the ratio of shearing area to bearing area shows that the bars with low grand average stresses have an excessively high shearing area, and for bars with high grand average stresses this ratio is substantially constant between the values of 6 and 11.

The data from tests of bar 10 indicate that the inclination of the bearing face of the deformations is also a factor. Bar 10 has crescent-shaped deformations and these point in the same direction on both sides of the bar. One face of the deformation is substantially normal to the axis of the bar while the opposite face is inclined at an average angle of less than 30 degrees with the axis of the bar. Tests were made with some specimens in which the bar was pulled with the crescents pointing in the direction of pull and others in which the bar was pulled with the crescents pointing in the opposite direction. The bearing area, shearing area, height of deformation and the distance the deformations extend around the bar were, of course, the same in all the tests of bar 10. Therefore, the only variable was the inclination of the face of the deformations and, to some extent, the shape of the concrete surface which was subjected to bearing stress. The load-slip relation for slip at the loaded end is shown in Fig. 14. The solid curves in each case are the average of specimens of bar 10, in which the bar was pulled in the direction in which the crescents pointed and the bearing on the deformations was

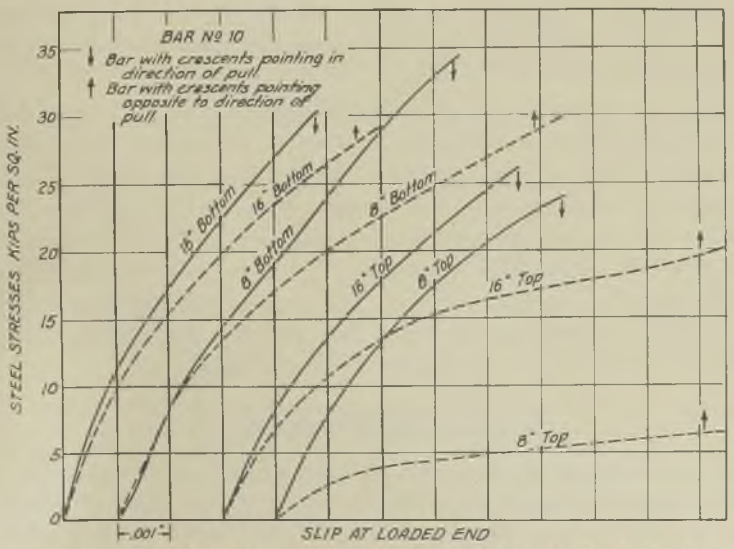


Fig. 14—Load-slip curves for bar 10 determined for the loaded end.

against the face normal to the direction of pull. The dotted curves represent the opposite conditions. For all cases—8-in. embedment, 16-in. embedment, bottom bars and top bars—considerably higher values were shown when the bearing on the deformations was against the face nearly normal to the surface of the bar.

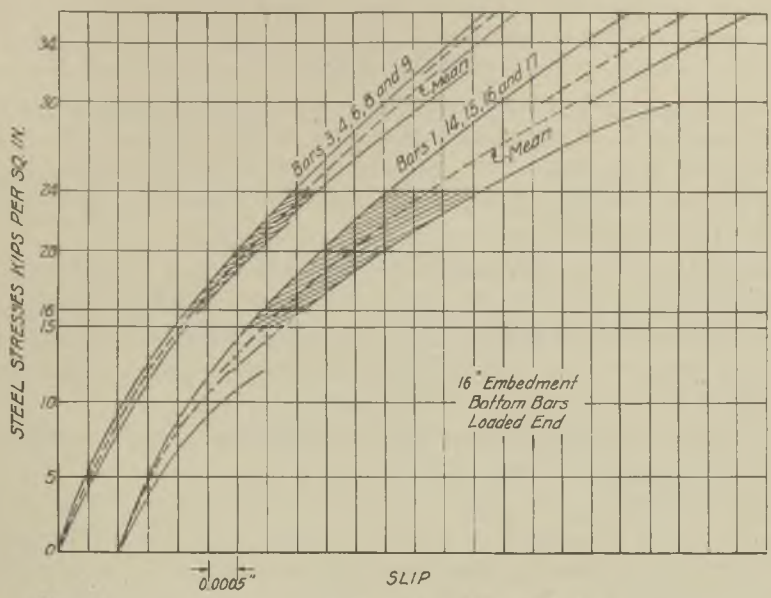
Five of the bars included in the tests (1, 14, 15, 16 and 17) have been in commercial production for a number of years and are representative of deformed bars which have been generally accepted and used. Comparing the average of the grand average stresses of these five bars with that of bars 8, 9, 6, 3, 4 and 5 indicates the increase in bond which might be expected by the use of bars of the better types. The percentage of increase depends on the basis used for comparison. Based on slip at the loaded end, the minimum increase (21.6 percent) was shown by the bottom bars with 16-in. embedment; the maximum (64.7 percent) by the top bars with 8-in. embedment. The average increase of all bars at the loaded end was 39.8 percent. Based on slip at the free end, the minimum (47.1 percent) was shown by bottom bars with 16-in. embedment; the maximum (137.8 percent) by top bars with 16-in. embedment, and the average of all bars at the free end was 78.7 percent. The grand overall average increase was 60.1 percent.

The load-slip relation at the loaded end for the five bars with the highest ratings in this experiment and the group of five commercial bars cast in the bottom position for 16-in. embedment is shown in Fig. 15 and and for 8-in. embedment in Fig. 16. The range of usual design stresses (16,000 to 24,000 lb. per sq. in.) is indicated by the cross-hatched portion. The five bars with the highest ratings show much less variation than the commercial bars.

The data for the two groups of bars presented in Fig. 15 and 16 represent the results obtained with bars tested in triplicate. In those cases where one specimen of a set of three failed prematurely either by splitting of the concrete or slipping of the bar, the data for that bar were discontinued at the maximum load representing all three specimens; these "breaks" occur in the lower enveloping curves where they are indicated by discontinuities in the curves. The upper enveloping curves also show "breaks" which are due to the fact that the load-slip curves of the several bars in a given group intersected each other.

SUMMARY OF RESULTS

The test specimens were of the pull-out type in which the bars were cast in a horizontal position with length of embedment and depth of concrete under the bar varied. Provision was made for measuring the slip of the bar at both the loaded end and the free end of the bar.



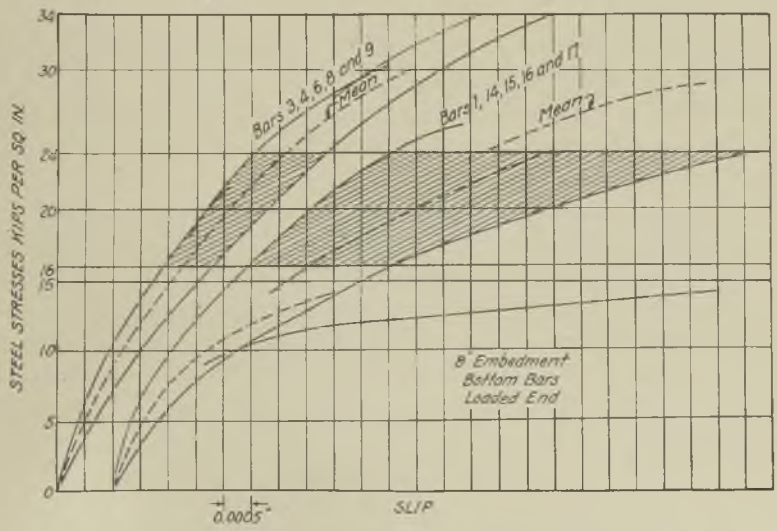


Fig. 15 (top)—Load-slip curves for the five commercial bars (1, 14, 15, 16 and 17) and the five best bars (3,4, 6, 8 and 9) in bottom position embedded 16 in., determined for the loaded end.

Fig. 16 (bottom)—Load-slip curves for the five commercial bars (1, 14, 15, 16 and 17) and the five best bars (3, 4, 6, 8 and 9) in bottom position embedded 8 in., determined for the loaded end.

The method used for evaluating the efficiency of the bars in bond resistance was based on the average performance for a range of slips as measured at the loaded end and at the free end for the bar with 2 in. and 15 in. of concrete under the bar and for 8-in. and 16-in. embedment.

The eight separate determinations for rating the bars gave, in general, consistent results. Considering the five bars rating highest in the grand average, one of the five rated highest in six of the determinations, one of the five rated second highest in all eight determinations, and one of the five rated third highest in all but one of the eight.

The average of the grand averages for bars 8, 9, 6, 3, 4 and 5 was approximately 60 percent higher than for the five commercial bars.

Bars 8, 9, 6, 3, 4 and 5 gave results which varied only about 11 percent from the highest. The five commercial bars gave results which varied by more than 41 percent from the highest.

In the top position, bars 8, 9, 6, 3, 4 and 5 were about two-thirds as effective in bond as in the bottom position.

Height of deformations appears as an important factor in determining the effect of the settlement of the concrete under the bar. The pattern of the deformation does not seem to be an important factor in determining the bond resistance.

The inclination of the face of the deformations is an important factor determining the bond resistance.

Attention is called to the fact that the conclusions arrived at herein do not necessarily apply when the conditions are different from those which prevailed in this investigation.

The outline and general procedure for this series was prepared by R. R. Zipprodt, Research Engineer for the Committee on Reinforced Concrete Research and acknowledgment is made of the very willing cooperation of all members of the Bureau staff, and in particular that of D. E. Parsons, Chief of the Division of Clay and Silicate Products, and David Watstein, Materials Engineer, for their helpfulness and continued interest.

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Proposed Revision of

Building Regulations for Reinforced Concrete (ACI 318-41)*

Reported by ACI Committee 318

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A. BURTON COHEN
ROLLAND P. CRAVENS
JOSEPH DI STASIO
ABRAHAM EPSTEIN
ROBERT O. JAMESON
VERNON P. JENSEN
N. D. MITCHELL
I. E. MORRIS
T. D. MYLREA
N. M. NEWMARK
JOHN R. NICHOLS
D. E. PARSONS
DEAN PEABODY, JR.
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HALE SUTHERLAND
W. H. WHEELER
C. A. WILLSON
R. B. YOUNG

EDITORIAL NOTE

In the following pages the current code appears in full in the larger type. The proposed revisions are shown in the smaller type in a narrower measure. Note the proposed new title is "Building Code Requirements for Reinforced Concrete". The report of Committee 318 with its proposed changes has been released by the Standards Committee for convention action. There were no dissenting votes among the membership of Committee 318 on a letter ballot of the Committee in which all members voted.

Although the closing date for written discussion is April 1, 1947, prospective discussors are urged to present their ideas to the convention. Those who cannot be present should have their manuscript (in triplicate) in the Secretary's office not later than February 15, 1947 to insure consideration at the convention.

Title—change to, "Building Code Requirements for Reinforced Concrete."

CHAPTER 1—GENERAL

101—

Line 1, change "These regulations cover"; to "This code covers."

Line 6, delete "specific."

Line 7 and 8, change "these regulations" to "this code."

*Adopted as a Standard of the American Concrete Institute at its 37th Annual Convention, February 20, 1941 as reported by Committee 318; Ratified by Letter Ballot July 21, 1941 (with editorial corrections from previous printings, in accordance with 'Errata" leaflet issued 1943.) The Committee acknowledges the active cooperation of the Committee on Engineering Practice of the Concrete Reinforcing Steel Institute.

101—Scope

(*a*) These regulations cover the use of reinforced concrete and plain concrete in any structure to be erected under the provisions of the building code of which they form a part. They are intended to supplement the general provisions of the code in order to provide for the proper design and construction of structures of these materials. In all matters pertaining to design and construction where these specific regulations are in conflict with other provisions of the code, these regulations shall govern.

102—Permits and drawings

(*a*) Drawings and typical details of all reinforced concrete construction showing the size and position of all structural members, metal reinforcement, design strength of concrete, and the live load used in the design shall be filed with the building department as a permanent record before a permit to construct such work will be issued. All plans submitted for approval or use on the work shall clearly show the strength of concrete at a specified age for which all parts of the structure were designed. Calculations pertaining to the design shall be filed with the drawings when required by the Commissioner of Buildings.

103—Special systems of reinforced concrete

(*a*) The sponsors of any system of reinforced concrete which has been in successful use, or the adequacy of which has been shown by test, and the design of which is either in conflict with, or not covered by these regulations shall have the right to present the data on which their design is based to a "Board of Examiners for Special Construction" appointed by the Commissioner of Buildings. This Board shall be composed of competent engineers, architects and builders, and shall have the authority to investigate the data so submitted and to formulate rules governing the design and construction of such systems. These rules when approved by the Commissioner of Buildings shall be of the same force and effect as the provisions of this code.

104—Definitions

(*a*) The following terms are defined for use in this code:

Delete "Aggregate"—term and definition; substitute: Aggregate, Fine—Natural sand, or sand prepared from stone, blast furnace slag, or gravel, or, subject to the approval of the Commissioner of Buildings, other inert materials having similar characteristics.

Aggregate, Coarse—Crushed stone, gravel, blast furnace slag, or other approved inert materials of similar characteristics, or combinations thereof having hard, strong, durable pieces, free from adherent coatings.

Aggregate—Inert material which is mixed with portland cement and water to produce concrete.

Column—An upright compression member the length of which exceeds three times its least lateral dimension.

Column Capital—Delete "upper" in first line.

Column Capital—An enlargement of the upper end of a reinforced concrete column designed and built to act as a unit with the column and flat slab.

Column Strip—A portion of a flat slab panel one-half panel in width consisting of the two adjacent quarter-panels on either side of the column center lines and extending through the panel in the direction of the span considered for bending.

Combination Column—Line 1, change "section" to "member."
Line 4, add "thereon" after "allowed."

Combination Column—A column in which a structural steel section, designed to carry the principal part of the load, is wrapped with wire and encased in concrete of such quality that some additional load may be allowed.

Composite Column—Change "section" to "structural member."

Composite Column—A column in which a steel or cast-iron section is completely encased in concrete containing spiral and longitudinal reinforcement.

Concrete—A mixture of portland cement, fine aggregate, coarse aggregate and water.

Deformed Bar—Reinforcing bars with closely spaced shoulders, lugs or projections formed integrally with the bar during rolling. Wire mesh with welded intersections not farther apart than twelve inches in the direction of the principal reinforcement and with cross wires not smaller than No. 10 W. & M. gage may be rated as a deformed bar.

Delete term and definition for Diagonal Band, Direct Band.

Diagonal Band—A group of reinforcing bars covering a width approximately 0.4 the average span, placed symmetrically with respect to the diagonal running from corner to corner of the panel of a flat slab.

Direct Band—A group of reinforcing bars, covering a width approximately 0.4 of l_1, placed symmetrically with respect to the center lines of the supporting columns of a flat slab.

Drop Panel—Line 2, after "column" add "or column."

Drop Panel—The structural portion of a flat slab which is thickened in the area surrounding the column capital.

Effective Area of Concrete—The area of a section which lies between the centroid of the tensile reinforcement and the compression face of the flexural member.

Effective Area of Reinforcement—The area obtained by multiplying the right cross-sectional area of the reinforcement by the cosine of the angle between its direction and the direction for which the effectiveness is to be determined.

Flat Slab—A concrete slab reinforced in two or more directions, generally without beams or girders to transfer the loads to supporting columns.

Middle Strip—A portion of a flat slab panel one-half panel in width, symmetrical about the panel center line and extending through the panel in the direction of the span considered for bending.

Paneled Ceiling—Omit term and definition.

Paneled Ceiling—A flat slab in which approximately that portion of the area enclosed within the intersection of the two middle strips is reduced in thickness.

Panel Length—The distance along a panel side from center to center of columns of a flat slab.

Pedestal—An upright compression member whose height does not exceed three times its least lateral dimension.

Plain Concrete—Concrete without reinforcement, or reinforced only for shrinkage or temperature changes.

Ratio of Reinforcement—Omit term and definition.

Ratio of Reinforcement—The ratio of the effective area of the reinforcement to the effective area of the concrete at any section of a flexural member.

Reinforced Concrete—Concrete in which reinforcement other than that provided for shrinkage or temperature changes is embedded in such a manner that the two materials act together in resisting forces.

Surface Water—The water carried by the aggregate except that held by absorption within the aggregate particles themselves.

Add new Section:

105—ASTM Specifications cited in this code

The specifications of the American Society for Testing Materials referred to in this code are listed below with their serial designation including the year of latest revision. They are declared to be a part of this code the same as if fully set forth elsewhere herein:

A7-42 Standard Specifications for Steel for Bridges and Buildings

A15-39 Standard Specifications for Billet-Steel Bars for Concrete Reinforcement

A16-35	Standard Specifications for Rail-Steel Bars for Concrete Reinforcement
A44-41	Standard Specifications for Cast Iron Pit-Cast Pipe for Water or Other Liquids
A82-34	Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement
A160-39	Standard Specifications for Axle-Steel Bars for Concrete Reinforcement
A185-37	Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement
C31-44	Standard Method of Making and Curing Concrete Compression and Flexure Test Specimens in the Field
C33-44	Standard Specifications for Concrete Aggregates
C39-44	Standard Method of Test for Compressive Strength of Molded Concrete Cylinders
C94-44	Standard Specifications for Ready-Mixed Concrete
C130-42	Standard Specifications for Lightweight Aggregates for Concrete
C150-44	Standard Specifications for Portland Cement
C192-44T	Tentative Method of Making Concrete Compression and Flexure Test Specimens in the Laboratory

CHAPTER 2—MATERIALS AND TESTS

200—

Line 1, after "test" add "relative to the ends of the span *L*."

Line 2, after "load test" add "(the shorter span of flat slabs and of floors supported on four sides)."

200—Notation

D = Deflection of a floor member under load test.

L = Span of member under load test.

t = The total thickness or depth of a member under load test.

201—

(a) Line 3, change, "when there is reasonable doubt as to" to "to determine."

Line 7, delete "the."

Line 8, delete "reasonable."

(b) Line 4 and 5, delete "by the Commissioner of Buildings at all times."

Line 5, after "work" add "and for two years thereafter".

Line 6, change, "two years after the completion of the structure"; to "that purpose."

201—Tests

(*a*) The Commissioner of Buildings, or his authorized representative, shall have the right to order the test of any material entering into concrete or reinforced concrete when there is reasonable doubt as to its suitability for the purpose; to order reasonable tests of the concrete from time to time to determine whether the materials and methods in use are such as to produce concrete of the necessary quality; and to order the test under load of any portion of a completed structure, when the conditions have been such as to leave reasonable doubt as to the adequacy of the structure to serve the purpose for which it is intended.

(*b*) Tests of materials and of concrete shall be made in accordance with the requirements of the American Society for Testing Materials as noted elsewhere in this chapter. The complete records of such tests shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work, and shall be preserved by the engineer or architect for two years after the completion of the structure.

202—

Line 3, change "one and one-half" to "two."

Line 8, after "or," delete the comma and add "else."

Line 9, change "shall" to "may."

Line 11, after "following:" delete colon; add "formula:"; change formula (1) to read:

$$D = \frac{L^2}{12{,}000\,t} \qquad (1)$$

Line 13, add "in which" before "all"; after "terms" add "are".

Par. 2, line 1, add "(b)" before "if."

Line 3, change "the member or portion" to end of paragraph to: ". . .the residual deflection does not exceed either forty percent of the maximum deflection observed under load or sixty percent of that given by formula (1). Under no circumstances will the construction be considered acceptable if the deflection under load exceeds three times that given by the formula."

202—Load tests

(*a*) When a load test is required, the member or portion of the structure under consideration shall be subject to a superimposed load equal to one and one-half times the live load plus one-half of the dead load. This load shall be left in position for a period of twenty-four hours before removal. If, during the test, or upon removal of the load, the member or portion of the structure shows evident failure, such changes or modifications as are necessary to make the structure adequate for the rated capacity shall be made; or, where lawful, a lower rating shall be established. The structure shall be considered to have passed the test if the maximum deflection at the end of the twenty-four hour period does not exceed the value of D as given in the following:

$$D = \frac{.001\,L^2}{12\,t} \qquad (1)$$

all terms expressed in the same units.

If the deflection exceeds the value of D as given in formula (1), the construction shall be considered to have passed the test if within twenty-four hours after the removal of the load the member or portion of the structure shows a recovery of at least seventy-five per cent of the observed deflection.

203—

Line 1, delete "All"; capitalize "c" in "concrete"; add "preferably" after

"supervised"; change "architect or engineer" to "the engineer or architect."

Line 3, change "the architect or engineer" to "him."

Line 5, delete second "and";

Line 6, after "reinforcing steel" add "and the general progress of the work"; change sentence "A . . . curing" to "When the temperature falls below 40 degrees F, a complete record of the temperatures and of the protection given to the concrete while curing shall be kept."

Line 9 and 10, delete "by the Commissioner of Buildings at all times."

Line 11, after "the work and" add "for two years thereafter and"; change architect or engineer" to "engineer or architect"; change "for two years after completion of the work" to "for that purpose."

203—Supervision

(*a*) All concrete work shall be supervised by the architect or engineer responsible for its design, or by a competent representative responsible to the architect or engineer. A record shall be kept of such supervision, which record shall cover the quality and quantity of concrete materials, the mixing and placing of the concrete, and the placing of the reinforcing steel. A complete record shall also be kept of the progress of the work and of the temperatures, when these fall below 40 degrees F., and of the protection given to the concrete while curing. This record shall be available for inspection by the Commissioner of Buildings at all times during the progress of the work and shall be preserved by the architect or engineer for two years after the completion of the work.

204—

Line 2, change all of section after "Portland Cement" to "(A.S.T.M. Designation: C150)."

204—Portland cement

(*a*) Portland cement shall conform to the "Standard Specifications for Portland Cement" (A.S.T.M. Serial Designation: C9-38) or the "Standard Specifications for High-Early-Strength Portland Cement" (A.S.T.M. Serial Designation: C74-39).

205—

(a) Line 2, delete "Serial" and "-40".

Line 3, change "provided however," to "or to the "Standard Specifications for Lightweight Aggregates for Concrete" (A.S.T.M. Designation: C130), except"; after "aggregates," add "failing to meet these specifications but"; after "by," add "special."

Lines 4 and 5, change "strength, durability, water-tightness, fire-resistance, and wearing qualities" to "quality."

205—Concrete aggregates

(*a*) Concrete aggregates shall conform to the "Standard Specifications for Concrete Aggregates" (A.S.T.M. Serial Designation: C33-40), provided however, that aggregates which have been shown by test or actual service to produce concrete of the required strength, durability, water-tightness, fire-resistance, and wearing qualities may be used under

Section 302(a) Method 2, where authorized by the Commissioner of Buildings.

(*b*) The maximum size of the aggregate shall be not larger than one-fifth of the narrowest dimension between sides of the forms of the member for which the concrete is to be used nor larger than three-fourths of the minimum clear spacing between reinforcing bars.

206—Water

(*a*) Water used in mixing concrete shall be clean, and free from injurious amounts of oils, acids, alkalis, organic materials, or other deleterious substances.

207—

(a) Line 1, change "Metal reinforcement" to "Reinforcing bars."
Line 3, delete "Serial" and "-39"; after "or" add "Standard Specifications for."
Line 4, delete "Serial" and "-35"
Line 5, after "or" add "Standard Specifications for"; delete "Serial."
Line 6, delete "-39".
(b) Line 4, delete "Serial" and "-34".
Line 5, delete "Serial".
Line 6, delete "-37."
(c) Line 2, delete "Structural."
Line 3, delete "Serial" and "-39."
(d) Line 2, change "Tentative" to "Standard."
Line 3, delete "Serial" and "-39T."

207—Metal reinforcement

(*a*) Metal reinforcement shall conform to the requirements of the "Standard Specifications for Billet-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A15-39), or for "Rail-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A16-35), or for "Axle-Steel Bars for Concrete Reinforcement" (A. S. T. M. Serial Designation: A160-39).

(*b*) Cold-drawn wire or welded wire fabric for concrete reinforcement shall conform to the requirements of the "Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (A. S. T. M. Serial Designation: A82-34), or "Standard Specifications for Welded Steel Wire Fabric for Concrete Reinforcement" (A. S. T. M. Serial Designation: A185-37).

(*c*) Structural steel shall conform to the requirements of the "Standard Specifications for Structural Steel for Bridges and Buildings" (A. S. T. M. Serial Designation: A7-39).

(*d*) Cast-iron sections for composite columns shall conform to the "Tentative Specifications for Cast Iron Pit-cast Pipe for Water and Other Liquids" (A. S. T. M. Serial Designation: A44-39T).

208—Storage of materials

(*a*) Cement and aggregates shall be stored in such a manner as to prevent deterioration or intrusion of foreign matter. Any material which has deteriorated or which has been damaged shall not be used for concrete.

Chapter 3—Title, change "Working" to "Allowable".

CHAPTER 3—CONCRETE QUALITY AND WORKING STRESSES

300—

Line 2, delete, "Ultimate"; capitalize "c" in "compressive."
Line 4, change "Allowable" to "Compressive."
Line 6, change "working" to "allowable".

300—Notation

f_c = Compressive unit stress in extreme fiber of concrete in flexure.
f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.
f_r = Allowable unit stress in the metal core of a composite column.
f_s = Tensile unit stress in longitudinal reinforcement; nominal working stress in vertical column reinforcement.
f_v = Tensile unit stress in web reinforcement.
n = Ratio of modulus of elasticity of steel to that of concrete.
u = Bond stress per unit of surface area of bar.
v = Shearing unit stress.
v_c = Shearing unit stress permitted on the concrete.

301—

(a) Line 2, change "working" to "allowable".
Line 3, delete "ultimate".
Line 4, delete "ultimate".
(b) Line 1, change "All" to "No"; change "the" before "weather" to "freezing."
Line 2, change "of not to exceed" to "exceeding"; delete hyphen in "water-content"; delete asterisk and footnote to which it refers.

301—Concrete quality

(*a*) For the design of reinforced concrete structures, the value of f'_c used for determining the working stresses as stipulated in Section 305 shall be based on the specified minimum ultimate 28-day compressive strength of the concrete, or on the specified minimum ultimate compressive strength at the earlier age at which the concrete may be expected to receive its full load. All plans, submitted for approval or used on the job, shall clearly show the assumed strength of concrete at a specified age for which all parts of the structure were designed.

(*b*) All concrete exposed to the action of the weather shall have a water-content of not to exceed six gallons per sack of cement.*

*In climates where frost action is not severe this section should be omitted.

302—

Change title to "Methods for determining strength of concrete."

(a) Method 1, line 1, delete hyphen in "water-content" also in left column heading Table 302(a) and footnote.

Method 2, line 1, delete hyphen in "water-content".

Lines 4 and 5, change "of Making Compression Tests of Concrete" to "of Test for Compressive Strength of Moulded Concrete Cylinders"; delete "Serial "and "-39."

Lines 6 and 11, delete hyphen in "water-content".

302—Determination of strength-quality of materials

(*a*) The determination of the proportions of cement, aggregate and water to attain the required strengths shall be made by one of the following methods:

Method 1—Concrete made from average materials:

When no preliminary tests of the materials to be used are made, the water-content per sack of cement shall not exceed the values in Table 302(*a*). Method 2 shall be employed when artificial aggregates or admixtures are used.

TABLE 302(a)—ASSUMED STRENGTH OF CONCRETE MIXTURES

Water-Content U. S. Gallons Per 94-lb. Sack of Cement	Assumed Compressive Strength at 28 Days—p.s.i.
7½	2000
6¾	2500
6	3000
5	3750

NOTE—In interpreting this table, surface water carried by the aggregate must be included as part of the mixing water in computing the water-content.

Method 2—Controlled Concrete:

Water-content other than shown in Table 302(*a*) may be used provided that the strength-quality of the concrete proposed for use in the structure shall be established by tests which shall be made in advance of the beginning of operations, using the consistencies suitable for the work and in accordance with the "Standard Method of Making Compression Tests of Concrete" (A.S.T.M. Serial Designation: C39-39). A curve representing the relation between the water-content and the average 28-day compressive strength or earlier strength at which the concrete is to receive its full working load, shall be established for a range of values including all the compressive strengths called for on the plans.

The curve shall be established by at least three points, each point representing average values from at least four test specimens. The maximum allowable water-content for the concrete for the structure shall be as determined from this curve and shall correspond to a strength which is fifteen percent greater than that called for on the plans. No substitutions shall be made in the materials used on the work without additional tests in accordance herewith to show that the quality of the concrete is satisfactory.

303—

Change title to "Concrete Proportions and Consistency"; change text to that of present Section 304 (a) and (b).

(*a*) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of

the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than thirty percent nor greater than fifty percent of the total, except that these proportions do not necessarily apply to light-weight aggregates.

(*b*) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work.* Measurement of materials for ready mixed concrete shall conform to the "Standard Specifications for Ready-Mixed Concrete" (A. S. T. M. Designation: C94).

303—Tests on concrete

(*a*) The Commissioner of Buildings shall require a reasonable number of compression tests to be made during the progress of the work. Such tests shall be made in accordance with the "Standard Method of Making and Storing Compression Test Specimens of Concrete in the Field" (A. S. T. M. Serial Designation C31-39), and cured in accordance with the requirements for laboratory control tests.

(*b*) Not less than three specimens shall be made for each test; nor less than one test for each 250 cu. yd. of concrete.

(*c*) The standard age of test shall be 28 days, but 7-day tests may be used provided that the relation between the 7- and 28-day strengths of the concrete is established by test for the materials and proportions used.

(*d*) Where the average strength of the laboratory control cylinders for any portion of the structure falls below the minimum ultimate compressive strengths called for on the plans, the Commissioner of Buildings shall have the right to order a change in the mixture or in the water content for the remaining portion of the structure. In cases where the average strength of the cylinders cured on the job falls below the required strength, the Commissioner of Buildings shall have the right to require conditions of temperature and moisture necessary to secure the required strength. If the average strength of either the laboratory control cylinders or the cylinders cured on the job falls below the required strength, load tests as specified in Section 202 may be required on the portion of the structure so affected.

304—

Change title to "Tests on concrete".

Change text to the following:

(*a*) The Commissioner of Buildings may require a reasonable number of tests to be made during the progress of the work. Not less than three specimens shall be made for each test, nor less than one test for each 250 cu. yd. of concrete. Specimens shall be made and cured in accordance with the "Tentative Method

*Wherever practicable such measurement shall be by weight rather than by volume.

of Making Concrete Compression and Flexure Test Specimens in the Laboratory" (A.S.T.M. Designation: C192) or, if in the opinion of the Commissioner of Buildings there is a possibility of the air temperature falling below 40 F., he may require the specimens to be made in accordance with the "Standard Method of Making Concrete Compression and Flexure Test Specimens in the Field (A.S.T.M. Designation: C31).

(*b*) The standard age of test shall be 28 days, but 7-day tests may be used provided that the relation between the 7- and 28-day strengths of the concrete is established by tests for the materials and proportions used.

(*c*) If the average strength of the laboratory control cylinders for any portion of the structure falls below the compressive strength required by the design, the Commissioner of Buildings shall have the right to order a change in the proportions or the water content of the concrete for the remaining portions of the structure. If the average strength of the cylinders cured on the job falls below the required strength, the Commissioner of Buildings shall have the right to require changes in the conditions of temperature and moisture necessary to secure the required strength.

(*d*) In addition, where there is question as to the quality of the concrete in the structure, the Commissioner of Buildings may require tests in accordance with the "Standard Methods of Securing, Preparing and Testing Specimens of Hardened Concrete for Compressive and Flexural Strengths", (A.S.T.M. Designation C42) or order load tests as outlined in Section 202 for that portion of the structure where the questionable concrete has been placed.

304—Concrete proportions and consistency

(*a*) The proportions of aggregate to cement for any concrete shall be such as to produce a mixture which will work readily into the corners and angles of the forms and around reinforcement with the method of placing employed on the work, but without permitting the materials to segregate or excess free water to collect on the surface. The combined aggregates shall be of such composition of sizes that when separated on the No. 4 standard sieve, the weight passing the sieve (fine aggregate) shall not be less than thirty percent nor greater than fifty percent of the total, except that these proportions do not necessarily apply to lightweight aggregates.

(*b*) The methods of measuring concrete materials shall be such that the proportions can be accurately controlled and easily checked at any time during the work.* Measurement of materials for ready mixed concrete shall conform to the "Standard Specifications for Ready-Mixed Concrete" (A. S. T. M. Serial Designation: C94-38).

305—

(a) Line 3, delete "ultimate".

Table 305(a), Column 3 head, delete "as Fixed by Test"; Column 4, 5, 6 and 7 top head, change "When Strength of Concrete is Fixed by the Water-Content in Accordance with Section 302" to "For Strength of Concrete Shown Below".

Column 1 under Flexure: f_c, add new line "Extreme fiber stress in tension in plain concrete footings . . . | f_c | $0.03f'_c$ | 60 | 75 | 90 | 113".

*Wherever practicable such measurements shall be by weight rather than by volume.

After Shear: v, add "(as a measure of diagonal tension)"; before Bond: u, delete "‡" and footnote to which it refers.

Change present text of table to:

Bond: u						
In beams and slabs:						
Plain Bars	u	$0.04f'_c$ but not to exceed 160 p.s.i.	80	100	120	150
Deformed Bars	u	$0.05f'_c$ but not to exceed 200 p.s.i.	100	125	150	188
In beams and slabs and one-way footings:						
Plain bars (hooked)	u	$0.06f'_c$ but not to exceed 200 p.s.i.	120	150	180	200
Deformed bars (hooked)	u	$0.075f'_c$ but not to exceed 250 p.s.i.	150	188	225	250
In two-way footings:						
Plain bars (hooked)	u	$0.045f'_c$ but not to exceed 160 p.s.i.	90	113	135	160
Deformed bars (hooked)	u	$0.056f'_c$ but not to exceed 200 p.s.i.	112	140	168	200

Under bearing: f_c, line 2, change f_e to f_c.

305—Allowable unit stresses in concrete

(*a*) The unit stresses in pounds per square inch on concrete to be used in the design shall not exceed the values of Table 305(a) where f'_c equals the minimum specified ultimate compressive strength at 28 days, or at the earlier age at which the concrete may be expected to receive its full load.

306—

Line 1, change "these Regulations" to "this Code".

(c) Line 1, change "working" to "allowable".

306—Allowable unit stresses in reinforcement

Unless otherwise provided in these Regulations, steel for concrete reinforcement shall not be stressed in excess of the following limits:

(*a*) *Tension*

(f_s = Tensile unit stress in longitudinal reinforcement)

and (f_v = Tensile unit stress in web reinforcement)

20,000 p.s.i. for Rail-Steel Concrete Reinforcement Bars, Billet-Steel Concrete Reinforcement Bars (of intermediate and hard grades), Axle-Steel Concrete Reinforcement Bars (of intermediate and hard grades), and Cold-Drawn Steel Wire for Concrete Reinforcement.

TABLE 305(a)—ALLOWABLE UNIT STRESSES IN CONCRETE

Description		Allowable Unit Stresses: For Any Strength of Concrete as Fixed by Test in Accordance with Section 302 $n = \frac{30000}{f'_c}$	When Strength of Concrete is Fixed by the Water-Content in Accordance with Section 302: f'_c = 2000 p.s.i. $n = 15$	f'_c = 2500 p.s.i. $n = 12$	f'_c = 3000 p.s.i. $n = 10$	f'_c = 3750 p.s.i. $n = 8$
Flexure: f_c						
Extreme fiber stress in compression	f_c	$0.45f'_c$	900	1125	1350	1688
Shear: v						
Beams with no web reinforcement and without special anchorage of longitudinal steel	v_c	$0.02f'_c$	40	50	60	75
Beams with no web reinforcement but with special anchorage of longitudinal steel	v_c	$0.03f'_c$	60	75	90	113
Beams with properly designed web reinforcement but without special anchorage of longitudinal steel	v	$0.06f'_c$	120	150	180	225
Beams with properly designed web reinforcement and with special anchorage of longitudinal steel	v	$0.12f'_c$	240	300	360	450
*Flat slabs at distance d from edge of column capital or drop panel	v_c	$0.03f'_c$	60	75	90	113
**Footings	v_c	$0.03f'_c$ but not to exceed 75 p.s.i.	60	75	75	75
‡Bond: u						
In beams and slabs and one-way footings:						
Plain bars	u	$0.04f'_c$ but not to exceed 160 p.s.i.	80	100	120	150
Deformed bars	u	$0.05f'_c$ but not to exceed 200 p.s.i.	100	125	150	188
In two-way footings:						
Plain bars (hooked)	u	$0.045f'_c$ but not to exceed 160 p.s.i.	90	113	135	160
Deformed bars (hooked)	u	$0.056f'_c$ but not to exceed 200 p.s.i.	112	140	168	200
Bearing: f_c						
On full area	f_c	$0.25f'_c$	500	625	750	938
On one-third area or less†	f_c	$0.375f'_c$	750	938	1125	1405

*See Section 807. **See Section 905(a) and 808(a).

†The allowable bearing stress on an area greater than one-third but less than the full area shall be interpolated between the values given.

‡Where special anchorage is provided (see Section 903(a), one and one-half times these values in bond may be used in beams, slabs and one-way footings, but in no case to exceed 200 p.s.i. for plain bars and 250 p.s.i. for deformed bars. The values given for two-way footings include an allowance for special anchorage.

18,000 p.s.i. for Billet-Steel Concrete Reinforcement Bars (of structural grade), and Axle-Steel Concrete Reinforcement Bars (of structural grade).

(b) *Tension in One-Way Slabs of Not More Than 12 Feet Span* (f_s = Tensile unit stress in main reinforcement).

For the main reinforcement, ⅜ inch or less in diameter, in one-way slabs, 50 percent of the minimum yield point specified in the Standard

Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

(*c*) *Compression, Vertical Column Reinforcement*

(f_s = Nominal working stress in vertical column reinforcement).

Forty percent of the minimum yield point specified in the Standard Specifications of the American Society for Testing Materials for the particular kind and grade of reinforcement used, but in no case to exceed 30,000 p.s.i.

(f_r = Allowable unit stress in the metal core of composite and combination columns):

Structural steel sections . . . . . 16,000 p.s.i.
Cast iron sections . . . . . 10,000 p.s.i.
Steel pipe . . . . . See limitations of Section 1106(b)

(*d*) *Compression, Flexural Members*

For compression reinforcement in flexural members see Section 706(b).

CHAPTER 4—MIXING AND PLACING CONCRETE

401—Preparation of equipment and place of deposit

(*a*) Before placing concrete, all equipment for mixing and transporting the concrete shall be cleaned, all debris and ice shall be removed from the spaces to be occupied by the concrete, forms shall be thoroughly wetted (except in freezing weather) or oiled, and masonry filler units that will be in contact with concrete shall be well drenched (except in freezing weather), and the reinforcement shall be thoroughly cleaned of ice or other coatings.

(*b*) Water shall be removed from place of deposit before concrete is placed unless otherwise permitted by the Commissioner of Buildings.

402—
(d) Delete "Serial" and "-38."

402—Mixing of concrete

(*a*) Unless otherwise authorized by the Commissioner of Buildings, the mixing of concrete shall be done in a batch mixer of approved type.

(*b*) The concrete shall be mixed until there is a uniform distribution of the materials and shall be discharged completely before the mixer is recharged.

(*c*) For job mixed concrete, the mixer shall be rotated at a speed recommended by the manufacturers and mixing shall be continued for at least one minute after all materials are in the mixer. A longer mixing period may be required for mixers larger than one cubic yard capacity.

(*d*) Ready-mixed concrete shall be mixed and delivered in accordance with the requirements set forth in the "Standard Specifications for Ready-Mixed Concrete" (A. S. T. M. Serial Designation C94-38).

403—Conveying

(*a*) Concrete shall be conveyed from the mixer to the place of final deposit by methods which will prevent the separation or loss of the materials.

(*b*) Equipment for chuting, pumping and pneumatically conveying concrete shall be of such size and design as to insure a practically continuous flow of concrete at the delivery end without separation of the materials.

404—Depositing

(*a*) Concrete shall be deposited as nearly as practicable in its final position to avoid segregation due to rehandling or flowing. The concreting shall be carried on at such a rate that the concrete is at all times plastic and flows readily into the spaces between the bars. No concrete that has partially hardened or been contaminated by foreign materials shall be deposited on the work, nor shall retempered concrete be used.

(*b*) When concreting is once started, it shall be carried on as a continuous operation until the placing of the panel or section is completed. The top surface shall be generally level. When construction joints are necessary, they shall be made in accordance with Section 508.

(*c*) All concrete shall be thoroughly compacted by suitable means during the operation of placing, and shall be thoroughly worked around the reinforcement and embedded fixtures and into the corners of the forms. Vibrators may be used to aid in the placement of the concrete provided they are used under experienced supervision, and the forms are designed to withstand their action.

(*d*) Where conditions make compacting difficult, or where the reinforcement is congested, batches of mortar containing the same proportions of cement to sand as used in the concrete, shall first be deposited in the forms to a depth of at least one inch.

405—Curing

(*a*) In all concrete structures, concrete made with normal portland cement shall be maintained in a moist condition for at least the first seven days after placing and high-early-strength concrete shall be so maintained for at least the first three days.

406—Cold weather requirements

(*a*) Adequate equipment shall be provided for heating the concrete materials and protecting the concrete during freezing or near-freezing weather. No frozen materials or materials containing ice shall be used.

(*b*) All concrete materials and all reinforcement, forms, fillers and ground with which the concrete is to come in contact, shall be free from frost. Whenever the temperature of the surrounding air is below 40 degrees Fahrenheit, all concrete when placed in the forms shall have a temperature of between 60 and 90 degrees Fahrenheit and shall be maintained at a temperature of not less than 50 degrees Fahrenheit for at least 72 hours for normal concrete or 24 hours for high-early-strength concrete, or for as much more time as is necessary to insure proper rate of curing of the concrete. The housing, covering or other protection used in connection with curing shall remain in place and intact at least twenty-four hours after the artificial heating is discontinued. No dependence shall be placed on salt or other chemicals for the prevention of freezing. Manure, when used for protection, shall not be allowed to come into contact with the concrete.

CHAPTER 5—FORMS AND DETAILS OF CONSTRUCTION

501—Design of forms

(*a*) Forms shall conform to the shape, lines, and dimensions of the members as called for on the plans, and shall be substantial and sufficiently tight to prevent leakage of mortar. They shall be properly braced or tied together so as to maintain position and shape.

502—Removal of forms

(*a*) Forms shall be removed in such manner as to insure the complete safety of the structure. Where the structure as a whole is supported on shores, the removable floor forms, beam and girder sides, column and similar vertical forms may be removed after twenty-four hours, providing the concrete is sufficiently hard not to be injured thereby. In no case shall the supporting forms or shoring be removed until the members have acquired sufficient strength to support safely their weight and the load thereon. The results of suitable control tests may be used as evidence that the concrete has attained such sufficient strength.

503—

Place "*" after title and add footnote:

"*Since this section was adopted a number of concrete floors have been built with pipes embedded for radiant heating which is prohibited by this section. The Committee is studying the problem for the purpose of revising the requirements to permit the safe use of such pipes in structural concrete."

503—Pipes, conduits, etc., embedded in concrete

(*a*) Pipes which will contain liquid, gas or vapor at other than room temperature shall not be embedded in concrete necessary for structural stability or fire protection. Drain pipes and pipes whose contents will be under pressure greater than atmospheric pressure by more than one pound per square inch shall not be embedded in structural concrete

except in passing through from one side to the other of a floor, wall or beam. Electric conduits and other pipes whose embedment is allowed shall not, with their fittings, displace that concrete of a column on which stress is calculated or which is required for fire protection, to greater extent than four per cent of the area of the cross section. Sleeves or other pipes passing through floors, walls or beams shall not be of such size or in such location as unduly to impair the strength of the construction; such sleeves or pipes may be considered as replacing structurally the displaced concrete, provided they are not exposed to rusting or other deterioration, are of uncoated iron or steel not thinner than standard wrought-iron pipe, have a nominal inside diameter not over two inches, and are spaced not less than three diameters on centers. Embedded pipes or conduits other than those merely passing through, shall not be larger in outside diameter than one-third the thickness of the slab, wall or beam in which they are embedded; shall not be spaced closer than three diameters on centers, nor so located as unduly to impair the strength of the construction. Circular uncoated or galvanized electric conduit of iron or steel may be considered as replacing the displaced concrete.

504—

Line 4, after "bar" add, "Hooks shall conform to the requirements of Section 906."

504—Cleaning and bending reinforcement

(*a*) Metal reinforcement, at the time concrete is placed, shall be free from rust scale or other coatings that will destroy or reduce the bond. Bends for stirrups and ties shall be made around a pin having a diameter not less than two times the minimum thickness of the bar. Bends for other bars shall be made around a pin having a diameter not less than six times the minimum thickness of the bar, except that for bars larger than one inch, the pin shall be not less than eight times the minimum thickness of the bar. All bars shall be bent cold.

505—

(a) Line 10. Add "Where reinforcement in beams or girders is placed in two or more layers, the clear distance between layers shall be not less than 1 in., and the bars in the upper layers shall be placed directly above those in the bottom layer."

505—Placing reinforcement

(*a*) Metal reinforcement shall be accurately placed and adequately secured in position by concrete or metal chairs and spacers. The minimum clear distance between parallel bars shall be one and one-half times the diameter for round bars and twice the side dimension for square bars. If special anchorage as required in Section 903 is provided, the minimum clear distance between parallel bars shall be equal to the diameter for round bars and one and one-half times the side dimension for square bars.

In no case shall the clear distance between bars be less than one in., nor less than one and one-third times the maximum size of the coarse aggregate.

(*b*) When wire or other reinforcement, not exceeding one-fourth inch in diameter is used as reinforcement for slabs not exceeding ten feet in span, the reinforcement may be curved from a point near the top of the slab over the support to a point near the bottom of the slab at mid-span; provided such reinforcement is either continuous over, or securely anchored to the support.

506—

Title, delete, "and offsets".

(a) Line 3, change sentence "In such . . . Section 505," to "The clear distance between bars shall also apply to the clear distance between a contact splice and adjacent contact splices or bars".

(b) Change paragraph to "Splices in the reinforcement of columns are specified in Section 1103(c)".

506—Splices and offsets in reinforcement

(*a*) In slabs, beams and girders, splices of reinforcement at points of maximum stress shall generally be avoided. Splices shall provide sufficient lap to transfer the stress between bars by bond and shear. In such splices the mimimum spacing of bars shall be as specified in Section 505.

(*b*) Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded. Where offset, the slope of the inclined portion shall not be more than 1 in 6, and in the case of tied columns the ties shall be spaced not over three inches on centers for a distance of one foot below the actual point of offset.

507—

(b) Line 6, delete "metal".

(c) Line 1, change "these Regulations form" to "this Code forms".

507—Concrete protection for reinforcement

(*a*) The reinforcement of footings and other principal structural members in which the concrete is deposited against the ground shall have not less than three inches of concrete between it and the ground contact surface. If concrete surfaces after removal of the forms are to be exposed to the weather or be in contact with the ground, the reinforcement shall be protected with not less than two inches of concrete for bars more than ⅝ inch in diameter and one and one-half inches for bars ⅝ inch or less in diameter.

(*b*) The concrete protective covering for reinforcement at surfaces not exposed directly to the ground or weather shall be not less than three-fourths inch for slabs and walls; and not less than one and one-half

inches for beams, girders and columns. In concrete joist floors in which the clear distance between joists is not more than thirty inches, the protection of metal reinforcement shall be at least three-fourths inch.

(*c*) If the code of which these regulations form a part specifies, as fire-protective covering of the reinforcement, thicknesses of concrete greater than those given in this section, then such greater thicknesses shall be used.

(*d*) Concrete protection for reinforcement shall in all cases be at least equal to the diameter of round bars, and one and one-half times the side dimension of square bars.

(*e*) Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion by concrete or other adequate covering.

508—

(a) Line 5, delete "but not saturated".

508—Construction joints

(*a*) Joints not indicated on the plans shall be so made and located as to least impair the strength of the structure. Where a joint is to be made, the surface of the concrete shall be thoroughly cleaned and all laitance removed. In addition to the foregoing, vertical joints shall be thoroughly wetted but not saturated, and slushed with a coat of neat cement grout immediately before placing of new concrete.

(*b*) At least two hours must elapse after depositing concrete in the columns or walls before depositing in beams, girders, or slabs supported thereon. Beams, girders, brackets, column capitals, and haunches shall be considered as part of the floor system and shall be placed monolithically therewith.

(*c*) Construction joints in floors shall be located near the middle of the spans of slabs, beams, or girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. In this last case provision shall be made for shear by use of inclined reinforcement.

CHAPTER 6—DESIGN—GENERAL CONSIDERATIONS

600—

Line 1, delete "Ultimate"; capitalize "c" in "compressive".

600—Notation

f'_c = Ultimate compressive strength of concrete at age of 28 days, unless otherwise specified.

n = Ratio of modulus of elasticity of steel to that of concrete = $\frac{E_s}{E_c}$; assumed as equal to $\frac{30{,}000}{f'_c}$.

601—

(a) Line 2, change "working" to "allowable".

601—Assumptions

(*a*) The design of reinforced concrete members shall be made with reference to working stresses and safe loads. The accepted theory of flexure as applied to reinforced concrete shall be applied to all members resisting bending. The following assumptions shall be made:

1. The steel takes all the tensile stress.
2. In determining the ratio n for design purposes, the modulus of elasticity for the concrete shall be assumed as 1000 f'_c, and that for steel as 30,000,000 p.s.i.

602—Design loads

(*a*) The provisions for design herein specified are based on the assumption that all structures shall be designed for all dead- and live-loads coming upon them, the live-loads to be in accordance with the general requirements of the building code of which this forms a part, with such reductions for girders and lower story columns as are permitted therein.

603—Resistance to wind forces

(*a*) The resisting elements in structures required to resist wind forces shall be limited to the integral structural parts.

(*b*) The moments, shears, and direct stresses resulting from wind forces determined in accordance with recognized methods shall be added to the maximum stresses which obtain at any section for dead- and live-loads.

(*c*) In proportioning the component parts of the structure for the maximum combined stresses, including wind stresses, the unit stresses shall not exceed the allowable stresses for combined live- and dead-loads provided in Sections 305, 306 and 1110 by more than one-third. The structural members and their connections shall be so proportioned as to provide suitable rigidity of structure.

CHAPTER 7—FLEXURAL COMPUTATIONS

700—

Delete terms and definitions: A, B, e_A, e_B, F_AA, F_BB, F_A, F_B, K_A, K_B, K_{AR}, K_{BR}, N, q_A, q_B, r_A, r_B, x.

Change definition for "b" to: "Width of rectangular flexural member or width of flanges for T and I sections."

Change definition for "b'" to: "Width of web in T and I flexural members"

Line 5, change "center" to "centroid".

In "l'" line 1, after "moment" add "and shear".

700—Notation

A = Span length between opposite supports in one direction.

B = Span length at right angles to A.

b = Width of rectangular beam or width of flange of T-beam.

b' = Width of web in beams of I or T sections.

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement; the diameter of a round bar or side of a square bar.

e_A = Factor modifying r_A, used in obtaining an equivalent uniform load for bending moments on span A.

e_B = Factor modifying r_B, used in obtaining an equivalent uniform load for bending moments on span B.

E = The modulus of elasticity of concrete in compression.

F_AA = The distance between lines of inflection in span A, considering span A only to be loaded.

F_BB = The distance between lines of inflection in span B, considering span B only to be loaded.

F_A = Ratio of the distance between assumed inflections points of the span A to span A in an isolated strip extending the entire width of the structure when a uniformly distributed load is applied to span A only.

F_B = Ratio as defined above, but applying to span B.

h = Unsupported length of a column.

I = Moment of inertia of a section about the neutral axis for bending.

K = The stiffness factor, that is, the moment of inertia divided by the span.

K_A = Stiffness factor $\frac{I}{A}$ for span A of panel AB.

K_B = Stiffness factor $\frac{I}{B}$ for span B of panel AB.

K_{AR} = Stiffness factors for any span adjacent to and continuous with span A.

K_{BR} = Stiffness factors for any span adjacent to and continuous with span B.

l = Span length of slab or beam.

l' = Clear span for positive moment and the average of the two adjacent clear spans for negative moment (See Section 701).

N = The sum of the lengths of those edges of panel AB which are also edges of adjacent panels continuous with AB.

q_A = $6r_A\ (1 - e_A)$.

q_B = $6r_B\ (1 - e_B)$.

r_A = Proportion of the total load carried by span A of slab.
r_B = Proportion of the total load carried by span B of slab.
t_1 = Minimum total thickness of slab.
w = Uniformly distributed load per unit of length of beam or per unit area of slab.
x = Distance from face of support to point in span.

701—General requirements

(*a*) All members of frames or continuous construction shall be designed to resist at all sections the maximum moments and shears produced by dead load, live load and wind load, as determined by the theory of elastic frames in which the simplified assumptions of Section 702 may be used.

(*b*) Approximate methods of frame analysis are satisfactory for buildings of usual types of construction, spans and story heights.

(*c*) In the case of two or more approximately equal spans (the larger of two adjacent spans not exceeding the shorter by more than 20 per cent) with loads uniformly distributed, where the unit live load does not exceed three times the unit dead load, design for the following moments and shears is satisfactory:

Positive moment at center of span

End spans . . . . . $\frac{1}{14} wl'^2$

Interior Spans . . . . . $\frac{1}{16} wl'^2$

Negative moment at exterior face of first interior support

Two spans . . . . . $\frac{1}{9} wl'^2$

More than two spans . . . . . $\frac{1}{10} wl'^2$

Negative moment at other faces of interior supports $\frac{1}{11} wl'^2$

Negative moment at face of all supports for, (a) slabs with spans not exceeding ten feet, and (b) beams and girders where ratio of sum of column stiffnesses to beam stiffness exceeds eight $\frac{1}{12} wl'^2$

Shear in end members at first interior support $1.15 \frac{wl'}{2}$

Shear at other supports $\frac{wl'}{2}$

702—

(e) After sub-paragraphs 1, 2 and 3, add "(Use b instead of b′ for rectangular flexural members.)"

702—Conditions of design*

(*a*) *Arrangement of Live Load*

1. The live load may be considered to be applied only to the floor under consideration, and the far ends of the columns may be assumed as fixed.

2. Consideration may be limited to combinations of dead load on all spans with full live load on two adjacent spans and with full live load on alternate spans.

(*b*) *Span length*

1. The span length, l, of members that are not built integrally with their supports shall be the clear span plus the depth of the slab or beam but shall not exceed the distance between centers of supports.

2. In analysis of continuous frames, center to center distances, l and h, may be used in the determination of moments. Moments at faces of supports may be used for design of beams and girders.

3. Solid or ribbed slabs with clear spans of not more than ten feet that are built integrally with their supports may be designed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and the width of beams otherwise neglected.

(*c*) *Stiffness*

1. The stiffness, K, of a member is defined as EI divided by l or h.

2. In computing the value of I of slabs, beams, girders, and columns, the reinforcement may be neglected. In T-shaped sections allowance shall be made for the effect of flange.

3. Any reasonable assumption may be adopted as to relative stiffness of columns and of floor system. The assumption made shall be consistent throughout the analysis.

(*d*) *Haunched Floor Members*

1. When members are widened near the supports, the additional width may be neglected in computing moments, but may be considered as resisting the resulting moments and shears.

2. When members are deepened near the supports, they may be analyzed as members of constant depth provided the minimum depth only is considered as resisting the resulting moments; otherwise an

*Chapter 7 deals with floor members only. For moments in columns see Section 1108.

analysis taking into account the variation in depth is required. In any case, the actual depth may be considered as resisting shear.

(*e*) *Limitations*

1. Wherever at any section positive reinforcement is indicated by analysis, the amount provided shall be not less than .005 $b'd$ except in slabs of uniform thickness.

2. Not less than 0.005 $b'd$ of negative reinforcement shall be provided at the outer end of all members built integrally with their supports.

3. Where analysis indicates negative reinforcement along the full length of a span, the reinforcement need not be extended beyond the point where the required amount is 0.0025 $b'd$ or less.

4. In slabs of uniform thickness the minimum amount of reinforcement in the direction of the span shall be:

For structural, intermediate and hard grades and rail steel . . 0.0025 bd
For steel having a minimum yield point of 56,000 p. s. i. . . . . 0.002 bd

703—Depth of beam or slab

(*a*) The depth of the beam or slab shall be taken as the distance from the centroid of the tensile reinforcement to the compression face of the structural members. Any floor finish not placed monolithically with the floor slab shall not be included as a part of the structural member. When the finish is placed monolithically with the structural slab in buildings of the warehouse or industrial class, there shall be placed an additional depth of one-half inch over that required by the design of the member.

704—Distance between lateral supports

(*a*) The clear distance between lateral supports of a beam shall not exceed thirty-two times the least width of compression flange.

705—
(c) Line 5, after "T-beam" add, "The flange shall be assumed to act as a cantilever".
Line 5, change "assumed as" to "required for".

705—Requirements for T-beams

(*a*) In T-beam construction the slab and beam shall be built integrally or otherwise effectively bonded together. The effective flange width to be used in the design of symmetrical T-beams shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times the thickness of the slab nor one-half the clear distance to the next beam.

(*b*) For beams having a flange on one side only, the effective overhanging flange width shall not exceed one-twelfth of the span length of

the beam, nor six times the thickness of the slab, nor one-half the clear distance to the next beam.

(*c*) Where the principal reinforcement in a slab which is considered as the flange of a T-beam (not a joist in concrete joist floors) is parallel to the beam, transverse reinforcement shall be provided in the top of the slab. This reinforcement shall be designed to carry the load on the portion of the slab assumed as the flange of the T-beam. The spacing of the bars shall not exceed five times the thickness of the flange, nor in any case eighteen inches.

(*d*) Provision shall be made for the compressive stress at the support in continuous T-beam construction, care being taken that the provisions of Section 505 relating to the spacing of bars, and 404(d), relating to the placing of concrete shall be fully met.

(*e*) The overhanging portion of the flange of the beam shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

(*f*) Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web and a total flange width not more than four times the web thickness.

706—Compression steel in flexural members

(*a*) Compression steel in beams, girders, or slabs shall be anchored by ties or stirrups not less than ¼ inch in diameter spaced not farther apart than 16 bar diameters, or 48 tie diameters. Such stirrups or ties shall be used throughout the distance where the compression steel is required.

(*b*) The effectiveness of compression reinforcement in resisting bending may be taken at twice the value indicated from the calculations assuming a straight-line relation between stress and strain and the modular ratio given in Section 601, but not of greater value than the allowable stress in tension.

707—Shrinkage and temperature reinforcement

(*a*) Reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided in floor and roof slabs where the principal reinforcement extends in one direction only. Such reinforcement shall provide for the following minimum ratios of reinforcement area to concrete *bd*, but in no case shall such reinforcing bars be placed farther apart than five times the slab thickness nor more than eighteen inches:

Floor slabs where plain bars are used	0.0025
Floor slabs where deformed bars are used	0.002

Floor slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0018
Roof slabs where plain bars are used	0.003
Roof slabs where deformed bars are used	0.0025
Roof slabs where wire fabric is used, having welded intersections not farther apart in the direction of stress than twelve inches	0.0022

708—

(c) Change second sentence to "Shrinkage reinforcement shall be provided in the slab at right angles to the joists as required in Section 707, substituting t_1, (total thickness of slab) for d".

(d) Change second sentence to "Such slab shall be reinforced at right angles to the joists with a minimum of at least the amount of reinforcement required for flexure giving due consideration to concentrations, if any, but in no case shall the reinforcement be less than that required by Section 707, considering t_1 as the full thickness of the slab".

Add paragraph: (g) Shrinkage reinforcement shall not be required in the slab parallel to the joists".

708—Concrete joist floor construction

(*a*) Concrete joist floor construction consists of concrete joists and slabs placed monolithically with or without burned clay or concrete tile fillers. The joists shall not be farther apart than thirty inches face to face. The ribs shall be straight, not less than four inches wide, nor of a depth more than three times the width.

(*b*) When burned clay or concrete tile fillers, of material having a unit compressive strength at least equal to that of the designed strength of the concrete in the joists are used, and the fillers are so placed that the joints in alternate rows are staggered, the vertical shells of the fillers in contact with the joists may be included in the calculations involving shear or negative bending moment. No other portion of the fillers may be included in the design calculations.

(*c*) The concrete slab over the fillers shall be not less than one and one-half inches in thickness, nor less in thickness than one-twelfth of the clear distance between joists. Shrinkage reinforcement in the slab shall be provided as required in Section 707.

(*d*) Where removable forms or fillers not complying with (*b*) are used, the thickness of the concrete slab shall not be less than one-twelfth of the clear distance between joists and in no case less than two inches. Such slab shall be reinforced at right angles to the joists with a minimum of .049 sq. in. of reinforcing steel per foot of width, and in slabs on which the prescribed live loads does not exceed fifty lb. per sq. ft., no additional reinforcement shall be required.

(*e*) When the finish used as a wearing surface is placed monolithically with the structural slab in buildings of the warehouse or industrial class, the thickness of the concrete over the fillers shall be one-half inch greater than the thickness used for design purposes.

(*f*) Where the slab contains conduits or pipes, the thickness shall not be less than one inch plus the total over-all depth of such conduits or pipes at any point. Such conduits or pipes shall be so located as not to impair the strength of the construction.

709—

Title, change "(1) (2)" to "*".

Change entire section to:

(a) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with fillers of hollow concrete units or clay title, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. The slab shall be supported by walls or beams on all sides and if not securely attached to supports, shall be reinforced as specified in 709(b).

(b) Where the slab is not securely attached to the supporting beams or walls, special reinforcement shall be provided at exterior corners in both the bottom and top of the slab. This reinforcement shall be provided for a distance in each direction from the corner equal to 1/5 the longest span. The reinforcement in the top of the slab shall be parallel to the diagonal from the corner. The reinforcement in the bottom of the slab shall be at right angles to the diagonal or may be of bars in two directions parallel to the sides of the slab. The reinforcement in each band shall be of equivalent size and spacing to that required for the maximum positive moment in the slab.

(c) The slab and its supports shall be designed by approved methods which shall take into account the effect of continuity at supports, the ratio of length to width of slab and the effect of two-way action.

(d) In no case shall the slab thickness be less than 4 in. nor less than the perimeter of the slab divided by 180. The spacing of reinforcement shall be not more than 3 times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

Change footnote "(1) (2)" to:

*The requirements of this section are satisfied by either of the following methods of design:

METHOD 1

Notation—

L = Length of clear span

L_1 = Length of clear span in the direction normal to L

g = Ratio of span between lines of inflection to L in the direction of span L, when span L only is loaded.

g_1 = Ratio of span between lines of inflection to L_1 in the direction of span L_1, when span L_1 only is loaded.

$r = \dfrac{gL}{g_1L_1}$

w = Total uniform load per sq. ft.

W = Total uniform load between opposite supports on slab strip of any width or total slab load on beam when considered as one-way construction.

x = Ratio of distance from support to any section of slab or beam, to span L or L_1.

C = Factor modifying bending moments prescribed for one-way construction for use in proportioning the slabs and beams in the direction of L of slabs supported on four sides.

C_s = Ratio of the shear at any section of a slab strip distant xL from the support to the total load W on the strip in direction of L.

C_b = Ratio of the shear at any section of a beam distant xL from the support to the total load W on the beam in the direction of L.

(*Revised footnote continued p. 429*)

(*Revised code footnote, sec. 709 continued from p. 428*).

B = Bending moment coefficient for one-way construction.

W_1, C_1, C_{a1}, C_{b1}, are corresponding values of W, C, C_a, C_b, for slab strip or beam in direction of L_1.

(*a*) *Lines of inflection for determination of r*—The lines of inflection shall be determined by elastic analysis of the continuous structure in each direction, when only the span under consideration is loaded.

When the span L or L_1 is at least 2/3 and at most 3/2 of the adjacent continuous span or spans, the values of g or g_1 may be taken as 0.87 for exterior spans and 0.76 for interior spans. (See Fig. 1).

For spans discontinuous at both ends, g or g_1 shall be taken as unity.

(*b*) *Bending moments and shear*—Bending moments shall be determined in each direction with the coefficients prescribed for one-way construction in Sections 701 and 702 and modified by factor C or C_1 from Tables 1 or 2.

TABLE 1—SLABS

Upper figure / Lower figure		C_a / C_{a1}					C / C_1
	$\frac{1}{r}$	Values of x					
r		0.0	.1	.2	.3	.4	
0.00	∞	.50 / .00	.40 / .00	.30 / .00	.20 / .00	.10 / .00	1.00 / .00
.50	2.00	.44 / .06	.36 / .03	.27 / .02	.18 / .00	.09 / .00	.89 / .06
.55	1.82	.43 / .07	.33 / .04	.23 / .02	.15 / .01	.07 / .00	.79 / .08
.60	1.67	.41 / .09	.30 / .05	.20 / .03	.12 / .01	.05 / .00	.70 / .10
.65	1.54	.39 / .11	.28 / .06	.18 / .03	.10 / .01	.04 / .00	.64 / .13
.70	1.43	.37 / .13	.26 / .08	.16 / .04	.09 / .01	.03 / .00	.58 / .15
.80	1.25	.33 / .17	.22 / .10	.13 / .06	.07 / .02	.02 / .00	.48 / .21
.90	1.11	.29 / .21	.19 / .13	.11 / .07	.05 / .03	.01 / .01	.40 / .27
1.00	1.00	.25 / .25	.16 / .16	.09 / .09	.04 / .04	.01 / .01	.33 / .33
1.10	.91	.21 / .29	.13 / .19	.07 / .11	.03 / .05	.01 / .01	.28 / .39
1.20	.83	.18 / .32	.11 / .21	.06 / .13	.02 / .06	.00 / .02	.23 / .45
1.30	.77	.16 / .34	.10 / .23	.05 / .14	.02 / .07	.00 / .03	.19 / .51
1.40	.71	.13 / .37	.08 / .25	.04 / .16	.02 / .09	.00 / .03	.16 / .57
1.50	.67	.11 / .39	.07 / .27	.04 / .17	.01 / .10	.00 / .04	.14 / .61
1.60	.63	.10 / .40	.06 / .29	.03 / .19	.01 / .11	.00 / .05	.12 / .66
1.80	.55	.07 / .43	.04 / .33	.02 / .23	.01 / .15	.00 / .07	.08 / .79
2.00	.50	.06 / .44	.03 / .36	.02 / .27	.00 / .18	.00 / .09	.06 / .89
∞	0.00	.00 / .50	.00 / .40	.00 / .30	.00 / .20	.00 / .10	.00 / 1.00

(*Revised footnote continued p. 430*)

(Revised code footnote, sec. 709, continued from p. 429)

	In L Direction	In L_1 Direction
B.M. for slab strip	$M = CBWL$	$M_1 = C_1BW_1L_1$
B.M. for beam	$M = (1\text{-}C)BWL$	$M_1 = (1\text{-}C_1)\ BW_1L_1$

When the coefficients prescribed in 701(c) are used, the average value of Cw or C_1w for the two spans adjacent to a support shall be used in determining the negative bending moment at the face of the support.

The shear at any section distant xL or xL_1 from supports shall be determined by modifying the total load on the slab strip or beam by the factors C_s, C_{s1}, C_b or C_{b1} taken from Tables 1 or 2.

	In L Direction	In L_1 Direction
Shear for Slab Strip	$V = C_sW$	$V_1 = C_{s1}W_1$
Shear for Beam	$V = C_bW$	$V_1 = C_{b1}W_1$

For spans where the end moments are unbalanced, shear values at any section shall be adjusted in accordance with Sections 701 and 702.

(c) *Arrangement of reinforcement:*

1. In any panel, the area of reinforcement per unit width in the long direction shall be at least one-third that provided in the short direction.
2. The area of positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced 25 per cent.
3. At a non-continuous edge the area of negative moment reinforcement per unit width shall be at least one-half of that required for maximum positive moment.

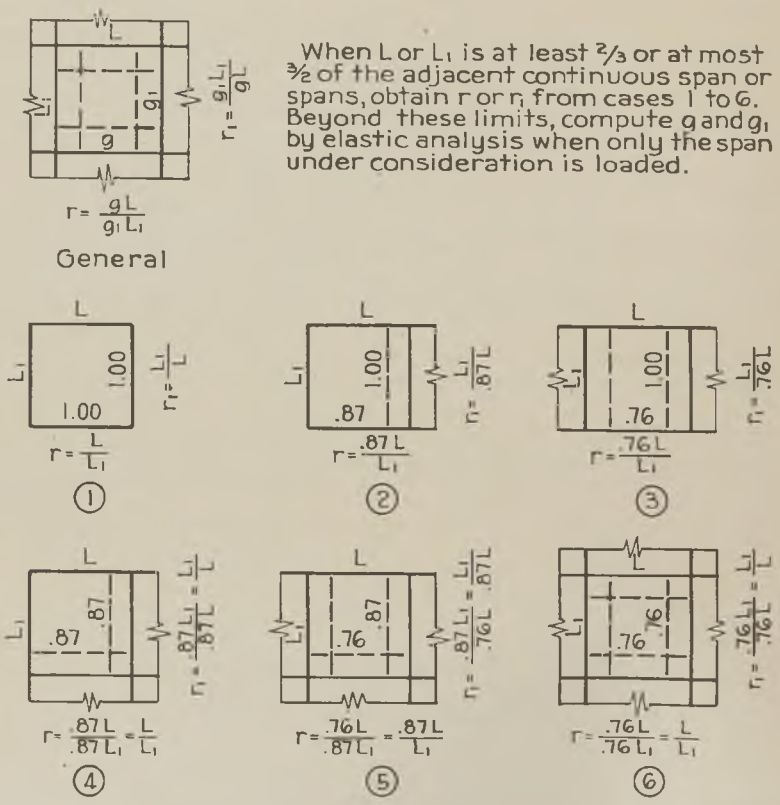

Fig. 1

(Revised footnote continued p. 431)

(Revised footnote, sec. 709, continued from p. 430)

TABLE 2—BEAMS

Upper figure / Lower figure		C_b / C_{b1}					$1-C$ / $1-C_1$
	$\frac{1}{r}$	Values of x					
r		0.0	.1	.2	.3	.4	
0.00	∞	.00 / .50	.00 / .40	.00 / .30	.00 / .20	.00 / .10	.00 / 1.00
.50	2.00	.06 / .44	.04 / .37	.03 / .28	.02 / .20	.01 / .10	.11 / .94
.55	1.82	.07 / .43	.07 / .36	.07 / .28	.05 / .19	.03 / .10	.21 / .92
.60	1.67	.09 / .41	.10 / .35	.10 / .27	.08 / .19	.05 / .10	.30 / .90
.65	1.54	.11 / .39	.12 / .34	.12 / .27	.10 / .19	.06 / .10	.36 / .87
.70	1.43	.13 / .37	.14 / .32	.14 / .26	.11 / .19	.07 / .10	.42 / .85
.80	1.25	.17 / .33	.18 / .30	.17 / .24	.13 / .18	.08 / .10	.52 / .79
.90	1.11	.21 / .29	.21 / .27	.19 / .23	.15 / .17	.09 / .09	.60 / .73
1.00	1.00	.25 / .25	.24 / .24	.21 / .21	.16 / .16	.09 / .09	.67 / .67
1.10	.91	.29 / .21	.27 / .21	.23 / .19	.17 / .15	.09 / .09	.72 / .61
1.20	.83	.32 / .18	.29 / .19	.24 / .17	.18 / .14	.10 / .08	.77 / .55
1.30	.77	.34 / .16	.30 / .17	.25 / .16	.18 / .13	.10 / .07	.81 / .49
1.40	.71	.37 / .13	.32 / .15	.26 / .14	.18 / .11	.10 / .07	.84 / .43
1.50	.67	.39 / .11	.33 / .13	.26 / .13	.19 / .10	.10 / .06	.86 / .39
1.60	.63	.40 / .10	.34 / .11	.27 / .11	.19 / .09	.10 / .05	.88 / .34
1.80	.55	.43 / .07	.36 / .07	.28 / .07	.19 / .05	.10 / .03	.92 / .21
2.00	.50	.44 / .06	.37 / .04	.28 / .03	.20 / .02	.10 / .01	.94 / .11
∞	0.00	.50 / .00	.40 / .00	.30 / .00	.20 / .00	.10 / .00	1.00 / .00

METHOD 2

Notation—

C = Moment coefficient for two-way slabs as given in Table 3
m = Ratio of short span to long span for two-way slabs
S = Length of short span for two-way slabs
The span shall be considered as the center-to-center distance between supports or the clear span plus twice the thickness of slab, whichever value is the smaller.
w = Total uniform load per sq. ft.

(*a*) *Limitations*—A two-way slab shall be considered as consisting of strips in each direction as follows:

A middle strip one-half panel in width, symmetrical about panel center line and extending through the panel in the direction in which moments are considered.

A column strip one-half panel in width, occupying the two quarter-panel areas outside the middle strip.

(Revised footnote concluded p. 432)

(Revised footnote, sec. 709, concluded)

TABLE 3—MOMENT COEFFICIENTS

Moments	Short span, Values of m: 1.0	0.9	0.8	0.7	0.6	0.5 and less	Long span, all values of m
Case 1—Interior panels							
Negative moment at—							
Continuous edge	.033	.040	.048	.055	.063	.083	.033
Discontinuous edge	—	—	—	—	—	—	—
Positive moment at midspan	.025	.030	.036	.041	.047	.062	.025
Case 2—One edge discontinuous							
Negative moment at—							
Continuous edge	.041	.048	.055	.062	.069	.085	.041
Discontinuous edge	.021	.024	.027	.031	.035	.042	.021
Positive moment at midspan	.031	.036	.041	.047	.052	.064	.031
Case 3—Two edges discontinuous							
Negative moment at—							
Continuous edge	.049	.057	.064	.071	.078	.090	.049
Discontinuous edge	.025	.028	.032	.036	.039	.045	.025
Positive moment at midspan	.037	.043	.048	.054	.059	.068	.037
Case 4—Three edges discontinuous							
Negative moment at—							
Continuous edge	.058	.066	.074	.082	.090	.098	.058
Discontinuous edge	.029	.033	.037	.041	.045	.049	.029
Positive moment at midspan	.044	.050	.056	.062	.068	.074	.044
Case 5—Four edges discontinuous							
Negative moment at—							
Continuous edge	—	—	—	—	—	—	—
Discontinuous edge	.033	.038	.043	.047	.053	.055	.033
Positive moment at midspan	.050	.057	.064	.072	.080	.083	.050

Where the ratio of short to long span is less than 0.5 the middle strip in the short direction shall be considered as having a width equal to the difference between the long and short span, the remaining area representing the two column strips.

The critical sections for moment calculations are referred to as principal design sections and are located as follows:

For negative moment, along the edges of the panel at the faces of the supporting beams.

For positive moment, along the center lines of the panels.

(*b*) *Bending Moments*—The bending moments for the middle strips shall be computed from the formula

$$M = CwS^2$$

The average moments per foot of width in the column strip shall be two-thirds of the corresponding moments in the middle strip. In determining the spacing of the reinforcement in the column strip, the moment may be assumed to vary from a maximum at the edge of the middle strip to a minimum at the edge of the panel.

Where the negative moment on one side of a support is less than 80 per cent of that on the other side, two-thirds of the difference shall be distributed in proportion to the relative stiffnesses of the slabs.

(*c*) *Shear*—The shearing stresses in the slab may be computed on the assumption that the load is distributed to the supports in accordance with (*d*).

(*d*) *Supporting Beams*—The loads on the supporting beams for a two-way rectangular panel may be assumed as the load within the tributary areas of the panel bounded by the intersection of 45-degree lines from the corners with the median line of the panel parallel to the long side.

The bending moments may be determined approximately by using an equivalent uniform load per lineal foot of beam for each panel supported as follows:

For the short span, $\frac{wS}{3}$

For the long span, $\frac{wS}{3} \quad \frac{(3-m^2)}{2}$

709—Floors with supports on four sides [1] [2]

(*a*) This construction, consisting of floors reinforced in two directions and supported on four sides, includes solid reinforced concrete slabs; concrete joists with burned clay or concrete tile fillers, with or without concrete top slabs; and concrete joists with top slabs placed monolithically with the joists. The supports for the floor slabs may be walls, reinforced concrete beams, or steel beams fully encased in concrete.

(*b*) *Minimum Slab Thickness*

The slab thickness shall satisfy prescribed working stresses and shall be not less than 4 inches nor less than

$$t_1 = \frac{A + B - 0.10N}{72} \qquad (2)$$

(*c*) *Bending Moments and Shears*

The bending moment at any section shall be determined with coefficients derived as prescribed for one-way construction (Sections 701 and 702), using the following equivalent load per unit length of span considered:

Footnotes:

(1) For comparative use the moment of inertia of a slab shall be taken as that of the total plain concrete section.

(2) Formulas for F_A, F_B, e_A, e_B, r_A, r_B. (See "Slabs Supported on Four Sides" by J. DiStasio and M. P. van Buren, JOURNAL of the A. C. I., January-February, 1936).

End Span, continuous at one end only

$$F_A = 1 - \frac{0.25}{1 + \frac{7K_A}{8K_{AR}}} \qquad (7)$$

Interior continuous span with K_{AR} the same for both adjacent spans continuous with A

$$F_A = \sqrt{1 - \frac{1}{1.5 + \frac{7K_A}{8K_{AR}}}} \qquad (8)$$

For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AR} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AR} for the span at the other end.

To obtain F_B replace K_A with K_B and K_{AR} with K_{BR}.

$$r_A = \frac{1}{1 + \left(\frac{F_A A}{F_B B}\right)^3} = 1 - r_B \qquad (9)$$

$$e_A = \frac{2}{4 - \frac{F_B B}{F_A A}} \qquad (10a) \qquad\qquad e_B = \frac{2}{\frac{4}{3} - \frac{F_A A}{F_B B}} \qquad (10b)$$

$$e_A = 1.0 \text{ for } \frac{F_B B}{F_A A} = 2 \qquad\qquad e_B = 0.5, \text{ as } \frac{F_A A}{F_B B} = 0$$

The total load carried by a strip of slab of unit width, span A, equals $r_A w A$ and is considered to vary in intensity from $r_A w\,(3e_A - 2)$ at the center of the span, to $r_A w\,(4 - 3\,e_A)$ at the supports.

The total load carried by a beam of span A, one-half panel tributary width, equals

$$(1 - r_A)\frac{wBA}{2} \qquad (11)$$

and varies uniformly in intensity from $(1 + 2r_A - 3e_A r_A)\frac{wB}{2}$ at the center of the span to

$(1 - 4r_A + 3e_A r_A)\frac{wB}{2}$ at the supports.

When considering the B spans use the above expressions, replacing A with B, B with A, r_A with r_B, and e_A with e_B.

(*Footnote (2) continued next page*)

Slab: Strip of unit width, span A, ... $(e_A r_A)w$ . . . . . (3)

Beam: Span A, carrying one half of load from panel width B,

$$\frac{B}{2}\ (1 - e_A r_A)w \quad \ldots\ldots (4)$$

The shear at any section at a distance x from the face of the support shall be taken as:

Slab: Strip of unit width, span A,

$$\left(\frac{A}{2} - x\right)\left(r_A - \frac{q_A x}{A}\right)w \quad \ldots\ldots (5)$$

Beam: Span A, carrying one half of load from panel, width B,

$$\frac{B}{2}\left(\frac{A}{2} - x\right)\left(1 - r_A + \frac{q_A x}{A}\right)w \quad \ldots\ldots (6)$$

(*Footnote (2) continued from previous page.*

TABLE 1—F_A AND F_B

The values given in the table are for F_A directly. They are also the values for F_B when the designation K_A/K_{AB} is replaced by K_B/K_{BA}.

Span A	$\frac{K_A}{K_{AB}}$ =	0.00	0.25	0.50	0.67	0.80	1.00	1.25	1.50	1.00	4.00	0. ∞
Interior* . .	F_A =	0.58	0.65	0.69	0.72	0.74	0.76	0.78	0.80	0.83	0.89	1.00
End	F_A =	0.75	0.80	0.83	0.84	0.85	0.87	0.88	0.89	0.91	0.95	1.00
Simple	F_A =	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

*For interior spans where the spans adjacent to and in continuation of the span A under consideration differ in stiffness, for F_A use the average of the two values, one obtained using K_{AB} for the span in continuation on one end of the span A, and the other obtained by using the value of K_{AB} for the span at the other end.

For values of K_A/K_{AB} between 2/3 and 3/2 the values of F_A may be taken as 0.76 for interior spans and 0.87 for end spans.

TABLE 2

The value of e_A or e_B shall be taken as unity for the computation of shear and bending moment in slabs and beams where the span in direction under consideration is not rigidly attached to the supports at one or both ends of the span.

$\frac{F_A A}{F_B B}$	r_A or $1 - r_B$	e_A	$e_A r_A$	$1 - e_A r_A$	q_A = $6r_A\ (1 - e_A)$
0.00	1.00	1.00	1.00	0.00	0.00
0.50	0.89	1.00	0.89	0.11	0.00
0.55	0.86	0.92	0.79	0.21	0.41
0.60	0.82	0.86	0.71	0.29	0.69
0.65	0.78	0.81	0.63	0.37	0.89
0.70	0.74	0.78	0.58	0.42	0.98
0.80	0.66	0.73	0.48	0.52	1.07
0.90	0.58	0.69	0.40	0.60	1.08
1.00	0.50	0.67	0.33	0.67	1.00
1.10	0.43	0.65	0.28	0.72	0.90
1.20	0.37	0.63	0.23	0.77	0.82
1.30	0.31	0.62	0.19	0.81	0.71
1.40	0.27	0.61	0.16	0.84	0.63
1.50	0.23	0.60	0.14	0.86	0.55
1.60	0.20	0.59	0.12	0.88	0.49
1.80	0.15	0.58	0.09	0.91	0.38
2.00	0.11	0.57	0.06	0.94	0.28
3.00	0.04	0.55	0.02	0.98	0.11
$\frac{F_B B}{F_A A}$	r_B or $1 - r_A$	e_B	$e_B r_B$	$1 - e_B r_B$	$6r_B\ (1 - e_B)$ = q_B

For span B, use the above expressions substituting A for B, B for A, e_B for e_A, r_A for r_B, and q_B for q_A.

The factors e_A, r_A, etc., may be taken from Table 2, footnote (2) below after the ratio F_AA/F_BB or F_BB/F_AA on which they depend has been determined by the aid of Table 1, footnote (2); or the several factors may be computed from the formulas which appear in the footnote (2).

(*d*) *Arrangement of Reinforcement*

1. In any panel, the reinforcement per unit width in the long direction shall be at least one-third of that provided in the short direction.

2. The positive moment reinforcement adjacent to a continuous edge only and for a width not exceeding one-fourth of the shorter dimension of the panel may be reduced twenty-five per cent.

3. At a non-continuous edge negative moment reinforcement per unit width in amount at least as great as one-half of that required for maximum positive moment for the center one-half of the panel shall be provided across the entire width of the exterior support.

4. The spacing of the reinforcement shall be not more than three times the slab thickness and the ratio of reinforcement shall be at least 0.0025.

710—Maximum spacing of principal slab reinforcement

(*a*) In slabs other than concrete joist floor construction or flat slabs, the principal reinforcement shall not be spaced farther apart than three times the slab thickness, nor shall the ratio of reinforcement be less than specified in Section 707(a).

CHAPTER 8—SHEAR AND DIAGONAL TENSION

800—

Change definition for "b" to: "Width of rectangular flexural member or width of flanges for T and I sections".

Change definition for "b'" to: "Width of web in T and I flexural members".

Line 8, change "center" to "centroid".

Line 10, delete "ultimate"; capitalize "C" in "Compressive."

800—Notation

A_v = Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement), or the total area of all bars bent up in any one plane.

α = Angle between inclined web bars and axis of beam.

b = Width of rectangular beam or width of flange of T-beam.

b' = Width of web in beams of I or T sections.
d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.
f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.
f_v = Tensile unit stress in web reinforcement.
j = Ratio of distance between centroid of compression and centroid of tension to the depth d.
s = Spacing of stirrups or of bent bars in a direction parallel to that of the main reinforcement.
t_2 = Thickness of flat slab without drop panels, or the thickness of flat slab through the drop panels where such are used.
t_3 = Thickness of flat slab (with drop panels) at points outside the drop panel.
v = Shearing unit stress.
V = Total shear.
V' = Excess of the total shear over that permitted on the concrete.

801—Shearing unit stress

(*a*) The shearing unit stress v, as a measure of diagonal tension, in reinforced concrete flexural members shall be computed by formula (12):

$$v = \frac{V}{bjd} \qquad (12)$$

(*b*) For beams of I or T section, b' shall be substituted for b in formula (12).

(*c*) In concrete joist floor construction, where burned clay or concrete tile are used, b' may be taken as a width equal to the thickness of the concrete web plus the thicknesses of the vertical shells of the concrete or burned clay tile in contact with the joist as in Section 708(b).

(*d*) When the value of the shearing unit stress computed by formula (12) exceeds the shearing unit stress v_c permitted on the concrete of an unreinforced web (see Section 305), web reinforcement shall be provided to carry the excess.

802—Types of web reinforcement

(*a*) Web reinforcement may consist of:

1. Stirrups or web reinforcement bars perpendicular to the longitudinal steel.

2. Stirrups or web reinforcement bars welded or otherwise rigidly attached to the longitudinal steel and making an angle of 30 degrees or more thereto.

3. Longitudinal bars bent so that the axis of the inclined portion of the bar makes an angle of 15 degrees or more with the axis of the longitudinal portion of the bar.

4. Special arrangements of bars with adequate provisions to prevent slip of bars or splitting of the concrete by the reinforcement (See Section 804(f)).

(*b*) Stirrups or other bars to be considered effective as web reinforcement shall be anchored at both ends, according to the provisions of Section 904.

803—Stirrups

(*a*) The area of steel required in stirrups placed perpendicular to the longitudinal reinforcement shall be computed by formula (13).

$$A_v = \frac{V's}{f_v jd} \quad \ldots\ldots (13)$$

(*b*) Inclined stirrups shall be proportioned by formula (15) (Section 804(d).)

(*c*) Stirrups placed perpendicular to the longitudinal reinforcement shall not be used alone as web reinforcement when the shearing unit stress (v) exceeds $0.08f'_c$.

804—

(e) Line 1, change "two" to "three".

804—Bent bars

(*a*) When the web reinforcement consists of a single bent bar or of a single group of bent bars the required area of such bars shall be computed by formula (14).

$$A_v = \frac{V'}{f_v \sin \alpha} \quad \ldots\ldots (14)$$

(*b*) In formula (14) V' shall not exceed $0.040\, f'_c\, bjd$.

(*c*) Only the center three-fourths of the inclined portion of such bar, or group of bars, shall be considered effective as web reinforcement.

(*d*) Where there is a series of parallel bent bars, the required area shall be determined by formula (15).

$$A_v = \frac{V's}{f_v jd\, (\sin \alpha + \cos \alpha)} \quad \ldots\ldots (15)$$

(*e*) When bent bars, having a radius of bend of not more than two times the diameter of the bar are used alone as web reinforcement, the allowable shearing unit stress shall not exceed $0.060\, f'_c$. This shearing unit stress may be increased at the rate of $0.01\, f'_c$ for each increase of four bar diameters in the radius of bend until the maximum allowable shearing unit stress is reached. (See Section 305(a).)

(*f*) The shearing unit stress permitted when special arrangements of bars are employed shall be that determined by making comparative tests, to destruction, of specimens of the proposed system and of similar specimens reinforced in conformity with the provisions of this code, the same factor of safety being applied in both cases.

805—Combined web reinforcement

(*a*) Where more than one type of reinforcement is used to reinforce the same portion of the web, the total shearing resistance of this portion of the web shall be assumed as the sum of the shearing resistances computed for the various types separately. In such computations the shearing resistance of the concrete shall be included only once, and no one type of reinforcement shall be assumed to resist more than $\frac{2V'}{3}$.

806—Spacing of web reinforcement

(*a*) Where web reinforcement is required it shall be so spaced that every 45 degree line (representing a potential crack) extending from the mid-depth of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement. If a shearing unit stress in excess of $0.06\ f'_c$ is used, every such line shall be crossed by at least two such lines of web reinforcement.

807—Shearing stress in flat slabs

(*a*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance $t_2 - 1\frac{1}{2}$ in. beyond the edge of the column capital and parallel or concentric with it, shall not exceed the following values when computed by formula (12) (in which d shall be taken as $t_2 - 1\frac{1}{2}$ in.):

1. $0.03\ f'_c$, when at least 50 per cent of the total negative reinforcement in the column strip passes directly over the column capital.

2. $0.025\ f'_c$, when 25 per cent or less of the total negative reinforcement in the column strip passes directly over the column capital.

3. For intermediate percentages, intermediate values of the shearing unit stress shall be used.

(*b*) In flat slabs, the shearing unit stress on a vertical section which lies at a distance of $t_3 - 1\frac{1}{2}$ in. beyond the edge of the drop panel and parallel with it shall not exceed $0.03\ f'_c$ when computed by formula (12) (in which d shall be taken as $t_3 - 1\frac{1}{2}$ in.). At least 50 per cent of the cross-sectional area of the negative reinforcement in the column strip must be within the width of strip directly above the drop panel.

808—Shear and diagonal tension in footings

(*a*) In isolated footings the shearing unit stress computed by formula (12) on the critical section (see 1205(a)), shall not exceed $0.03\,f'_c$, nor in any case shall it exceed 75 p.s.i.

CHAPTER 9—BOND AND ANCHORAGE

900—

Line 1, change "center" to "centroid".

Line 3, delete "ultimate"; capitalize "C" in "compressive."

900—Notation

d = Depth from compression face of beam or slab to center of longitudinal tensile reinforcement.

f'_c = Ultimate compressive strength of concrete at age of 28 days unless otherwise specified.

j = Ratio of distance between centroid of compression and centroid of tension to the depth d.

Σo = Sum of perimeters of bars in one set.

u = Bond stress per unit of surface area of bar.

V = Total shear.

901—Computation of bond stress in beams

(*a*) In flexural members in which the tensile reinforcement is parallel to the compression face, the bond stress at any cross section shall be computed by formula (16).

$$u = \frac{V}{\Sigma o\, jd} \qquad (16)$$

in which V is the shear at that section.

(*b*) Adequate end anchorage shall be provided for the tensile reinforcement in all flexural members to which formula (16) does not apply, such as footings, brackets and other tapered or stepped beams in which the tensile reinforcement is not parallel to the compression face.

902—

Change section to:

(a) Tensile negative reinforcement in any span of a continuous, restrained or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar, whether required for positive or negative reinforcement, shall be extended at least twelve diameters beyond the point at which it is no longer needed to resist stress. In cases where the length from the point of maximum tensile stress in the bar to the end of the bar is not sufficient to develop this maximum stress by bond alone, the bar shall be extended to such a point that with the addition of a standard hook (see Section 906(c)), the maximum tensile unit stress can be developed. If preferred, the bar may be bent across the web at an angle of not less than 15 degrees with the longitudinal portion

of the bar and made continuous with the reinforcement which resists moment of opposite sign.

(b) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of ten or more bar diameters. Where extension of the reinforcement into the support a distance of ten or more bar diameters is impracticable the bars shall be extended as far as possible into the support and terminated in standard hooks or other adequate anchorage.

(c) In simple beams, or at the outer or freely supported ends of end spans of continuous beams, at least one-half the positive reinforcement shall extend along the same face of the beam into the support a distance of twelve or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.

902—Ordinary anchorage requirements

(*a*) Tensile negative reinforcement in any span of a continuous, restrained, or cantilever beam, or in any member of a rigid frame shall be adequately anchored by bond, hooks or mechanical anchors in or through the supporting member. Within any such span every reinforcing bar shall be extended at least twelve diameters beyond the point at which it is no longer needed to resist stress. In cases where the length from the point of maximum tensile stress in the bar to the end of the bar is not sufficient to develop this maximum stress by bond, the bar shall extend into a region of compression and be anchored by means of a standard hook or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and either made continuous with the positive reinforcement or anchored in a region of compression.

(*b*) Of the positive reinforcement in continuous beams not less than one-fourth the area shall extend along the same face of the beam into the support a distance of ten or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks, or other adequate anchorage.

(*c*) In simple beams, or at the outer or freely supported ends of end spans of continuous beams, at least one-half the positive reinforcement shall extend along the same face of the beam into the support a distance of twelve or more bar diameters, or shall be extended as far as possible into the support and terminated in standard hooks.

903—

Change section to:

(a) Where increased shearing or bond stresses are permitted because of the use of special anchorage (see Section 305), every bar except those specifically mentioned in Section 902(b), shall be terminated in a standard hook in a region of compression, or shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the reinforcement resisting moment of opposite sign.

903—Special anchorage requirements

(*a*) Where increased shearing or bond stresses are permitted because of the use of special anchorage (See Section 305), every bar shall be terminated in a standard hook in a region of compression, or it shall be bent across the web at an angle of not less than 15 degrees with the longitudinal portion of the bar and made continuous with the negative or positive reinforcement.

904—Anchorage of web reinforcement

(*a*) Single separate bars used as web reinforcement shall be anchored at each end by one of the following methods:

1. Welding to longitudinal reinforcement.
2. Hooking tightly around the longitudinal reinforcement through 180 degrees.
3. Embedment above or below the mid-depth of the beam on the compression side, a distance sufficient to develop the stress to which the bar will be subjected at a bond stress of not to exceed $.04\ f'_c$ on plain bars nor $.05\ f'_c$ on deformed bars.
4. Standard hook (see Section 906(a)), considered as developing 10,000 p.s.i., plus embedment sufficient to develop by bond the remainder of the stress to which the bar is subjected. The unit bond stress shall not exceed that specified in Table 305(a). The effective embedded length shall not be assumed to exceed the distance between the mid-depth of the beam and the tangent of the hook.

(*b*) The extreme ends of bars forming simple U or multiple stirrups shall be anchored by one of the methods of Section 904(a) or shall be bent through an angle of at least 90 degrees tightly around a longitudinal reinforcing bar not less in diameter than the stirrup bar, and shall project beyond the bend at least twelve diameters of the stirrup bar.

(*c*) The loops or closed ends of such stirrups shall be anchored by bending around the longitudinal reinforcement through an angle of at least 90 degrees, or by being welded or otherwise rigidly attached thereto.

(*d*) Hooking or bending stirrups or separate web reinforcement bars around the longitudinal reinforcement shall be considered effective only when these bars are perpendicular to the longitudinal reinforcement.

(*e*) Longitudinal bars bent to act as web reinforcement shall, in a region of tension, be continuous with the longitudinal reinforcement. The tensile stress in each bar shall be fully developed in both the upper and the lower half of the beam by one of the following methods:

1. As specified in Section 904(a), (3).
2. As specified in Section 904(a), (4).
3. By bond, at a unit bond stress not exceeding $.04\ f'_c$ on plain bars nor $.05\ f'_c$ on deformed bars, plus a bend of radius not less than

two times the diameter of the bar, parallel to the upper or lower surface of the beam, plus an extension of the bar of not less than twelve diameters of the bar terminating in a standard hook. This short radius bend extension and hook shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 p.s.i.

4. By bond, at a unit bond stress not exceeding .04 f'_c on plain bars nor .05 f'_c on deformed bars, plus a bend of radius not less than two times the diameter of the bar, parallel to the upper or lower surface of the beam and continuous with the longitudinal reinforcement. The short radius bend and continuity shall together not be counted upon to develop a tensile unit stress in the bar of more than 10,000 p.s.i.

5. The tensile unit stress at the beginning of a bend may be increased from 10,000 p.s.i. when the radius of bend is two bar diameters, at the rate of 1,000 p.s.i. tension for each increase of 1½ bar diameters in the radius of bend, provided that the length of the bar in the bend and extension is sufficient to develop this increased tensile stress by bond at the unit stresses given in Section 904(e), (3).

(*f*) In all cases web reinforcement shall be carried as close to the compression surface of the beam as fireproofing regulations and the proximity of other steel will permit.

905—Anchorage of bars in footing slabs

(*a*) All bars in footing slabs shall be anchored by means of standard hooks. The outer faces of these hooks shall be not less than three inches nor more than six inches from the face of the footing.

906—Hooks

(*a*) The terms "hook" or "standard hook" as used herein shall mean either

1. A complete semicircular turn with a radius of bend on the axis of the bar of not less than three and not more than six bar diameters, plus an extension of at least four bar diameters at the free end of the bar, or

2. A 90° bend having a radius of not less than four bar diameters plus an extension of twelve bar diameters.

Hooks having a radius of bend of more than six bar diameters shall be considered merely as extensions to the bars, and shall be treated as in section 904 (e), (5).

(*b*) In general, hooks shall not be permitted in the tension portion of any beam except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

(*c*) No hook shall be assumed to carry a load which would produce a tensile stress in the bar greater than 10,000 p.s.i.

(*d*) Hooks shall not be considered effective in adding to the compressive resistance of bars.

(*e*) Any mechanical device capable of developing the strength of the bar without damage to the concrete may be used in lieu of a hook. Tests must be presented to show the adequacy of such devices.

CHAPTER 10—FLAT SLABS—WITH SQUARE OR RECTANGULAR PANELS

1000—Notation

A = The distance from the center line of the column, in the direction of any span, to the intersection of a 45-degree diagonal line from the center of the column to the bottom of the flat slab or drop panel, where such line lies wholly within the column, capital, or bracket, provided such capital or bracket is structurally capable of resisting shears and moments without excessive unit stress. In no case shall A be greater than one-eighth the span in the direction considered.

A_{av} = Average of the two values of A for the two columns at the ends of a column strip, in the direction of the spans considered.

c = Diameter or width of column capital at the under side of the slab or drop panel. No portion of the column capital shall be considered for structural purposes which lies outside the largest right circular cone, with 90 degrees vertex angle, that can be included within the outlines of the column capital.

L = Span length of slab center to center of columns in the direction of which bending is considered.

M_o = Sum of the positive and the average negative bending moments at the critical design sections of a flat slab panel. See Section 1003(b).

W = Total dead and live load uniformly distributed over a single panel area.

W_{av} = The average of the total load on two adjacent panels.

x = Coefficient of span L which gives the distance from the center of column to the critical section for negative bending in design according to Section 1002(a).

1001—Scope

(*a*) The term flat slab shall mean a reinforced concrete slab supported by columns with or without flaring heads or column capitals,

with or without depressed or drop panels and generally without beams or girders.

(*b*) Recesses or pockets in flat slab ceilings, located between reinforcing bars and forming cellular or two-way ribbed ceilings, whether left open or filled with permanent fillers, shall not prevent a slab from being considered a flat slab; but allowable unit stresses shall not be exceeded.

(*c*) This chapter provides for two methods of design of flat slab structures.

1. Any type of flat slab construction may be designed by application of the principles of continuity, using the method outlined in Section 1002, or using other recognized methods of elastic analysis. In either case, the design must be subject to the provisions of Sections 1005(a) and (c), 1006, 1008 and 1009.

2. The common cases of flat slab construction described in Section 1003 may be designed by the use of moment coefficients, given in Sections 1003 and 1004, and subject to the provisions of Sections 1005, 1006, 1007, 1008 and 1009.

1002—

(a) Sub-paragraph 10—Lines 6 and 7, delete, "in the manner provided in Chapter 11."

1002—Design of flat slabs as continuous frames

(*a*) Except in the cases of flat slab construction where specified coefficients for bending may be used, as provided in Section 1003, bending and shear in flat slabs and their supports shall be determined by an analysis of the structure as a continuous frame, and all sections shall be proportioned to resist the moments and shears thus obtained. In the analysis, the following assumptions may be made:

1. The structure may be considered divided into a number of bents, each consisting of a row of columns and strips of supported slabs, each strip bounded laterally by the center line of the panel on either side of the row of columns. The bents shall be taken longitudinally and transversely of the building.

2. Each such bent may be analyzed in its entirety; or each floor thereof and the roof may be analyzed separately with its adjacent columns above and below, the columns being assumed fixed at their remote ends. Where slabs are thus analyzed separately, in bents more than four panels long, it may be assumed in determining the bending at a given support that the slab is fixed at any support two panels distant therefrom beyond which the slab continues.

3. The joints between columns and slabs may be considered rigid and this rigidity may be assumed to extend in the slabs a dis-

tance A from the center of the columns, and in the column to the intersection of the sides of the column and the 45 degree line defining A. The change in length of columns and slabs due to direct stress, and deflections due to shear, may be neglected. Where metal column capitals are used, account may be taken of their contributions to stiffness and resistance to bending and shear.

4. The supporting columns may be assumed free from settlement or lateral movement unless the amount thereof can be reasonably determined.

5. The moment of inertia of slab or column at any cross-section may be assumed to be that of the gross section of the concrete. Variation in the moments of inertia of the slabs and columns along their axes shall be taken into account.

6. Where the load to be supported is definitely known, the structure shall be analyzed for that load. Where the live load is variable but does not exceed three-quarters of the dead load, or the nature of the live load is such that all panels will be loaded simultaneously, the maximum bending may be assumed to obtain at all sections under full live load. Elsewhere, maximum positive bending near mid-span of a panel may be assumed to obtain under full live load in the panel and in alternate panels; and maximum negative bending at a support may be assumed to obtain under full live load in the adjacent panels only.

7. Where neither beams nor girders help to transfer the slab load to the supporting column, the critical section for negative bending may be assumed as not more than the distance xL from the column center, where

$$x = 0.073 + 0.57 \frac{A}{L} \qquad (17)$$

In slabs supported by beams, girders, or walls, the critical section for negative bending shall be assumed at the face of such support.

8. The numerical sum of the maximum positive and the average maximum negative bending moments for which provision is made in the design in the direction of either side of a rectangular panel shall be assumed as not less than

$$\frac{1}{10} W_{av} L \left(1 - \frac{4A_{av}}{3L}\right)^2 \qquad (18)$$

9. The bending at critical sections across the slabs of each bent may be apportioned between the column strip and middle strip, as defined in Section 1005, in the ratio of the specified coefficients which affect such apportionment in the special cases of flat slabs provided for in Section 1003.

10. The maximum bending in columns may be assumed to obtain under full live load in alternate panels. Columns shall be proportioned to resist the maximum bending combined with the maximum direct load consistent therewith; and for maximum direct load combined with the bending under full load, the direct load subject to allowable reductions, in the manner provided in Chapter 11. In computing moments in columns at any floor, the far ends of the columns may be considered fixed.

(*b*) The foregoing provisions outline the method to be followed in analyzing and designing flat slabs in the general case. In all instances the design must conform to the requirements for panel strips and critical design sections, slab thickness and drop panels, capitals and brackets, arrangement of reinforcement and openings in flat slabs, as provided in Sections 1005(a) and (c), 1006, 1008 and 1009.

1003—Design of flat slabs by moment coefficients

(*a*) In those cases of flat slab construction which fall within the following limitations as to continuity and dimensions, the bending moments at critical sections may be determined by the use of specified coefficients as provided in Section 1004.

1. The ratio of length to width of panel does not exceed 1.33.
2. The slab is continuous for at least three panels in each direction.
3. The successive span lengths in each direction differ by not more than twenty per cent of the shorter span.

(*b*) In such slabs, the numerical sum of the positive and negative bending moments in the direction of either side of an interior rectangular panel shall be assumed as not less than

$$M_o = 0.09\, WL \left(1 - \frac{2c}{3L}\right)^2 \qquad (19)$$

(*c*) Three-fourths of the width of the strip shall be taken as the width of the section in computing compression due to bending, except that, on a section through a drop panel, three-fourths of the width of the drop panel shall be taken. Account shall be taken of any recesses which reduce the compressive area. Tension reinforcement distributed over the entire strip shall be included in the computations.

(*d*) The design of slabs under the procedure given in this section is subject to the provisions of all subsequent sections of this chapter (Sections 1004 to 1009).

1004—Bending moment coefficients

(*a*) The bending moments at the critical sections of the middle and column strips of an interior panel shall be assumed as given in Table 1004(a).

TABLE 1004(a)—BENDING MOMENTS IN INTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Negative moment	$0.50M_o$
	Positive moment	$0.20M_o$
Middle strip	Negative moment	$0.15M_o$
	Positive moment	$0.15M_o$
Without drop panel		
Column strip	Negative moment	$0.46M_o$
	Positive moment	$0.22M_o$
Middle strip	Negative moment	$0.16M_o$
	Positive moment	$0.16M_o$

TABLE 1004(b)—BENDING MOMENTS IN EXTERIOR FLAT SLAB PANEL

With drop panel		
Column strip	Exterior negative	$0.45M_o$
	Positive moment	$0.25M_o$
	Interior negative	$0.55M_o$
Middle strip	Exterior negative	$0.10M_o$
	Positive moment	$0.19M_o$
	Interior negative	$0.165M_o$
Without drop panel		
Column strip	Exterior negative	$0.41M_o$
	Positive moment	$0.28M_o$
	Interior negative	$0.50M_o$
Middle strip	Exterior negative	$0.10M_o$
	Positive moment	$0.20M_o$
	Interior negative	$0.176M_o$

TABLE 1004(c)—BENDING MOMENTS IN PANELS WITH MARGINAL BEAMS OR WALLS

		Marginal Beams with Depth greater than 1½ times the Slab Thickness; or Bearing Wall.		Marginal Beams with depth 1½ times the Slab Thickness or less.	
(a) Load to be carried by Marginal Beam or Wall		Loads directly superimposed upon it plus a uniform load equal to one-quarter of the total live and dead panel load.		Loads directly superimposed upon it exclusive of any panel load.	
		With Drop	Without Drop	With Drop	Without Drop
(b) Moment to be used in the design of Half Column Strip adjacent and parallel to Marginal Beam or Wall.	Neg.	$0.125M_o$	$0.115M_o$	$0.25M_o$	$0.23M_o$
	Pos.	$0.05M_o$	$0.055M_o$	$0.10M_o$	$0.11M_o$
(c) Negative Moment to be used in Design of Middle Strip continuous across a Beam or Wall	Neg.	$0.195M_o$	$0.208M_o$	$0.15M_o$	$0.16M_o$

(*b*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the exterior supports consist of reinforced concrete columns or reinforced concrete bearing walls integral with the slab, the ratio of stiffness of the support to that of the slab being at least as great as the ratio of the live load to the dead load and not less than one, shall be assumed as given in Table 1004(b). Where a flat slab is so supported by a wall providing restraint at the discontinuous edge, the coefficient for negative bending at the edge shall be assumed more nearly equal in the column and middle strips, the sum remaining as given in Table 1004(b), but that for the column strip shall not be less than 0.30 M_o. Bending in middle strips parallel to a discontinuous edge, except in a corner panel, shall be assumed the same as in an interior panel. M_o shall be determined as provided in Section 1003(b) for an interior panel.

(*c*) The bending moments at critical sections of strips, in an exterior panel, at right angles to the discontinuous edge, where the exterior supports are masonry bearing walls or other construction which provide only negligible restraint to the slab, shall be assumed as given in Table 1004(b) with the following modifications.

1. On critical sections at the face of the exterior support, negative bending in each strip shall be assumed as 0.05 M_o.
2. The coefficients for positive bending shall be increased by forty per cent.
3. The coefficients for negative bending at the first interior columns shall be increased thirty per cent.

(*d*) The bending moments in panels with marginal beams or walls, in the strips parallel and close thereto, and in the beams, shall be determined upon the basis of assumptions presented in Table 1004(c).

(*e*) For design purposes any of the moment coefficients of Tables 1004(a), 1004(b), and 1004(c) may be varied by not more than six per cent, but the numerical sum of the positive and negative moments in a panel shall not be taken as less than the amount specified.

(*f*) Panels supported by marginal beams on opposite edges shall be designed as solid one or two-way slabs to carry the entire panel load.

(*g*) The ratio of reinforcement in any strip shall not be less than 0.0025.

1005—Panel strips and critical design sections

(*a*) A flat slab panel shall be considered as consisting of strips in each direction as follows:

A middle strip one half panel in width, symmetrical about panel center line and extending through the panel in the direction of the span for bending.

A column strip consisting of the two adjacent quarter-panels either side of the column center lines.

(*b*) The critical sections for bending are located as follows:

Sections for negative bending shall be taken along the edges of the panel, on column center lines between capitals and around the perimeters of column capitals.

Sections for positive bending shall be taken at mid-span of the strips.

(*c*) Only the reinforcement which crosses a critical section within a strip may be considered effective to resist bending in the strip at that section. Reinforcement which crosses such section at an angle with the center-line of the strip shall be assumed to contribute to the resistance of bending only its effective area in the direction of the strip, as defined in Chapter 1.

1006—Slab thickness and drop panels

(*a*) The thickness of a flat slab and the size and thickness of the drop panel, where used, shall be such that the compressive stress due to bending at the critical sections of any strip and the shear about the column capital and the drop panel shall not exceed the unit stresses allowed in concrete of the quality used.

(*b*) The shearing stresses in the slab outside the capital or drop panel shall be computed as provided in Section 807.

(*c*) Slab thickness shall not, however, be less than

$$\frac{L}{40} \text{ with drop panels}$$

or

$$\frac{L}{36} \text{ without drop panels}$$

(*d*) The thickness of the drop panel below the slab shall not be more than one-fourth the distance from the edge of the column capital to the edge of the drop panel.

1007—Capitals and brackets

(*a*) Where a column is without a flaring concrete capital the distance c shall be taken as the diameter of the column. Structural metal embedded in the slab or drop panel may be regarded as contributing to resistance in bending and shear.

(*b*) Where a reinforced concrete beam frames into a column without capital or bracket on the same side with the beam, the value of c may be taken as the width of the column plus twice the projection of the beam above or below the slab or drop panel for computing bending in strips parallel to the beam.

(*c*) Brackets capable of transmitting the negative bending and the shear in the column strips to the columns without excessive unit stress may be substituted for column capitals at exterior columns. The value of c where brackets are used shall be taken as twice the distance from the center of the column to a point where the bracket is 1½ inches thick, but not more than the thickness of the column plus twice the depth of the bracket.

(*d*) The average of the diameters c of the column capitals at the four corners of a panel shall be used in determining the bending in the middle strips of the panel. The average of the diameters c of the two column capitals at the ends of a column strip shall be used in determining bending in the strip.

1008—Arrangement of reinforcement

(*a*) Slab reinforcement shall be provided to resist the bending and bond stresses not only at critical sections, but also at intermediate sections.

(*b*) Bars shall be spaced evenly across strips or bands and the spacing shall not exceed three times the slab thickness.

(*c*) In exterior panels the reinforcement perpendicular to the discontinuous edge for positive bending, shall extend to the edge and have embedment of at least six inches in spandrel beams, walls or columns. All such reinforcement for negative bending shall be bent, hooked or otherwise anchored in spandrel beams, walls or columns.

1009—Openings in flat slabs

Openings of any size may be cut through a flat slab if provision is made for the total positive and negative resisting moments, as required in Sections 1002 or 1003, without exceeding the allowable stresses as given in Sections 305 and 306.

CHAPTER 11—REINFORCED CONCRETE COLUMNS AND WALLS

1100—

After f'_c delete "Ultimate"; capitalize "C" in "compressive."

After f_s change "working" to "allowable".

1100—Notation

A_c = Area of core of a spirally reinforced column measured to the outside diameter of the spiral; net area of concrete section of a composite column.

A_g = The overall or gross area of spirally reinforced or tied columns; the total area of the concrete encasement of combination columns.

A_r = Area of the steel or cast-iron core of a composite column; the area of the steel core in a combination column.

A_s = Effective cross-sectional area of reinforcement in compression in columns.

C = Ratio of allowable concrete stress, f_a, in axially loaded column to allowable fiber stress for concrete in flexure.

$D = \frac{t^2}{2R^2}$ = a factor, usually varying from 3 to 9. (The term R as used here is the radius of gyration of the entire column section.)

d = The least lateral dimension of a concrete column.

e = Eccentricity of the resultant load on a column, measured from the gravity axis.

$F = \frac{\textit{Yield point of pipe}}{45{,}000}$ (See Section 1106(b)).

f_a = Average allowable stress in the concrete of an axially loaded reinforced concrete column.

f_c = Computed concrete fiber stress in an eccentrically loaded column.

f'_c = Ultimate compressive strength of concrete at age of 28 days, unless otherwise specified.

f_p = Maximum allowable concrete fiber stress in an eccentrically loaded column.

f_r = Allowable unit stress in the metal core of a composite column.

f'_r = Allowable unit stress on unencased steel columns and pipe columns.

f_s = Nominal working stress in vertical column reinforcement.

f'_s = Useful limit stress of spiral reinforcement.

h = Unsupported length of column.

K = Least radius of gyration of a metal pipe section (in pipe columns).

$n = \frac{30{,}000}{f'_c}$

N = Axial load applied to reinforced concrete column.

p' = Ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals) of a spirally reinforced concrete column.

p_g = Ratio of the effective cross-sectional area of vertical reinforcement to the gross area A_g.

P = Total allowable axial load on a column whose length does not exceed ten times its least cross-sectional dimension.

P' = Total allowable axial load on a long column.

R = Least radius of gyration of a section.
t = Overall depth of column section.

1101—Limiting dimensions

(*a*) The following sections on reinforced concrete and composite columns, except Section 1107(a), apply to a short column for which the unsupported length is not greater than ten times the least dimension. When the unsupported length exceeds this value, the design shall be modified as shown in Section 1107(a). Principal columns in buildings shall have a minimum diameter of twelve inches, or in the case of rectangular columns, a minimum thickness of ten inches, and a minimum gross area of 120 sq. in. Posts that are not continuous from story to story shall have a minimum diameter or thickness of six inches.

1102—

(a) 1. Line 2, after "capital" add, "of the drop panel or of the slab, whichever is least".

1102—Unsupported length of columns

(*a*) For purposes of determining the limiting dimensions of columns, the unsupported length of reinforced concrete columns shall be taken as the clear distance between floor slabs, except that

1. In flat slab construction, it shall be the clear distance between the floor and the lower extremity of the capital.

2. In beam and slab construction, it shall be the clear distance between the floor and the under side of the deeper beam framing into the column in each direction at the next higher floor level.

3. In columns restrained laterally by struts, it shall be the clear distance between consecutive struts in each vertical plane; provided that to be an adequate support, two such struts shall meet the column at approximately the same level, and the angle between vertical planes through the struts shall not vary more than 15 degrees from a right angle. Such struts shall be of adequate dimensions and anchorage to restrain the column against lateral deflection.

4. In columns restrained laterally by struts or beams, with brackets used at the junction, it shall be the clear distance between the floor and the lower edge of the bracket, provided that the bracket width equals that of the beam or strut and is at least half that of the column.

(*b*) For rectangular columns, that length shall be considered which produces the greatest ratio of length to depth of section.

1103—

(a) Line 8, change "working" to "allowable".
Footnote; line 1, change "working" to "allowable".

1103—Spirally reinforced columns

(*a*) *Allowable Load*—The maximum allowable axial load, P, on columns with closely spaced spirals enclosing a circular concrete core reinforced with longitudinal bars shall be that given by Formula (20).

$$P = A_g (0.225 f'_c + f_s p_g) \quad (20)$$

Wherein

A_g = the gross area of the column

f'_c = compressive strength of the concrete

f_s = nominal working stress in vertical column reinforcement, to be taken at forty per cent of the minimum specification value of the yield point; viz., 16,000 p.s.i. for intermediate grade steel and 20,000 p.s.i. for rail or hard grade steel.*

p_g = ratio of the effective cross-sectional area of vertical reinforcement to the gross area, A_g.

(*b*) *Vertical Reinforcement*—The ratio p_g shall not be less than 0.01 nor more than 0.08. The minimum number of bars shall be six, and the minimum diameter shall be ⅝ in. The center to center spacing of bars within the periphery of the column core shall not be less than 2½ times the diameter for round bars or three times the side dimension for square bars. The clear spacing between bars shall not be less than 1½ inches or 1½ times the maximum size of the coarse aggregate used. These spacing rules also apply to adjacent pairs of bars at a lapped splice; each pair of lapped bars forming a splice may be in contact, but the minimum clear spacing between one splice and the adjacent splice should be that specified for adjacent single bars.

(*c*) *Splices in Vertical Reinforcement*—Where lapped splices in the column verticals are used, the minimum amount of lap shall be as follows:

1103—
(c) 1. Line 5, change "working" to "allowable".

1. For deformed bars—with concrete having a strength of 3000 p.s.i. or above, twenty-four diameters of bar of intermediate grade steel and thirty diameters of bar of hard grade steel. For bars of higher yield point, the amount of lap shall be increased in proportion to the nominal working stress. When the concrete strengths are less than 3000 p.s.i., the amount of lap shall be one-third greater than the values given above.

2. For plain bars—the minimum amount of lap shall be twenty-five per cent greater than that specified for deformed bars.

*Nominal working stresses for reinforcement of higher yield point may be established at forty percent of the yield point stress, but not more than 30,000 psi., when the properties of such reinforcing steels have been definitely specified by standards of A.S.T.M. designation. If this is done, the lengths of splice required by Section 1103 (c) shall be increased accordingly.

3. Welded splices or other positive connections may be used instead of lapped splices. Welded splices shall preferably be used in cases where the bar diameter exceeds 1¼ in. An approved welded splice shall be defined as one in which the bars are butted and welded and that will develop in tension at least the yield point stress of the reinforcing steel used.

Change sub-paragraph 4 to:

"Where longitudinal bars are offset at a splice, the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column. Adequate horizontal support at the offset bends shall be treated as a matter of design, and may be provided by metal ties, spirals or parts of the floor construction. Metal ties or spirals so designed shall be placed near (never more than 8 bar diameters from) the point of bend. The horizontal thrust to be resisted may be assumed as 1½ times the horizontal component of the nominal stress in the inclined portion of the bar.

Offset bars shall be bent before they are placed in the forms. No field bending of bars partially embedded in concrete shall be permitted."

4. Where changes in the cross section of a column occur, the longitudinal bars shall be offset in a region where lateral support is afforded by a concrete capital, floor slab or by metal ties or reinforcing spirals. Where bars are offset, the slope of the inclined portion from the axis of the column shall not exceed 1 in 6 and the bars above and below the offset shall be parallel to the axis of the column.

1103—

(d) Lines 11 to 13, change "The spiral . . . spacer bars" to "The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by vertical spacers, using at least two for spirals 20 in. or less in diameter, three for spirals 20 to 30 in. in diameter and four for spirals more than 30 in. in diameter, or composed of spiral rods ⅝-in. or larger in size."

(*d*) *Spiral Reinforcement*—The ratio of spiral reinforcement, p', shall not be less than the value given by Formula (21).

$$p' = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f'_s} \qquad (21)$$

Wherein

p' = ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).

f'_s = useful limit stress of spiral reinforcement, to be taken as 40,000 p.s.i. for hot rolled rods of intermediate grade, 50,000 p.s.i. for rods of hard grade, and 60,000 p.s.i. for cold drawn wire.

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical

spacer bars. The spirals shall be of such size and so assembled as to permit handling and placing without being distorted from the designed dimensions. The material used in spirals shall have a minimum diameter of ¼ in. for rolled bars or No. 4 W. & M. gage for drawn wire. Anchorage of spiral reinforcement shall be provided by 1½ extra turns of spiral rod or wire at each end of the spiral unit. Splices, when necessary shall be made in spiral rod or wire by welding or by a lap of 1½ turns. The center to center spacing of the spirals shall not exceed one-sixth of the core diameter. The clear spacing between spirals shall not exceed 3 in. nor be less than 1⅜ in. or 1½ times the maximum size of coarse aggregate used. The reinforcing spiral shall extend from the floor level in any story or from the top of the footing in the basement, to the level of the lowest horizontal reinforcement in the slab, drop panel or beam above. In a column with a capital, it shall extend to a plane at which the diameter or width of the capital is twice that of the column.

(*e*) *Protection of Reinforcement*—The column reinforcement shall be protected everywhere by a covering of concrete cast monolithically with the core, for which the thickness shall not be less than 1½ in. nor less than 1½ times the maximum size of the coarse aggregate, nor shall it be less than required by the fire protection and weathering provisions of Section 507.

1103—

(f) Line 3, transpose "of which" and "the sides".

(*f*) *Isolated Column with Multiple Spirals*—In case two or more interlocking spirals are used in a column, the outer boundary of the column shall be taken as a rectangle of which the sides are outside the extreme limits of the spiral at a distance equal to the requirements of Section 1103(*e*).

(*g*) *Limits of Section of Column Built Monolithically with Wall*—For a spiral column built monolithically with a concrete wall or pier, the outer boundary of the column section shall be taken either as a circle at least 1½ in. outside the column spiral or as a square or rectangle of which the sides are at least 1½ in. outside the spiral or spirals.

(*h*) *Equivalent Circular Columns*—As an exception to the general procedure of utilizing the full gross area of the column section, it shall be permissible to design a circular column and to build it with a square, octagonal, or other shaped section of the same least lateral dimension. In such case, the allowable load, the gross area considered, and the required percentages of reinforcement shall be taken as those of the circular column.

1104—Tied columns

(*a*) *Allowable Load*—The maximum allowable axial load on columns reinforced with longitudinal bars and separate lateral ties shall be 80

per cent of that given by Formula (20). The ratio, p_g, to be considered in tied columns shall not be less than 0.01 nor more than 0.04. The longitudinal reinforcement shall consist of at least four bars, of minimum diameter of ⅝ inch. Splices in reinforcing bars shall be made as described in Section 1103 (c).

(*b*) *Lateral Ties*—Lateral ties shall be at least ¼ in. in diameter and shall be spaced apart not over 16 bar diameters, 48 tie diameters or the least dimension of the column. When there are more than four vertical bars, additional ties shall be provided so that every longitudinal bar is held firmly in its designed position and has lateral support equivalent to that provided by a 90-degree corner of a tie.

(*c*) *Limits of Column Section*—In a tied column which for architectural reasons has a larger cross section than required by considerations of loading, a reduced effective area, A_g, not less than one-half of the total area may be used in applying the provisions of Section 1104 (a).

1105—
(b) Line 7, change "and" to ","; add "and (e)".

1105—Composite columns

(*a*) *Allowable Load*—The allowable load on a composite column, consisting of a structural steel or cast-iron column thoroughly encased in concrete reinforced with both longitudinal and spiral reinforcement, shall not exceed that given by Formula (22).

$$P = 0.225\, A_c f'_c + f_s A_s + f_r A_r \qquad (22)$$

Wherein A_c = net area of concrete section
$= A_g - A_s - A_r$
A_s = cross-sectional area of longitudinal bar reinforcement.
A_r = cross-sectional area of the steel or cast-iron core.
f_r = allowable unit stress in metal core, not to exceed 16,000 p.s.i. for a steel core; or 10,000 p.s.i. for a cast-iron core.

The remaining notation is that of Section 1103.

(*b*) *Details of Metal Core and Reinforcement*—The cross-sectional area of the metal core shall not exceed 20 per cent of the gross area of the column. If a hollow metal core is used it shall be filled with concrete. The amounts of longitudinal and spiral reinforcement and the requirements as to spacing of bars, details of splices and thickness of protective shell outside the spiral shall conform to the limiting values specified in Section 1103 (b), (c) and (d). A clearance of at least three inches shall be maintained between the spiral and the metal core at all points except that when the core consists of a structural steel H-column, the minimum clearance may be reduced to two inches.

(*c*) *Splices and Connections of Metal Cores*—Metal cores in composite columns shall be accurately milled at splices and positive provision shall be made for alignment of one core above another. At the column base, provision shall be made to transfer the load to the footing at safe unit stresses in accordance with Section 305 (a). The base of the metal section shall be designed to transfer the load from the entire composite column to the footing, or it may be designed to transfer the load from the metal section only, provided it is so placed in the pier or pedestal as to leave ample section of concrete above the base for the transfer of load from the reinforced concrete section of the column by means of bond on the vertical reinforcement and by direct compression on the concrete. Transfer of loads to the metal core shall be provided for by the use of bearing members such as billets, brackets or other positive connections; these shall be provided at the top of the metal core and at intermediate floor levels where required. The column as a whole shall satisfy the requirements of Formula (22) at any point; in addition to this, the reinforced concrete portion shall be designed to carry, in accordance with Formula (20), all floor loads brought onto the column at levels between the metal brackets or connections. In applying Formula (20), the value of A_g shall be interpreted as the area of the concrete section outside the metal core, and the allowable load on the reinforced concrete section shall be further limited to $0.35\, f'_c A_g$. Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams or girders.

(*d*) *Allowable Load on Metal Core Only*—The metal cores of composite columns shall be designed to carry safely any construction or other loads to be placed upon them prior to their encasement in concrete.

1106—Combination columns

(*a*) *Steel Columns Encased in Concrete*—The allowable load on a structural steel column which is encased in concrete at least 2½ inches thick over all metal (except rivet heads) reinforced as hereinafter specified, shall be computed by Formula (23).

$$P = A_r f'_r \left[1 + \frac{A_g}{100\, A_r} \right] \qquad (23)$$

Wherein A_r = cross-sectional area of steel column.
f'_r = allowable stress for unencased steel column.
A_g = total area of concrete section.

The concrete used shall develop a compressive strength, f'_c, of at least 2000 p.s.i. at 28 days. The concrete shall be reinforced by the equivalent of welded wire mesh having wires of No. 10 W. and M. gage, the wires encircling the column being spaced not more than four inches apart and those parallel to the column axis not more than eight inches apart. This

mesh shall extend entirely around the column at a distance of one inch inside the outer concrete surface and shall be lap-spliced at least forty wire diameters and wired at the splice. Special brackets shall be used to receive the entire floor load at each floor level. The steel column shall be designed to carry safely any construction or other loads to be placed upon it prior to its encasement in concrete.

(*b*) *Pipe Columns*—The allowable load on columns consisting of steel pipe filled with concrete shall be determined by Formula (24).

$$P = 0.225 f'_c A_c + f'_r A_r \qquad (24)$$

The value of f'_r shall be given by Formula (25).

$$f'_r = \left[18{,}000 - 70 \frac{h}{K}\right] F \qquad (25)$$

Wherein f'_r = allowable unit stress in metal pipe.
h = unsupported length of column
K = least radius of gyration of metal pipe section.
$F = \dfrac{\textit{yield point of pipe.}}{45{,}000}$

If the yield point of the pipe is not known, the factor F shall be taken as 0.5.

1107—
(a) Line 2, change "a" to "an unsupported".
Lines 5 and 6, change "Formulas (20) and (22)" to "sections 1103, 1104 and 1105".

1107—Long columns

(*a*) The maximum allowable load, P', on axially loaded reinforced concrete or composite columns having a length, h, greater than ten times the least lateral dimension, d, shall be given by Formula (26).

$$P' = P \left[1.3 - .03 \frac{h}{d}\right] \qquad (26)$$

where P is the allowable axial load on a short column as given by Formulas (20) and (22).

The maximum allowable load, P', on eccentrically loaded columns in which $\frac{h}{d}$ exceeds ten shall also be given by Formula (26), in which P is the allowable eccentrically applied load on a short column as determined by the provisions of Sections 1109 and 1110. In long columns subjected to definite bending stresses, as determined in Section 1108, the ratio $\frac{h}{d}$ shall not exceed twenty.

1108—

Change fourth sentence, "Wall columns . . . floor level" to "In computing moments in columns, the far ends may be considered fixed. Columns shall be designed to resist the axial forces from loads on all floors, plus the maximum bending due to loads on a single adjacent span of the floor under consideration".

1108—Bending moments in columns

(*a*) The bending moments in the columns of all reinforced concrete structures shall be determined on the basis of loading conditions and restraint and shall be provided for in the design. When the stiffness and strength of the columns are utilized to reduce moments in beams, girders, or slabs, as in the case of rigid frames, or in other forms of continuous construction wherein column moments are unavoidable, they shall be provided for in the design. In building frames, particular attention shall be given to the effect of unbalanced floor loads on both exterior and interior columns and of eccentric loading due to other causes. Wall columns shall be designed to resist moments produced by

1. Loads on all floors of the building
2. Loads on a single exterior bay at two adjacent floor levels, or
3. Loads on a single exterior bay at one floor level

Resistance to bending moments at any floor level shall be provided by distributing the moment between the columns immediately above and below the given floor in proportion to their relative stiffnesses and conditions of restraint.

1109—Determination of combined axial and bending stresses

(*a*) In a reinforced concrete column, designed by the methods of this Chapter, which is (1) symmetrical about two perpendicular planes through its axis and (2) subject to an axial load, N, combined with bending in one or both of the planes of symmetry (but with the ratio of eccentricity to depth, e/t, no greater than 1.0 in either plane), the combined fiber stress in compression may be computed on the basis of recognized theory applying to uncracked sections, using Formula 27.

$$f_c = \frac{N}{A_g}\left[\frac{1 + \frac{De}{t}}{1 + (n - 1)p_g}\right] \qquad (27)$$

Equating this calculated stress, f_c, to the allowable stress, f_p, in Formula 29, it follows that the column can be designed for an equivalent axial load, P, as given by Formula 28.*

$$P = N\left[1 + \frac{CDe}{t}\right] \qquad (28)$$

*For approximate or trial computations, D may be taken as eight for a circular spiral column and as five for a rectangular tied or spiral column.

When bending exists on both axes of symmetry, the quantity $\frac{De}{t}$ is to be computed as the numerical sum of the $\frac{De}{t}$ quantities in the two directions.

(*b*) For columns in which the load, N, has an eccentricity, e, greater than the column depth, t, or for beams subject to small axial loads, the determination of the fiber stress f_c shall be made by use of recognized theory for cracked sections, based on the assumption that no tension exists in the concrete. For such cases the tensile steel stress shall also be investigated.

1110—
Line 8, in the expression for "f_a" change "p" in the denominator to "p_g".

1110—Allowable combined axial and bending stress

(*a*) For spiral and tied columns, eccentrically loaded or otherwise subjected to combined axial compression and flexural stress, the maximum allowable compressive stress, f_p, is given by Formula (29).

$$f_p = f_a \left[\frac{1 + \frac{De}{t}}{1 + \frac{CDe}{t}} \right] = f_a \left[\frac{t + De}{t + CDe} \right] \quad \text{(29)}$$

Wherein the notation is that of Section 1103 and 1109, and, in addition f_a is the average allowable stress in the concrete of an axially loaded reinforced concrete column, and C is the ratio of f_a to the allowable fiber stress for members in flexure. Thus $f_a = \frac{0.225\, f'_c + f_s p_g}{1 + (n - 1)p}$ for spiral columns and 0.8 of this value for tied columns, and $C = \frac{f_a}{0.45\, f'_c}$.

1111—Wind stresses

(*a*) When the allowable stress in columns is modified to provide for combined axial load and bending, and the stress due to wind loads is also added, the total shall still come within the allowable values specified for wind loads in Section 603 (c).

1112—
(a) Lines 1 and 7, delete "working".

1112—Reinforced concrete walls

(*a*) The allowable working stresses in reinforced concrete bearing walls with minimum reinforcement as required by Section 1112(i),

shall be $0.25f'_c$ for walls having a ratio of height to thickness of ten or less, and shall be reduced proportionally to $0.15f'_c$ for walls having a ratio of height to thickness of twenty-five. When the reinforcement in bearing walls is designed, placed and anchored in position as for tied columns, the allowable working stresses shall be on the basis of Section 1104, as for columns. In the case of concentrated loads, the length of the wall to be considered as effective for each shall not exceed the center to center distance between loads, nor shall it exceed the width of the bearing plus four times the wall thickness. The ratio p_g shall not exceed 0.04.

(*b*) Walls shall be designed for any lateral or other pressure to which they are subjected. Proper provision shall be made for eccentric loads and wind stresses. In such designs the allowable stresses shall be as given in Section 305(a) and 603(c).

(*c*) Panel and enclosure walls of reinforced concrete shall have a thickness of not less than five inches and not less than one thirtieth the distance between the supporting or enclosing members.

(*d*) Bearing walls of reinforced concrete in building of fire-resistive construction shall be not less than six inches in thickness for the uppermost fifteen feet of their height; and for each successive twenty-five feet downward, or fraction thereof, the minimum thickness shall be increased one inch. In two story dwellings the walls may be six inches in thickness throughout.

(*e*) In buildings of non-fire-resistive construction bearing walls of reinforced concrete shall not be less than one and one-third times the thickness required for buildings of fire-resistive construction, except that for dwellings of two stories or less in height the thickness of walls may be the same as specified for buildings of fire-resistive construction.

(*f*) Exterior basement walls, foundation walls, fire walls and party walls shall not be less than eight inches thick whether reinforced or not.

(*g*) Reinforced concrete bearing walls shall have a thickness of at least one twenty-fifth of the unsupported height or width, whichever is the shorter; provided however, that approved buttresses, built-in columns, or piers designed to carry all the vertical loads, may be used in lieu of increased thickness.

(*h*) Reinforced concrete walls shall be anchored to the floors, columns, pilasters, buttresses and intersecting walls with reinforcement at least equivalent to three-eighths inch round bars twelve inches on centers, for each layer of wall reinforcement.

(*i*) Reinforced concrete walls shall be reinforced with an area of steel in each direction, both vertical and horizontal, at least equal to 0.0025 times the cross-sectional area of the wall, if of bars, and 0.0018 times the

area if of electrically welded wire fabric.* The wire of the welded fabric shall be of not less than No. 10 W. & M. gage. Walls more than ten inches in thickness shall have the reinforcement for each direction placed in two layers parallel with the faces of the wall. One layer consisting of not less than one-half and not more than two-thirds the total required shall be placed not less than two inches nor more than one-third the thickness of the wall from the exterior surface. The other layer, comprising the balance of the required reinforcement, shall be placed not less than three-fourths inches and not more than one-third the thickness of the wall from the interior surface. Bars, if used, shall not be less than the equivalent of three-eighths inch round bars, nor shall they be spaced more than eighteen inches on centers. Welded wire* reinforcement for walls shall be in flat sheet form.

(*j*) In addition to the minimum as prescribed in 1112(i) there shall be not less than two five-eighths inch diameter bars around all window or door openings. Such bars shall extend at least twenty-four inches beyond the corner of the openings.

(*k*) Where reinforced concrete bearing walls consist of studs or ribs tied together by reinforced concrete members at each floor level, the studs may be considered as columns, but the restrictions as to minimum diameter or thickness of columns shall not apply.

CHAPTER 12—FOOTINGS

1201—Scope

(*a*) The requirements prescribed in Sections 1202 to 1209 apply only to isolated footings.†

1202—Loads and reactions

(*a*) Footings shall be proportioned to sustain the applied loads and induced reactions without exceeding the allowable stresses as prescribed in Sections 305 and 306, and as further provided in Sections 1205, 1206 and 1207.

(*b*) In cases where the footing is concentrically loaded and the member being supported does not transmit any moment to the footing, computations for moments and shears shall be based on an upward reaction assumed to be uniformly distributed per unit area or per pile and a downward applied load assumed to be uniformly distributed over the area of the footing covered by the column, pedestal, wall, or metallic column base.

(*c*) In cases where the footing is eccentrically loaded and/or the member being supported transmits a moment to the footing, proper

*Expanded metal has been omitted until a specification can be formulated.

†The committee is not prepared at this time to make recommendations for combined footings—those supporting more than one column or wall.

allowance shall be made for any variation that may exist in the intensities of reaction and applied load consistent with the magnitude of the applied load and the amount of its actual or virtual eccentricity.

(*d*) In the case of footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at the center of the pile.

1203—Sloped or stepped footings

(*a*) In sloped or stepped footings, the angle of slope or depth and location of steps shall be such that the allowable stresses are not exceeded at any section.

(*b*) In sloped or stepped footings, the effective cross-section in compression shall be limited by the area above the neutral plane.

(*c*) Sloped or stepped footings shall be cast as a unit.

1204—Bending moment

(*a*) The external moment on any section shall be determined by passing through the section a vertical plane which extends completely across the footing, and computing the moment of the forces acting over the entire area of the footing on one side of said plane.

(*b*) The greatest bending moment to be used in the design of an isolated footing shall be the moment computed in the manner prescribed in Section 1204 (a) at sections located as follows:

1. At the face of the column, pedestal or wall, for footings supporting a concrete column, pedestal or wall.

2. Halfway between the middle and the edge of the wall, for footings under masonry walls.

3. Halfway between the face of the column or pedestal and the edge of the metallic base, for footings under metallic bases.

(*c*) The width resisting compression at any section shall be assumed as the entire width of the top of the footing at the section under consideration.

(*d*) In one-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to the moment computed in the manner prescribed in Section 1204(a); and the reinforcement thus determined shall be distributed uniformly across the full width of the section.

(*e*) In two-way reinforced footings, the total tensile reinforcement at any section shall provide a moment of resistance at least equal to eighty-five per cent of the moment computed in the manner prescribed in Section 1204(a); and the total reinforcement thus determined shall be distributed across the corresponding resisting section in the manner

prescribed for square footings in Section 1204(f), and for rectangular footings in Sec. 1204(g).

(*f*) In two-way square footings, the reinforcement extending in each direction shall be distributed uniformly across the full width of the footing.

(*g*) In two-way rectangular footings, the reinforcement in the long direction shall be distributed uniformly across the full width of the footing. In the case of the reinforcement in the short direction, that portion determined by formula (30) shall be uniformly distributed across a band-width (B) centered with respect to the center line of the column or pedestal and having a width equal to the length of the short side of the footing. The remainder of the reinforcement shall be uniformly distributed in the outer portions of the footing.

$$\frac{\textit{Reinforcement in band-width} \quad (B)}{\textit{Total reinforcement in short direction}} = \frac{2}{(S + 1)} \qquad (30)$$

In formula (30), "S" is the ratio of the long side to the short side of the footing.

1205—Shear and bond

(*a*) The critical section for shear to be used as a measure of diagonal tension shall be assumed as a vertical section obtained by passing a series of vertical planes through the footing, each of which is parallel to a corresponding face of the column, pedestal, or wall and located a distance therefrom equal to the depth d for footings on soil, and one-half the depth d for footings on piles.

(*b*) Each face of the critical section as defined in Section 1205(a) shall be considered as resisting an external shear equal to the load on an area bounded by said face of the critical section for shear, two diagonal lines drawn from the column or pedestal corners and making 45° angles with the principal axes of the footing, and that portion of the corresponding edge or edges of the footing intercepted between the two diagonals.

(*c*) Critical sections for bond shall be assumed at the same planes as those prescribed for bending moment in Section 1204(b); also at all other vertical planes where changes of section or of reinforcement occur.

(*d*) Computations for shear to be used as a measure of bond shall be based on the same section and loading as prescribed for bending moment in Section 1204(a).

(*e*) The total tensile reinforcement at any section shall provide a bond resistance at least equal to the bond requirement as computed from the following percentages of the external shear at the section:

1. In one-way reinforced footings, 100 per cent.
2. In two-way reinforced footings, 85 per cent.

(*f*) In computing the external shear on any section through a footing supported on piles, the entire reaction from any pile whose center is located six inches or more outside the section shall be assumed as producing shear on the section; the reaction from any pile whose center is located six inches or more inside the section shall be assumed as producing no shear on the section. For intermediate positions of the pile center, the portion of the pile reaction to be assumed as producing shear on the section shall be based on straight-line interpolation between full value at six inches outside the section and zero value at six inches inside the section.

(*g*) For allowable shearing stresses, see Section 305 and 808.

(*h*) For allowable bond stresses, see Section 305 and 901 to 905.

1206—Transfer of stress at base of column

(*a*) The stress in the longitudinal reinforcement of a column or pedestal shall be transferred to its supporting pedestal or footing either by extending the longitudinal bars into the supporting member, or by dowels.

(*b*) In case the transfer of stress in the reinforcement is accomplished by extension of the longitudinal bars, they shall extend into the supporting member the distance required to transfer to the concrete, by allowable bond stress, their full working value.

(*c*) In cases where dowels are used, their total sectional area shall be not less than the sectional area of the longitudinal reinforcement in the member from which the stress is being transferred. In no case shall the number of dowels per member be less than four and the diameter of the dowels shall not exceed the diameter of the column bars by more than one-eighth inch.

(*d*) Dowels shall extend up into the column or pedestal a distance at least equal to that required for lap of longitudinal column bars (see Section 1103) and down into the supporting pedestal or footing the distance required to transfer to the concrete, by allowable bond stress, the full working value of the dowel.

(*e*) The compressive stress in the concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The unit compressive stress on the loaded area shall not exceed the bearing stress allowable for the quality of concrete in the supporting member as limited by the ratio of the loaded area to the supporting area.

(*f*) For allowable bearing stresses see Table 305(a), Section 305.

(*g*) In sloped or stepped footings, the supporting area for bearing may be taken as the top horizontal surface of the footing, or assumed as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the area actually loaded, and having side slopes of one vertical to two horizontal.

1207—Pedestals and footings (plain concrete)

(*a*) The allowable compressive unit stress on the gross area of a concentrically loaded pedestal shall not exceed $0.25f'_c$. Where this stress is exceeded, reinforcement shall be provided and the member designed as a reinforced concrete column.

(*b*) The depth and width of a pedestal or footing of plain concrete shall be such that the tension in the concrete shall not exceed $.03f'_c$, and the average shearing stress shall not exceed $.02f'_c$ taken on sections as prescribed in Section 1204 and 1205 for reinforced concrete footings.

1208—Footings supporting round columns

(*a*) In computing the stresses in footings which support a round or octagonal concrete column or pedestal, the "face" of the column or pedestal shall be taken as the side of a square having an area equal to the area enclosed within the perimeter of the column or pedestal.

1209—Minimum edge-thickness

(*a*) In reinforced concrete footings, the thickness above the reinforcement at the edge shall be not less than six in. for footings on soil, nor less than twelve in. for footings on piles.

(*b*) In plain concrete footings, the thickness at the edge shall be not less than eight in. for footings on soil, nor less than fourteen in. above the tops of the piles for footings on piles.

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Studies of the Physical Properties of Hardened Portland Cement Paste*

By T. C. POWERS†
Member American Concrete Institute
and T. L. BROWNYARD‡

Part 3. Theoretical Interpretation of Adsorption Data§

CONTENTS

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Part 1. "A Review of Methods That Have Been Used for Studying the Physical Properties of Hardened Portland Cement Paste". ACI Journal, October, 1946.
Part 2. "Studies of Water Fixation"—Appendix to Part 2. ACI Journal, November, 1946.
Part 3. "Theoretical Interpretation of Adsorption Data."
Part 4. "The Thermodynamics of Adsorption"—Appendix to Parts 3 and 4.
Part 5. "Studies of the Hardened Paste by Means of Specific-Volume Measurements."
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Part 7. "Permeability and Absorptivity."
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†Manager of Basic Research, Portland Cement Assn. Research Laboratory, Chicago 10, Ill.
‡Navy Dept., Washington, D. C., formerly Research Chemist, Portland Cement Assn. Research Laboratory, Chicago 10, Ill.

§The characteristics of the cements mentioned in this section may be found in the Appendix to Part 2.

The adsorption isotherms obtained from hardened cement paste are identical in several respects with those obtained from other materials that are very different in chemical and physical properties. For example when glass spheres[1]* or oxide-coated cathodes of radio tubes[2] are exposed to nitrogen vapor (at the temperature of liquid air), or when a plane mercury surface is exposed to CCl_4 vapor (11 C),[3] the adsorption curves obtained are of the same type as those for water vapor on cement paste. Also, the same type of curve is obtained when crystalline solids such as titanic oxide, stannic oxide, zinc oxide or pulverized quartz are exposed to water vapor at room temperature[4]. Moreover, curves of the same type may be obtained with different vapors on the same solid[5].

The similarities just mentioned exist not only among materials that are not porous, that is, materials on which adsorption is confined to the visible surfaces, but also among many porous solids having negligible superficial surface areas. With suitable vapors the following materials, some porous, some not, all give the same type of isotherm: building stone[6]; cotton[7, 8]; asbestos fibre[9]; wood[10]; wood pulp[11]; carbon black[12]; titania gel, ferric oxide gel, rice grains[13]; cellophane[14]; bone-char[15]; cellulose[16]; silica gel[17]; proteins[18]; soils[19]; wool[20].

It appears therefore that the curves found for portland cement paste are not characteristic of the particular substances composing the paste but represent some factor common to many dissimilar substances. We will see in what follows that this common factor is probably nothing other than a solid surface that has an attraction for the adsorbed substance.

B.E.T. THEORY

Various theories have been advanced to explain the taking up of gases and vapors by solid materials[5]. Among the most recent, and at present the most useful, is the theory of Brunauer, Emmett and Teller,[21, 22] known as the multimolecular-adsorption theory, or the B.E.T. theory for brevity. It is beyond the scope of this paper to discuss the B.E.T. theory in full; reference should be made to Brunauer's book[5] or to the original papers for an adequate treatment. However, it is necessary to review here the main features of the theory.

The theory rests on a concept advanced earlier by Langmuir. The taking up of a gas by a solid is considered to be the result of a physical attraction between the molecules of the gas and the surface molecules of the solid.† This field of force is believed to arise from three different

*See references end of Part 3.
†Chemical reaction with the solid is not excluded by the theory but it will not be considered here.

causes, or to be made up of three different forces that are known collectively as *van der Waal's forces.* Hence, the process under discussion is called *van der Waal's adsorption.* These forces are of a lesser order of intensity than those involved in most chemical reactions, but they may be effective over greater distances.

A solid surface exposed to a continuous bombardment of gas molecules catches and holds some of the gas molecules, at least momentarily. Moreover, when the gas is also a vapor such as water, the molecules caught on the surface are in a condensed state and may be considered as a separate phase. Hence, when they are adsorbed, the molecules must give up their latent heat of vaporization. Besides this, adsorption usually is accompanied by a further evolution of heat which may be said to represent the energy of interaction between the solid and the condensed substance; this is called the *net heat of adsorption.*

Some of the adsorbed molecules acquire enough kinetic energy to escape from the force field of the solid surface. The over-all result is a continuous interchange between the surface region and the interior of the vapor phase, but the average molecular concentration at the solid surface remains higher than that of the interior of the vapor phase by virtue of the surface attraction.

The derivation of the mathematical statement of the theory starts with the assumption that the rate of condensation is directly proportional to the frequency of impact between the solid and the vapor molecules, which frequency is proportional to the vapor pressure when temperature is constant. The rate of evaporation is expressed as a function of the amount of energy a condensed molecule must acquire to escape from a particular situation in the condensed phase. In the derivation of the mathematical expressions that have found use, only two situations of a molecule in the condensed phase are recognized: (1) a molecule may be condensed on bare surface; (2) a molecule may be condensed on a layer of previously condensed molecules. It is assumed that a molecule in the second situation can escape when it acquires energy exactly equal to its heat of vaporization; a molecule in the first situation must acquire a different (usually greater) amount of energy to escape. In other words, the net heat of adsorption is assumed to be zero for all molecules not in the first layer.*

The theory requires that at any given vapor pressure the amount adsorbed is directly proportional to the surface area of the solid.

A logical extension of the assumptions made in the derivation is that at the saturation pressure there is no limit to the number of layers that might be condensed on an open surface.

*Brunauer, Emmett, and Teller have published an equation representing the assumption that the net heat of adsorption from the second layer also differs from zero. This equation includes also the assumption that the packing of molecules is different in the first from that in the succeeding layers. This equation does not appear to have found use (See Ref. 21, p. 313).

The most widely used mathematical statement of the theory is the following expression:

$$\frac{w}{V_m} = \frac{C(p/p_s)}{(1 - p/p_s)\ (1 - p/p_s + C(p/p_s)\ )} \quad \text{(A)}$$

in which

w = quantity of vapor adsorbed at vapor pressure p

V_m = quantity of adsorbate required for a complete condensed layer on the solid, the layer being 1 molecule deep

p = existing vapor pressure

p_s = pressure of saturated vapor

C is a constant related to the heat of adsorption as follows:

$$C = k\,e^{\frac{Q_1 - Q_L}{RT}}$$

where

k is a constant assumed to be 1.0 in computations.

Q_L = normal heat of condensation of the vapor per mole of vapor

Q_1 = total heat of adsorption per mole of vapor

$(Q_1 - Q_L)$ = net heat of adsorption per mole of vapor

R = the gas constant

T = absolute temperature

e = base of natural logarithms

Owing to the assumptions made in the derivation, eq. (A) would be expected to hold only for adsorption on exterior surfaces. It could hardly be expected to hold for surfaces on the interior of a porous solid where unlimited adsorption would obviously be impossible. Brunauer, Emmett, and Teller recognized this and introduced another constant n which was intended to be the maximum number of layers adsorbed. The resulting four-constant equation does not fit any known data from a porous adsorbent over the whole pressure range. However, Pickett[23] discovered a way to improve the derivation of the B.E.T. four-constant equation and supplied a better expression. This equation has the same four constants as the original. It fits many adsorption curves over about 90 percent of pressure range. In some cases it conforms to the extremes, but in the middle range the theoretical curve is above the experimental. There are also many curves that this equation does not fit in the high-pressure range. One reason for this will be explained in connection with the theory of capillary condensation, discussed below.

In general, the four-constant equations did not prove to be very useful in this study; hence, they are not given here. However, the three-constant eq. (A) can be used to represent the low-pressure part of the curve precisely. Of the three constants, p_s is the same for all curves and C is the same for most of them. Consequently, in most cases adsorption characteristics can be represented by V_m alone.

THE THEORY OF CAPILLARY CONDENSATION

Condensation of vapor in a porous cement paste seems to be most adequately explained by a combination of a theory based on the energy available at the solid surface, such as the B.E.T. theory discussed above, and a theory based on energy available at the surface of a liquid, the capillary-condensation theory.

The capillary-condensation theory rests on the fact that the surface of a liquid is the seat of available energy. The molecules at the surface of a liquid not being completely surrounded by other molecules of like kind are under an inwardly directed intermolecular force. When a given body of water is changed in shape so as to increase its surface area, work must be done against the forces tending to draw the molecules out of the surface. Consequently, when left to itself, a small body of liquid tends to become spherical, since that is the form giving a minimum of surface.

This phenomenon has an effect on the vapor pressure of the liquid. How this comes about can be seen by considering the behavior of water in a small glass cylinder, as shown in Fig. 3-1. The solid curve at the top

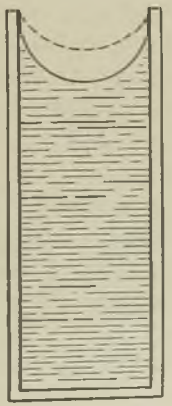


Fig. 3-1

represents the meniscus of the water surface. Owing to the surface tension, which strives to straighten the meniscus, that is, to reduce its curvature and thus reduce the surface area, the water in the vessel is under tension. Consequently, the vapor pressure of the water in the tube will be less than normal for the existing temperature. The greater the curvature of the meniscus, the greater the tension in the water and hence the lower the vapor pressure. The relationship between surface-curvature and vapor pressure was worked out by Lord Kelvin. It may be written

$$\ln p/p_s = -\frac{2\sigma M}{d_l R T r} \qquad (1)$$

where

p_s = vapor pressure over plane surface at temperature T, the saturated vapor pressure
p = the existing vapor pressure over a concave surface
σ = surface tension of liquid
d_f = density of liquid
M = molecular weight of liquid
R = the gas constant
T = absolute temperature
r = radius of curvature of the circular meniscus

Under the conditions pictured in Fig. 3-1 the greatest curvature that the liquid can have is limited by the radius of the container. When the liquid surface has this curvature, the liquid can be in equilibrium with only one vapor pressure, p, corresponding to the radius of curvature r, as given in eq. (1). If the pressure of the vapor is kept below p, all the liquid in the vessel will evaporate. If the pressure is maintained above p, some vapor will condense, increasing the amount of water in the vessel. But under the conditions pictured, condensation would lessen the curvature of the liquid as indicated by the dotted line and there would be a corresponding rise in the equilibrium vapor pressure. If the vapor pressure is kept equal to the saturation pressure, that is, the maximum possible over a plane surface at the existing temperature, condensation will proceed until the tube fills and the surface curvature disappears ($r = \infty$).

The minimum relative vapor pressure at which the liquid in the vessel can retain its meniscus depends on the maximum possible curvature of the meniscus. This in turn depends on the bore of the cylinder. Table 3-1 gives an idea of the curvature required to produce a given effect on vapor pressure.

The main point to note in Table 3-1 is that the surface must have a high degree of curvature to produce an appreciable effect on the vapor pressure of the liquid. For example, if the radius of curvature is 0.1 micron, the vapor pressure is 99 percent of the normal value, p_s. A radius of curvature of 0.0015 micron would reduce the pressure to 50 percent of p_s. The figures given should not be taken too literally; they are undoubtedly in error through the low-pressure range, for the equation takes no account of the fact that the physical properties of the liquid are affected by capillary and adsorption forces. The table serves only to indicate what the order of magnitude of the curvatures must be when capillary condensation occurs.

An "adsorption curve" of almost any shape could easily be accounted for by the capillary condensation theory. The simplest approach is to imagine first that the voids in a porous solid are cylindrical pores of

TABLE 3-1—RELATIONSHIP BETWEEN AQUEOUS RELATIVE VAPOR PRESSURE AND RADIUS OF CURVATURE

Calculated from Kelvin's Equation—Temperature = 25 C

Relative vapor pressure, p/p_s	Radius of curvature of meniscus	
	cm	microns
0.10	4.6×10^{-8}	.0005
0.20	6.6×10^{-8}	.0007
0.30	8.8×10^{-8}	.0009
0.40	11.5×10^{-8}	.0012
0.50	15.2×10^{-8}	.0015
0.60	20.8×10^{-8}	.0021
0.70	29.5×10^{-8}	.0030
0.80	47.2×10^{-8}	.0047
0.90	100.0×10^{-8}	.0100
0.95	204.0×10^{-8}	.0204
0.98	532.0×10^{-8}	.0532
0.99	1052.0×10^{-8}	.1052
1.00	∞	∞

various sizes and that when a cylinder is partly filled, the radius of curvature of the liquid meniscus equals the radius of the cylinder. At a given pressure p, all cylinders smaller than the corresponding r (see eq. 1) would become or remain full and all larger than r would become or remain empty. The shape of the curve would therefore depend on the range of sizes present and the total capacity of each size.

This simple analogy led Freyssinet [24] and others before him to consider an adsorption curve to be a means of ascertaining pore-size distribution in systems of submicroscopic pores. However, a naturally formed porous solid so constituted is hardly conceivable. It is much more likely that the pore spaces resemble those in an aggregation of particles. The shapes of the particles may be spherical, fibrous, or between these extremes. Capillary condensation could occur in the interstices in the manner illustrated in Fig. 3-2. At the point of contact between two spheres there is a space of wedge-shaped cross section. Condensation in this space would form a circular lens around the point of contact and the liquid would present a curved surface. Similarly, where two prismatic bodies make contact, water condensed in the region of contact would present a curved surface, somewhat as shown.

In the circumstances pictured, the water surface would have two curvatures. Hence, in this case Kelvin's equation would take the general form

$$ln\ p/p_s = -\frac{\sigma M}{d_l RT}\left(\frac{1}{r_1} + \frac{1}{r_2}\right) \qquad (2)$$

where

r_1 and r_2 are the principal radii of curvature.

For the conditions shown in Fig. 3-2 one of the r's would be negative.

Since the structure of paste is probably granular, fibrous, or perhaps plate-like, it is evident that Kelvin's equation provides no simple way of computing the size of the pores; it only gives the *effective* curvature of the water surface. It can, however, explain the condensation of vapor in a porous solid, and we will see evidence in the data indicating that a part of what is here called adsorbed water in cement pastes is taken up by capillary condensation.

COMBINING THE B.E.T. AND CAPILLARY CONDENSATION THEORIES

When adsorption of a vapor occurs in a porous solid of granular or fibrous structure, the liquid surfaces are certain not to be plane. They will be concave, at least in some regions. Therefore, the free surface energy of the solid and the free surface energy of the condensed liquid must both be causes of condensation.*

Brunauer, Emmett, and Teller tried to take this factor into account by adding a fifth constant representing the release of the surface energy of the liquid when two adsorbed layers merge[26]. This five-constant equation is very unwieldy and it embodies the assumption questioned by Pickett[23]. Also, it rests on the oversimplifying assumption that adsorption is occurring only between plane parallel surfaces.

By the B.E.T. theory, vapor would be expected to condense uniformly over all the available surface. But in a porous body, the surface tension

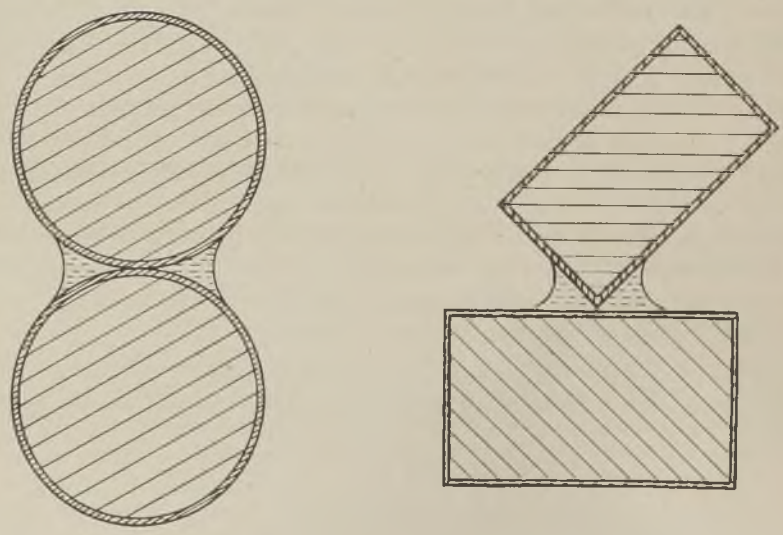


Fig. 3-2

*See Part 4.

of the liquid must influence the distribution of the condensed water, whether it is primarily responsible for the condensation or not. Thus, in Fig. 3-2, B.E.T. adsorption could lead only to a uniform layer of condensate around each sphere, but surface tension would require the liquid to collect as shown, for the condition pictured is the most stable one possible under the circumstances. This follows at once from the fact that if the liquid shown in the lens were spread evenly over the two spheres, the total surface area of the liquid would be increased and the work done would be equal to the product of the increase in area and the surface tension of the liquid.

If, as assumed in the B.E.T. equation, only the molecules in the first layer lose more than their heat of condensation, then all but the first layer should tend to collect in the lens. However, Harkins and Jura[27] have shown that the adsorption forces probably affect more than the first layer. Therefore, the net heat of adsorption probably represents the heat from several layers. This being true we can expect the adsorbed layer to vary in average thickness with the curvature of the liquid meniscus, the greater the curvature the greater the tendency for adsorbed molecules to collect in the lens.

On account of the effect of liquid surface tension the relationship between water content and vapor pressure depends on the characteristics of the pore system. Since the characteristics of the pore system are not predictable from any adsorption theory, it follows that no general equation can be expected to apply to all porous bodies over the whole pressure range. Since, however, no appreciable capillary condensation is to be expected below about $0.4p_s$, general theories may be expected to apply in this low range of vapor pressures.

As will be seen, little quantitative use is made of these theories, except the B. E. T. eq. (A) applied to the range $p = 0.05\ p_s$ to $0.40\ p_s$. However, many features of the interpretations given rest on the theories described above. That is, it was possible to use the theories qualitatively even where the mathematical expressions intended as expressions of the theoretical concepts were found to be inadequate.

Effect of soluble salts on adsorption curves

The foregoing discussion is based on the assumption that the adsorbed liquid is pure water. However, in hardened portland cement paste the water will contain dissolved salts, principally $Ca(OH)_2$, $NaOH$ and KOH.* At any given water content the observed reduction in vapor pressure is due not only to the capillary and surface effects described above, but also to the dissolved salts. The magnitude of the effect of the dissolved salts can be estimated from the vapor-pressure isotherms of the salt

*Because of its low solubility, $Ca(OH)_2$ can be ignored.

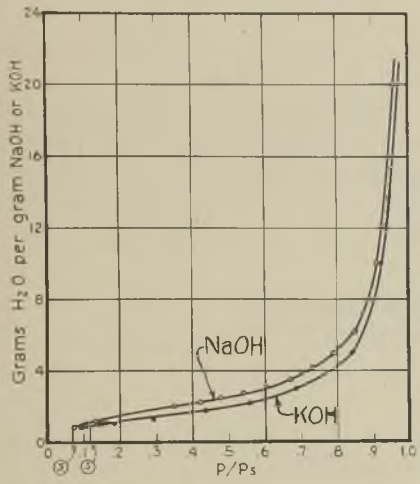
solutions and the amount of dissolved alkali in the sample of hardened paste.

Vapor-pressure data [28] for aqueous solutions of $NaOH$ and KOH are given in Fig. 3-3. The lowering of vapor pressure due to the alkalies can be determined from these curves, for any given concentration of the alkalies.

The amount of dissolved alkali in hardened paste probably varies considerably among the different samples. Sample No. 254-11-2 is considered to be representative of the average. The original mortar was prepared from cement 15758 (w_o/c = 0.46) and was cured 28 days in water. The cement originally contained 0.3 percent Na_2O and 0.4 percent K_2O which, in g/g of cement, corresponds to 0.0019 and 0.0024 of $NaOH$ and KOH, respectively. An unknown portion of these alkalies undoubtedly remained in the unhydrated cement and can be considered insoluble for the present purpose. Some of the soluble alkali was leached from the sample during the 28-day period of water curing. There is also reason to believe that some of it was adsorbed by the solid phase and thus effectively kept from the solution. Hence, the sample as tested must have contained considerably less than 0.004 g of soluble alkali (total of $NaOH$ and KOH) per g of cement. The adsorption isotherm from this sample is given in Fig. 3-4, upper curve. This curve presumably represents the combined effect of surface adsorption, capillary condensation, and dissolved alkali. The lower curve in Fig. 3-4 shows the amount of water that could have been held by 0.004 g of sodium hydroxide at any given relative vapor pressure above about 0.07 p_s, the vapor pressure of a saturated solution of sodium hydroxide at 25 C. (See Fig. 3-3.) Since the actual amount of alkali must have been less than 0.004 g, the amount of water that could have been held by the alkali alone would be represented somewhere below the curve as drawn.

It thus becomes apparent that at pressures below about 0.8 p_s only a very small part of the total water taken up by this cement paste could be accounted for by the dissolved alkalies. In the range of higher pressures, the possible effect of the alkali is greater. As p/p_s approaches 1.0, the amount of water that could be held by the alkali approaches infinity. Since, however, the topmost point of the curve is established by saturating the granular sample with liquid water, its position is determined by the capacity of the sample and the assumption that, at saturation, p/p_s = 1.0. In this case the capacity of the sample was 0.33 g of water per gram of original cement. With 0.004 g of alkali in 0.33 g of evaporable water the relative vapor pressure would be slightly below 1.0.

Since the effect of the alkali is small, no correction for its effect has been attempted; the topmost point is always plotted at p/p_s = 1.0, and



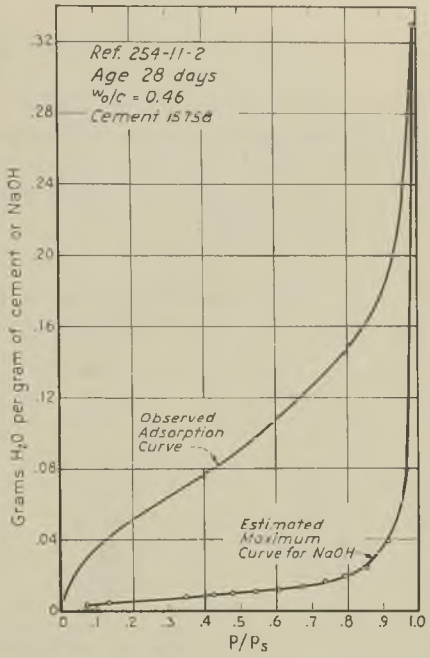


Fig. 3-3 (above)—Vapor pressure isotherms for aqueous solutions of $NaOH$ and KOH

S = Saturated solution

Fig. 3-4 (right)—Estimation of effect of dissolved alkali on adsorption isotherm

all other points are treated as if their positions were determined by surface adsorption and capillary condensation only.

Differences in the amount of soluble alkali among various samples, within the range that might reasonably be expected, probably have little effect on the lower part of the curve, the part that is used in the analyses presented below. However, such differences probably produce considerable effects on the shapes of the upper parts of the curves and hence contribute to the difficulty of interpreting the upper parts.

To minimize the effect of dissolved alkali, most of the specimens were stored in water for the first 28 days of their curing period to leach some of the alkali from the specimens. Longer periods of water storage were avoided because of undesirable leaching of $Ca(OH)_2$.

APPLICATION OF THE B.E.T. EQUATION (A)

Method of evaluating C and V_m

The constants C and V_m were evaluated in the manner recommended by Brunauer, Emmett, and Teller. Eq. (A), written in the form

$$\frac{1}{w}\frac{x}{1-x} = \frac{1}{V_m C} + \frac{C-1}{V_m C}x, \quad (x = p/p_s)$$

shows that a plot of experimental values for $\frac{1}{w}\frac{x}{1-x}$ against x should

give a straight line. The slope of the line will be $\frac{C - 1}{V_m C}$ and the intercept on the y-axis will be $1/V_m C$. The application of this method to data from cement pastes is illustrated in Fig. 3-5 and in Tables 3-2 and 3-3. Two of the four diagrams represent data obtained by the air-stream method, and the others represent data obtained by the high-vacuum method.

TABLE 3-2—TYPICAL DATA FOR COMPUTING V_m AND C

Data obtained by air-stream method

p/p_s (= x)	w g/g of dry paste	$\frac{x}{1 - x}$	$\frac{1}{w} \frac{x}{1 - x}$
Ref. 254-11-11—Cement 16213			
0.081	0.0189	0.088	4.65
0.161	0.0253	0.192	7.60
0.238	0.0298	0.312	10.45
0.322	0.0362	0.475	13.1
0.360	0.0395	0.562	14.2
0.530	0.0503	1.128	22.4
Ref. 254-9-15A-180—Cement 15365			
0.09	0.0267	0.099	3.79
0.20	0.0348	0.250	7.18
0.36	0.0464	0.562	12.1
0.47	0.0539	0.887	16.5
Ref. 254-9-15A-270A—Cement 15365			
0.081	0.0187	0.0873	4.68
0.161	0.0264	0.192	7.27
0.238	0.0310	0.312	10.0
0.322	0.0359	0.475	13.2
0.360	0.0400	0.563	14.1
0.530	0.0500	1.127	22.6

C and V_m can be computed conveniently from the intercept on the x-axis and from the ordinate at $x = 0.5$. Thus, when $\frac{1}{w} \frac{x}{1 - x} = 0$, $C = 1 - \frac{1}{x}$; and when $x = 0.5$, $V_m = \frac{1 + C}{2C} w$. In Fig. 3-5A we see that when $\frac{1}{w} \frac{x}{1 - x} = 0$, $x = -0.053$ and when $x = 0.5$, $1/w = 19.3$. Therefore,

$$C = 1 + \frac{1}{0.053} = 19.8, \text{ and } V_m = \frac{1 + 19.8}{2 \times 19.8} \times \frac{1}{19.3} = 0.027$$

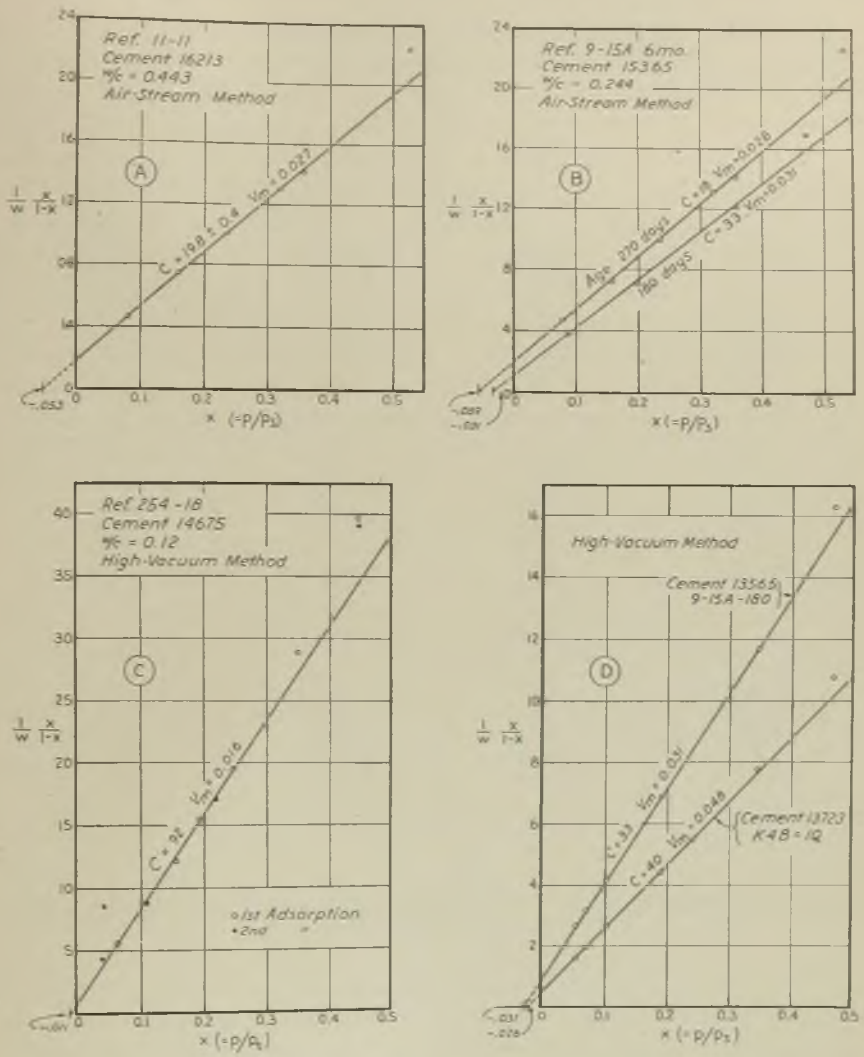


Fig. 3-5—Typical plots of the kind used for evaluating C and V_m
Data from Tables 3-2 & 3-3.

The nomenclature used here is the same as that used by Brunauer et al. except that they used V for the *volume* of gas adsorbed, whereas here w is used for *weight* adsorbed. w is expressed either in grams per gram of dry hardened paste (evaporable water removed) or as grams per gram of original cement. Having become accustomed to thinking of V_m as a factor proportional to surface area, we have somewhat illogically retained this symbol, but express it in grams instead of cc.

TABLE 3-3—TYPICAL DATA FOR COMPUTING V_m AND C

Data obtained by high-vacuum method

p/p_s ($= x$)	w g/g of dry paste	$\frac{x}{1-x}$	$\frac{1}{w}\frac{x}{1-x}$
Ref. 254-18-9-15A-180—Cement 15365			
0.055	0.0227	0.0598	2.63
0.073	0.0249	0.0790	3.17
0.104	0.0284	0.117	4.13
0.193	0.0347	0.240	6.91
0.244	0.0400	0.322	8.06
0.349	0.0455	0.534	11.7
0.472	0.0542	0.885	16.3
Ref. 254-K4B-1Q—Cement 13723			
0.055	0.0369	0.0598	1.62
0.073	0.0397	0.0790	1.99
0.104	0.0435	0.117	2.69
0.193	0.0540	0.240	4.40
0.244	0.0587	0.322	5.49
0.349	0.0686	0.534	7.78
0.472	0.0819	0.885	10.8
Ref. 254-18—Cement 14675			
0.064	0.0552	0.0684	1.24
0.155	0.0646	0.1835	2.84
0.193	0.0675	0.2395	3.55
0.249	0.0720	0.3310	4.60
0.350	0.0817	0.5380	6.58
0.446	0.0910	0.8040	8.84

THE RELATIONSHIP BETWEEN V_m AND NON-EVAPORABLE WATER (w_n) FOR SAMPLES CURED AT 70-75 F

The quantity V_m is considered to be proportional to the internal surface area of the sample. Since the specific surface of microcrystalline material is negligible compared with that of colloidal material, V_m is also considered to be proportional to the amount of colloidal material in the sample. The quantity w_n represents the amount of non-evaporable water in both colloidal and non-colloidal material. Therefore, if a given cement produces the same kind of hydration products at all stages of its hydration, the ratio V_m/w_n should be constant for any given cement under fixed curing conditions. However, the ratio of colloidal to non-colloidal hydration products should be expected to differ among cements of different chemical composition. Hence, the ratio V_m/w_n should be different for different cements.

V_m and w_n are plotted for several different cements and different mixes and ages, in Fig. 3-6 and 3-7. The points in most of the diagrams show a considerable amount of scatter. Some of this is due to random experimental vagaries and some of it is apparently due to variations in drying conditions discussed earlier.

In view of the fact that the variations show no consistent influence of the original water-cement ratio or of the age of the sample, the relationships indicate that for any given cement the ratio V_m/w_n is independent of age or water-cement ratio.

If the points fell on a straight line through the origin, that would indicate that the ratio of colloidal to non-colloidal products is precisely the same at all stages of hydration. The data are not sufficiently concordant to indicate definitely whether this is so or not. There is some reason to believe that the reactions during the first few hours cannot produce exactly the same products as those that occur later. Particularly, the gypsum is usually depleted within the first 24 hours and thus the reactions involving gypsum cannot occur at later periods.

If calcium sulfoaluminate as produced in paste is non-colloidal, the ratio of colloidal to non-colloidal material should be lower during the first 24 hours than it is at any later time. The effect on the graph would be that of causing the proportionality line that holds for the later ages to cut the w_n-axis to the right of the origin. As said before, the plotted data do not indicate definitely where the intercept should be. They do indicate, however, that if the intercept is to the right of the origin, it is nevertheless near the origin, as indeed it should be in view of the relatively small amount of calcium sulfoaluminate that is formed. To simplify the handling of the data, we have assumed that the line passes through the origin and thus have considered the ratio of colloidal to non-colloidal material to be the same at all ages for a given cement.

Fig. 3-6 represents the data from Series 254-8 and 254-9. It represents 5 different commercial cements that had been used in experimental highways. The data on this series are the least concordant of all obtained in the investigation.

Fig. 3-7 represents cements prepared from 5 other commercial clinkers. Different plant grinds of each clinker were blended to give each cement a specific surface of 1800 sq. cm. per g (Wagner).

The ratios determined for the cements in each of these two groups were averaged. The results are shown in Table 3-4.

The figures in the third column give the mean values and those in the fourth give the probable error of the mean. Those in the last column give the probable error of a single test value and thus reflect the degree of scatter of the data.

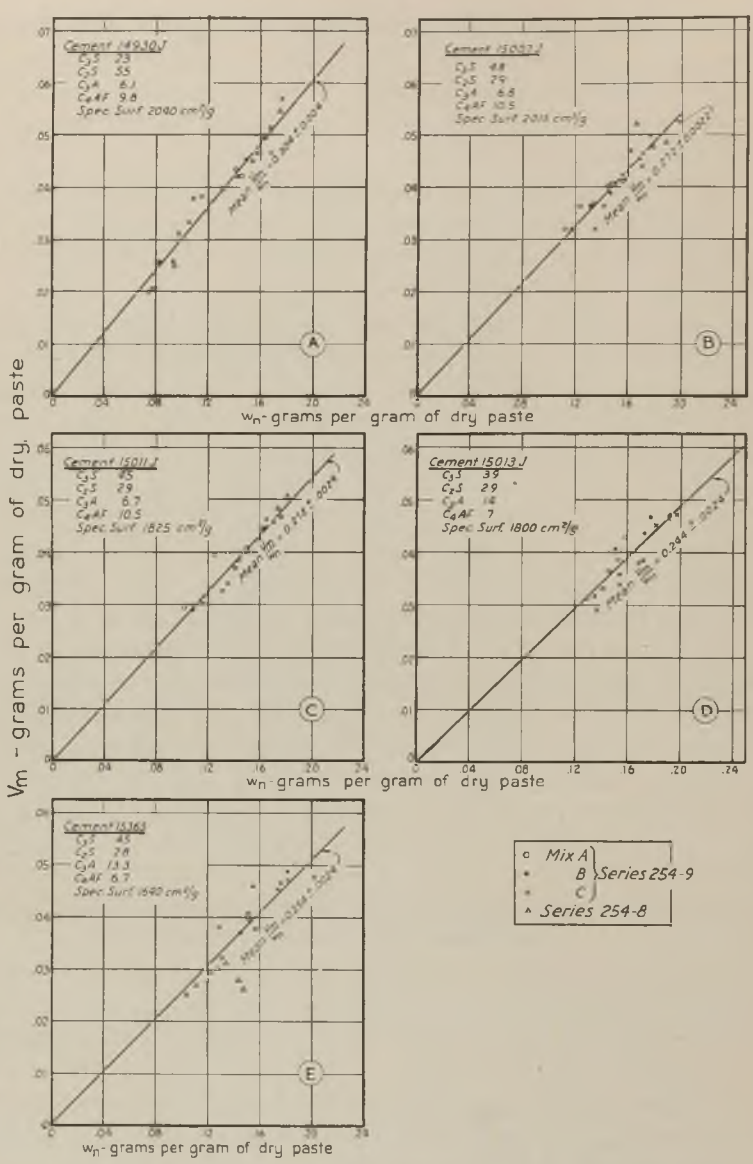


Fig. 3-6—Relationship between V_m and w_n for pastes of Series 254-9

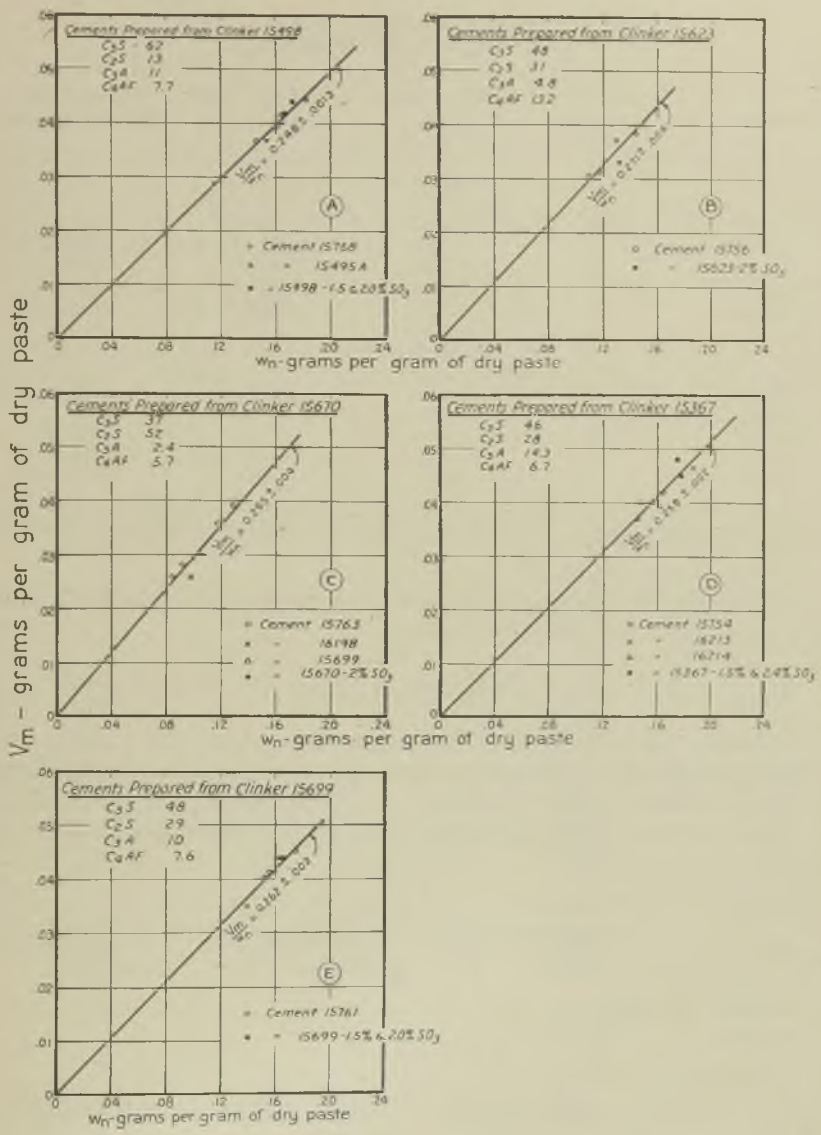


Fig. 3-7—Relationship between V_m and w_n for pastes of Series 254-11 and 254-13

TABLE 3-4—MEAN VALUES OF V_m/w_n FOR TWO GROUPS OF CEMENTS

Cement No.	ASTM classification*	Mean value of V_m/w_n	Probable error of mean	Probable error of single value
		Series 254-9		
14930J	IV	.304	.00392	.0176
15007J	II	.272	.00216	.0089
15011J	II	.272	.00242	.0102
15013J	I	.244	.00259	.0107
15365	I	.254	.00239	.0115
		Series 254-11		
15498	III	.248	.00131	.003
15623	II	.271	.00405	.008
15670	IV	.295	.00411	.010
15367	I	.258	.00244	.006
15699	I	.262	.00203	.004

*ASTM designation C-150-40T. The classification here is based only on the computed quantities of the four major compounds. The surface area requirements are not met in all cases.

To evaluate the effect of composition, the assumption was made that the form of the relationship would be

$$\frac{V_m}{w_n} = A(\%\ C_3S) + B(\%\ C_2S) + C(\%\ C_3A) + D(\%\ C_4AF)$$

The values of the coefficients were determined first from 200 items of data representing about 50 different cements. Then, owing to the fact that 100 of these items represented only 5 cements (Series 254-9), a second analysis was made excluding the data from Series 254-9. The results of the two analyses were as follows:*

For 200 items of data:

$$\text{Eq. (3): } \frac{V_m}{w_n} = \left[\underset{\pm .00006}{.00208}(\%C_3S) + \underset{\pm .00004}{.00326}(\%C_2S) + \underset{\pm .00016}{.00251}(\%C_3A) + \underset{\pm .00030}{.00549}(\%C_4AF) \right]$$

For 100 items of data:

$$\text{Eq. (4): } \frac{V_m}{w_n} = \left[\underset{\pm .00012}{.00230}(\%C_3S) + \underset{\pm .00006}{.00320}(\%C_2S) + \underset{\pm .00016}{.00317}(\%C_3A) + \underset{\pm .00046}{.00368}(\%C_4AF) \right]$$

The two equations give similar coefficients for C_2S and C_3S but considerably different coefficients for C_3A and C_4AF. However, applied to the same cements, the computed results are not far different, except for

*See footnote, Part 1, page 121.

certain compositions. This is shown by the data given in Table 3-5. Here the observed mean values of V_m/w_n for the cements appearing in Fig. 3-6 and 3-7 are compared with values computed from composition. It appears that the value of V_m/w_n can be computed from composition with fairly satisfactory accuracy by means of either equation. However, eq. (4) probably gives the more correct evaluation of the effect of variations in composition, since individual cements appear in the analysis with approximately equal weights.

The influence of composition as given by eq. (4) is illustrated in Table 3-6.

The results given in the last column of the table show that the influence of composition on this ratio is not very large. The variation occurs mainly among those cements whose compositions differ from the average chiefly in C_3S, C_4AF or both.

TABLE 3-5—COMPARISON OF OBSERVED AND COMPUTED VALUES OF V_m/w_n

Cement No.	ASTM type	Mean V_m/w_n observed	V_m/w_n computed from compound composition	
			Eq. (3)	Eq. (4)
15013J	I	0.244±0.003	0.253	0.256
15365	I	0.254±0.002	0.255	0.260
15367*	I	0.258±0.002	0.253	0.258
15699	I	0.262±0.002	0.254	0.255
Average		0.254	0.254	0.256
15007J	II	0.272±0.002	0.269	0.264
15011J	II	0.272±0.002	0.263	0.257
15623*	II	0.271±0.004	0.284	0.272
Average		0.272	0.272	0.264
14930J	IV	0.304±0.004	0.298	0.286
15670*	IV	0.295±0.004	0.284	0.281
Average		0.300	0.291	0.284
15498*	III	0.248±0.001	0.243	0.249

*This item represents all cements made from this clinker.

The theoretical significance of the coefficients of eq. (3) or (4) found by the method described is not certain, mainly because of the lack of definite knowledge concerning the chemical composition of the hydration products. The data support arguments given elsewhere to the effect that the hydration products of the alumina-bearing compounds are not microcrystalline as they are when these compounds are hydrated alone in an abundance of water; instead, the coefficients for C_3A and C_4AF,

TABLE 3-6—INFLUENCE OF CHEMICAL COMPOSITION ON V_m/w_n

Type of composition		Computed compound composition percent by wt.				Computed $\frac{V_m}{w_n}$ Eq. (4)
		C_3S	C_2S	C_3A	C_4AF	
Type I	Normal C_3A	45	29	9.7	7.5	0.255
	High C_3A	45	28	14.0	5.0	0.256
Type II	High iron	41	29	5.4	14.8	0.259
	High silica	40	41	6.4	9.7	0.279
Type III	Normal C_3A	59	13	10.4	7.6	0.238
	High C_3A	59	13	14.0	5.0	0.240
Type IV	High iron	25	48	6.2	13.8	0.282
	High silica	33	54	2.3	5.8	0.277

taken literally, indicate that, per gram of compound, these compounds contribute to the total surface area of the hydration products, that is, to V_m, as much as or more than do the two silicates, which are known to produce colloidal hydrates. In any event, the analysis shows that however the Al_2O_3 and Fe_2O_3 enter into combination in the hydration products, they must appear in solids having a high specific surface.

The coefficient of C_3S is less than that of C_2S. This result is compatible with the data of Bogue and Lerch[29] showing that both compounds produce a colloidal hydrous silicate, but only C_3S produces microcrystalline $Ca(OH)_2$. The indications are that $Ca(OH)_2$ does not decompose under the drying conditions of these experiments. Hence, the occurrence of $Ca(OH)_2$ contributes to w_n but contributes very little to V_m.

THE SPECIFIC SURFACE OF HARDENED PASTE

Computation of surface area

From the derivation of the B.E.T. equation it follows that the surface area of the adsorbent should be equal to the product of the number of adsorbed molecules in the first layer and the area covered by a single molecule. Hence, when V_m is expressed in grams per gram of adsorbent (dry paste),

$$S = a_1 \frac{V_m N}{M} \qquad (5)$$

where

S = the surface area of the adsorbent, sq. cm. per g

a_1 = the surface area covered by a single adsorbed molecule

N = 6.06 x 10^{23}, the number of molecules in a gram-molecular wt. (Avagadro's number)*

M = molecular weight of the adsorbed gas.

The value of a_1 has been estimated in several ways. Livingston[30] found a value for water of 10.6 x 10^{-16} sq. cm. per molecule. Gans, Brooks, and Boyd[4] used the same figure. Emmett[31] gave a formula for computing a_1 from the molecular weight and density of the condensed vapor which gives nearly the same result if the normal density of water is used. As will be shown, values for molecular area obtained in this way when introduced into eq. (5) actually give surface areas S that are in close agreement with the results obtained by other procedures. Hence, for water we may write,

$$S = \frac{10.6 \times 10^{-16} \times 6.06 \times 10^{23}\, V_m}{18}$$

$$= (35.7 \times 10^6) V_m \text{ sq. cm. per g} \qquad (6)$$

Verification of surface areas as computed from V_m

Gaudin and Bowdish[1] used pyrex-glass spheres calibrated by microscope measurements. Low temperature adsorption of nitrogen gave almost exactly the same specific surface as that computed from the mean size of the spheres. Harkins and Jura[32] developed a method of measuring surface area by first covering particles with a complete film of adsorbed water and then measuring the heat evolved when the particles were immersed in water. The results obtained by this method were compared with those from the B.E.T. method for 60 different solids. For 58 of the 60 solids the areas by the Harkins and Jura method differed from those obtained by the B.E.T. method by no more than 9 percent. Emmett[25, 31, 33] presented an extensive array of data showing that wherever particle size can be checked directly, as with the electron microscope, the results of the B.E.T. method look reasonable, to say the least. Because of such evidence as this, the B.E.T. method has been put to use by many investigators during the past few years.

The examples cited above were chosen from experiments made on solids that are non-porous. It is believed that V_m also gives the internal surface area of porous solids, provided that the molecules of the adsorbate are small enough to penetrate the pores and reach all parts of the surface. The fact that V_m is evaluated from data in the low pressure range only, and hence where capillary condensation in the pores is not a factor, supports this belief. With respect to cement paste, the pores are considered to be the spaces vacated by evaporable water when the sample is dried. The drying may be accompanied by irreversible shrinkage so that the

*6.023 x 10^{23} is now the accepted value for Avagadro's number. This did not come to our attention until all the computations were completed. Since other factors are uncertain, it did not seem worth while to make the slight corrections indicated.

dried sample may not faithfully represent the original paste. The extent of the irreversible alteration is not known but it is considered to be small. Evidence of this is found in the fact that the total pore space, as measured by the total evaporable water, is not greatly altered by the drying; in fact the data indicate that it is increased. (See Table 9, Part 2.) Hence, V_m is considered to give the internal surface area of the solid phase in the dried samples. If drying affects the measured surface area, it would probably be in the direction of making it smaller.

In the remainder of this discussion it is assumed that V_m as determined in this investigation is proportional to the surface area of the solid phase. It will be seen that this assumption leads to highly significant results. Thus the assumption seems justifiable.

Results from hardened paste

The magnitudes of the surface areas and the rates at which surface develops during hydration as computed from eq. (6) are indicated by Table 3-7.

The figures pertain to the whole solid phase; that is, they are based on the combined weights of the hydration products and residue of original clinker. Therefore, the specific surface of the hydration products is higher than the highest figure given except any that might represent completely hydrated cement.

Specific surface in terms of w_n

Since $V_m = kw_n$, the specific surface of a paste can be computed if w_n is known, and if k for the particular cement is known. That is,

$$\frac{S}{c} = 35.7 \times 10^6 \, k \, \frac{w_n}{c} \qquad (7)$$

$$\frac{S}{w_n} = 35.7 \times 10^6 \, k \qquad (8)$$

Also

$$\frac{S}{c + w_n} = \frac{35.7 \times 10^6 \, k \, w_n}{c + w_n} = \frac{35.7 \times 10^6 \, k \, w_n/c}{1 + w_n/c} \qquad (9)$$

Eq. (7) gives the surface area per unit of original cement, and eq. (9) gives it per unit of dry paste.

For the types of cement given in Table 3-6 the surface area per unit weight of non-evaporable water is as given in Table 3-8.

The non-evaporable water content may lie anywhere between zero and about 0.25 g per g of cement. Hence, according to these equations the specific surface may lie between zero and about 2.1 to 2.5 million sq. cm. per g of original cement.

TABLE 3-7—TYPICAL FIGURES FOR SPECIFIC SURFACE OF HARDENED PASTE

Period of hydration, days	Specific surface of paste for cements indicated; w_c/c = 0.45 (approx.)							
	14930J C_3S 23% C_3A 6%		15761 C_3S 45% C_3A 10%		15365 C_3S 45% C_3A 13%		15013J C_3S 40% C_3A 14%	
	Millions of sq. cm. per g of:							
	cement	dry paste	cement	dry paste	cement	dry paste	cement	dry paste
7	0.76	0.71	—	—	1.21	1.06	1.32	1.13
14	1.02	0.92	—	—	1.55	1.32	1.50	1.28
28	1.33	1.19	1.75	1.48	1.94	1.64	1.71	1.46
56	1.75	1.54	—	—	1.96	1.62	1.89	1.57
90	1.89	1.62	1.96	1.62	2.02	1.66	2.04	1.67
180	2.10	1.78	—	—	2.14	1.75	2.07	1.69
365	2.10	1.76	—	—	—	—	—	—

TABLE 3-8—SPECIFIC SURFACE OF HARDENED PASTE IN TERMS OF NON-EVAPORABLE WATER CONTENT

Type of cement		V_m/w_n (= k)	S/w_n
Type I	Normal C_3A	0.261	9.3×10^6
	High C_3A	0.256	9.2×10^6
Type II	High iron	0.259	9.3×10^6
	High silica	0.279	10.0×10^6
Type III	Normal C_3A	0.238	8.6×10^6
Type IV	High iron	0.282	10.1×10^6
	High silica	0.277	9.9×10^6

Of course it cannot be literally true that $S = 0$ when $w_n = 0$, since the initial surface area is that of the original cement. As measured by adsorption, the specific surface of unhydrated cement is much higher than that measured by methods previously used. Using a cement having a specific surface of 1890 sq. cm. per g (Wagner), Emmett and DeWitt[34] found a surface area of 10,800 sq. cm. per g by the nitrogen-adsorption method. By the air-permeability method, this cement would show about 3500 sq. cm. per g, which is probably close to the true *macroscopic* surface area. The difference between the macroscopic surface area and that as measured by adsorption might be due to microscopic, or submicroscopic, cracks in the clinker grains. Surfaces of such cracks would be measured by the adsorption method, but not by the other. Since such

cracks have not been commonly reported, it seems more likely that the difference is due to a slight coating of hydration products on the grain surfaces. As shown above, an average cement shows about 9.3×10^6 sq. cm. per g of non-evaporable water. Hence, to account for the 7400 sq. cm. difference between the two results, it is only necessary to assume that the cement had hydrated to the extent of

$$\frac{7400}{9.3 \times 10^6} = 800 \times 10^{-6} \text{ g of non-evaporable water per g of cement,}$$

or 0.08 percent of the weight of the cement. Such a small amount of hydration could easily occur during the normal handling of a sample during humid weather.

Whether the true surface area of the cement is of the order of 3000 or 10,000, it is clear that the initial surface area is negligible compared with that which finally develops.

THE SPECIFIC SURFACE OF STEAM-CURED PASTE

The effects of high-temperature steam-curing on the adsorption characteristics of cement paste were shown in Part 2, p. 300. In terms of the B.E.T. theory, the effects are as follows:

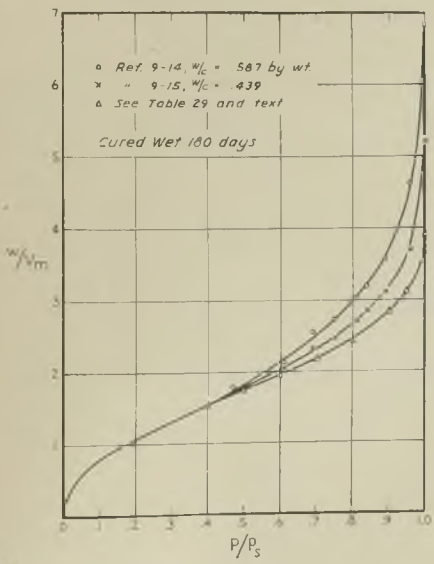
	Ref. 14-4 Normal curing	Ref. 14-6 Steam curing
C	15.4	20
V_m, g/g of cement	0.037	0.0020
V_m, g/g dry paste	0.032	0.0018
Sp. surface, sq. cm. per g of cement, millions	1.32	0.071
Sp. surface, sq. cm. per g of dry paste, millions	1.15	0.062
V_m/w_n	0.241	0.012
w_n, g/g of cement	0.1537	0.1615

The figures for V_m and specific surface of the steam-cured specimen are probably not very accurate because of the extreme smallness of the amounts of water taken up in the low-pressure range. The order of magnitude relative to the normally cured material is probably correct, however. The result indicates that all but about 5 percent of the colloidal material was converted to the microcrystalline state by high-temperature steam curing.

w/V_m CURVES

The samples of hardened paste used in these studies contained undetermined quantities of unhydrated material. Consequently, the weight of the adsorbent material could not be ascertained directly, a circumstance that increases the difficulty of interpreting the adsorption data. The problem was simplified by expressing the amount of adsorption, w, in terms of the surface area of the solid phase. Since the surface area is proportional to V_m, the ratio w/V_m could be used without computing the surface area.

Typical w/V_m curves are shown in Fig. 3-8. The uppermost curve represents the paste in a mortar specimen having $w/c = 0.587$, cured six months; the middle curve represents the paste from a richer mortar specimen of the same age, $w/c = 0.439$. The lowest curve represents the data given in the first group of Table 3-9. These data include water-cement ratios ranging from 0.12* to 0.32 by weight. The table shows that after long periods of curing and for w/c within this range, w/V_m is virtually the same for all samples at all vapor pressures. (In the lower range of pressures, w/V_m is always the same for all samples except for the effect of differences in C.†) The triangular points plotted in Fig. 3-8 represent the average values from this group.



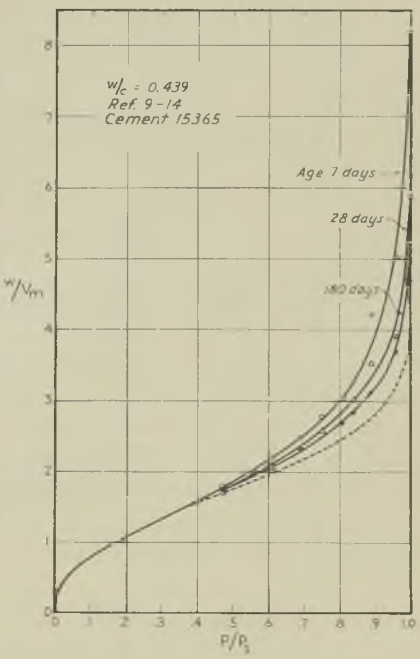


Fig. 3-8 (above)—Effect of original w/c on w/V_m curves

Fig. 3-9 (right)—Effect of wet-curing on w/V_m curves

*This very dry paste was molded by means of a press.
†See discussion in Part 4: "Significance of C of the B.E.T. Equation."

TABLE 3-9—TYPICAL w/V_m DATA

Data obtained by air-stream method, except as noted, and taken from plotted curves.

Ref. No. 254-	Cement No.	Net w/c	Age, days	C	V_m, g/g dry paste	w/V_m at value of p/p_s indicated								v_l‡	w_n/c
						0.4	0.5	0.6	0.7	0.8	0.9	0.95	SSD†		
Samples for which V_{max} = about $4V_m$															
8-1	14930J	—	447	21	.045	1.56	1.66	1.82	2.05	2.32	2.82	3.07	3.99	.88	.1807
9-1	14930J	.309	365	21	.043	1.52	1.72	1.90	2.12	2.40	2.82	3.10	3.89	.86	.1704
9-1	14930J	.309	180	21	.042	1.52	1.72	1.90	2.12	2.40	2.85	3.10	3.88	.86	.1620
9-4	15007J	.316	180	16	.041	1.48	1.72	1.96	2.23	2.42	2.88	3.15	3.99	.87	.1835
9-10	15013J	.324	180	29	.043	1.55	1.80	2.00	2.17	2.40	2.75	2.95	3.73	.86	.1877
9-13	15365	.319	180	25	.039	1.52	1.77	1.98	2.25	2.52	2.92	3.20	4.25	.87	.1815
9-13	15365	.319	90	20	.041	1.55	1.75	1.98	2.20	2.47	2.92	3.20	4.02	.87	.1789
9-15A	15365	.244	180	33	.031	1.57	1.80	2.00	2.25	2.50	2.75	3.18	3.58	—	—
9-15A*	15365	.244	180	33	.031	1.55	1.77	2.02	2.25	2.50	2.90	3.20	—	—	.1554
254-18*	14675	.12	4 yr.	92	.019	1.40	1.60	1.80	2.05	2.30	2.75	3.10	3.9	—	—
Avg.						1.53	1.74	1.94	2.17	2.43	2.84	3.12	3.91	.867	
Samples for which $V_{max} > 4V_m$															
9-14	15365	.439	180	19	.049	1.50	1.75	2.04	2.30	2.65	3.17	3.60	5.22	.89	.2221
9-14	15365	.439	28	19	.046	1.52	1.80	2.08	2.35	2.75	3.40	3.90	5.85	.90	.1841
9-14	15365	.439	7	26	.030	1.55	1.85	2.15	2.55	3.05	4.00	5.25	8.20	.91	.1400
9-15	15365	.587	180	19	.048	1.55	1.80	2.04	2.34	3.16	3.20	3.65	5.23	—	—
254-18	13495	.44§	7 yr.		.055	1.55	1.75	1.95	2.20	2.50	2.90	3.25	4.20	—	—

*Data obtained by high-vacuum method (not included in averages)
†Evaporable water content of saturated, surface-dry samples.
‡See Part 5.
§Estimated.

In Fig. 3-9 the effect of prolonging the period of wet-curing on a given paste is shown together with the lowest curve of Fig. 3-8 for comparison.

Minimum porosity and the cement-gel isotherm

Considering Fig. 3-8 and 3-9 together we may conclude that the densest paste possible contains a pore-volume equal to the volume of the quantity of adsorbed water represented by about $4V_m$.

The shape of the lowest curve in Fig. 3-8 also seems to represent a limit that is approached as the pastes are made denser. Hence, for brevity we will call the lower curve of Fig. 3-8 the *cement-gel isotherm* or just gel-curve, when the meaning is clear. The part of the total evaporable water equal to $4V_m$ will be called *gel-water*.

When a paste is such that at saturation it contains a quantity of evaporable water equal to $4V_m$, we may infer that all the originally water-filled space has become filled with porous hydration products. Thus, in such a paste the space outside the unhydrated clinker residue has only the porosity of the cement-gel itself.

When a paste is such that at saturation it contains a quantity of water exceeding $4V_m$, the excess over $4V_m$ is believed to occupy residual space outside the cement-gel. Water occupying this space is called *capillary water* in this discussion. It should be understood that this distinction between capillary water and gel-water is arbitrary, for some of the gel-water may be taken up by capillary condensation and is thus not different from the rest of the capillary water so far as the mechanism of adsorption is concerned. The distinction is justified by the fact that, in a saturated paste, the quantity of gel-water always bears the same ratio to the amount of gel, whereas the water called capillary water can be present in any amount according to the porosity of the paste as a whole.

Fig. 3-8 and 3-9, which are typical of all other w/V_m curves obtained, show that among various samples any increase in pressure up to about 0.45 p_s is always accompanied by approximately the same increment of adsorption, regardless of differences in porosity.* From this we may infer that the capillaries (the spaces outside the gel) do not begin to fill at pressures below about 0.45 p_s. At higher pressures, however, a given increment in pressure will be accompanied by an increment of adsorption that is larger the greater the porosity of the sample. This may be taken as direct evidence of capillary condensation. The amount of water held by capillary condensation at any given pressure is represented by the vertical distance of the point in question above the gel-curve.

These ideas can be represented by a model such as is illustrated in Fig. 3-10. In A the shaded areas represent cross sections of spherical

*Such differences as there may be are due to differences in C of eq. (A), as is explained in Part 4.

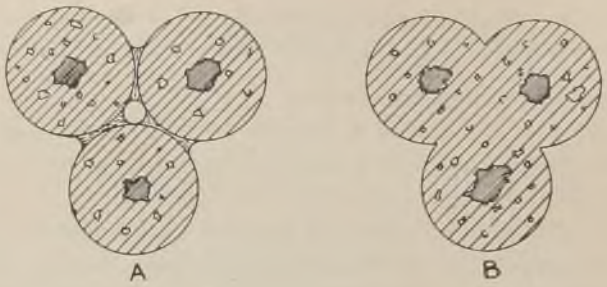


Fig. 3-10

bodies of cement gel with non-colloidal particles (microcrystalline hydrates and unreacted cement) embedded in them. Between the bodies is interstitial space containing capillary-condensed water, here pictured as lenses around the sphere-to-sphere contacts. The water content of the system is assumed to be below saturation, as indicated by the curvature of the lenses. At saturation, the interstitial space would be filled with capillary water, making the total water content equal to $4V_m$ plus the volume of capillary water.

Thus, if we consider the system pictured in Fig. 3-10A to be the sample represented by the upper curve of Fig. 3-8, at equilibrium with the pressure $p = 0.8\ p_s$, the water content of the spheres (the gel-water) would be 2.44 V_m, the capillary water would be 0.56 V_m, making a total of 3.00 V_m. At saturation the gel-water would be $4V_m$ and the capillary water 2.83 V_m, making a total of 6.83 V_m.

Fig. 3-10B represents a paste at the same stage of hydration as that represented in 3-10A but with a lower original w/c. Here the spheres of gel have merged into one body, eliminating all capillary water. The lowest w/V_m curve in Fig. 3-8, the cement-gel isotherm, would correspond to this case.

Estimation of pore- and particle-size

With data obtained from specimens containing no capillary space (as defined above) we can estimate the order of size of the elements of the solid phase and of its characteristic pores. Pore-size can be estimated from the hydraulic radius

$$\text{Hydraulic radius} = m = \frac{\text{volume of pores}}{\text{area of pore-walls}}$$

Pore volume is the space occupied by evaporable water and, in a paste without capillary space, this is equal to the volume of the gel water, i.e., the volume of $4V_m$. The area of the pore-walls is given by eq. (6).

$$S = 35.7 \times 10^6\ V_m$$

Hence,

$$m = \frac{4V_m v_g}{35.7 \times 10^6 V_m}$$

where

v_g = specific volume of the gel-water.

It will be shown later than the specific volume of the gel-water is about 0.90. Hence,

$$m = \frac{4 \times 0.90}{35.7 \times 10^6} = 10.01 \times 10^{-8} \text{ cm}$$

or approximately 10Å.*

The average size of a pore in the gel having a given hydraulic radius can be estimated by assuming that the cross section of the pore resembles a rectangular slit. Let b, h, and L be the width, thickness, and length, respectively, of the slit. Then

$$m = \frac{hbL}{(2h + 2b)L} = \frac{hb}{2(h + b)}$$

Solutions of this equation for various values of h and b are given below:

$h = b$; $m = 1/4\ b$
$h = 2b$; $m = 1/3\ b$
$h = 4b$; $m = 4/10\ b$
$h = 10b$; $m = 10/22\ b$
$h = 100b$; $m = 100/202\ b$

Thus, as h/b is made larger, m approaches ½ b as a limit. This means that the width of the pores is at least twice and at most four times the hydraulic radius. Since m was given as about 10Å, it follows that the average pore is from 20 to 40Å across, probably closer to 40Å than to 20Å if the particles are other than spherical.

The order of size of the colloidal particles cannot be computed directly because there is no way to correct for the volume of non-colloidal material, i.e., microcrystalline hydrates and unhydrated clinker. However, it is of interest to estimate the size without such correction since the volume of non-colloidal hydrates is relatively small and data are available for samples containing very little unhydrated clinker. The estimate is made by finding the size of spheres in an aggregation of spheres that would have the same total volume and surface area as dry hardened paste. The size of sphere is given by the relationship

$$r = \frac{3}{S'}$$

*Å = Ångstrom unit = 10^{-8} cm

where

r = radius of sphere in cm

S' = surface area in sq. cm per cu. cm

$$S' = d_p S$$

where

d_p = density of dry paste, g per cu. cm

and S = surface area of dry paste, sq. cm per g.

For a typical paste cured at least 6 mo.,

d_p = 2.44 g per cu. cm.

S = 1.8 x 10^6 sq. cm. per g

$$r = \frac{3}{2.44 \times 1.8 \times 10^6} = 68 \times 10^{-8} \text{ cm},$$

say 70Å.

This indicates that if the solid phase were an assemblage of equal spheres, each sphere would have a diameter of about 140Å. The units of colloid material are probably smaller than this, but not very much smaller since most of the hydration product is colloidal and since there was probably little unhydrated material in the specimen on which this estimate is based.

These figures give a picture of a material made up of solid units averaging about 140Å in diameter, with interstices averaging say 20 to 40Å across. This should, of course, be taken only as an indication of the order of size of the elements of the fine structure. It indicates, for example, the necessary resolving power of a microscope capable of differentiating these features of hardened paste.

* * * * *

It is hardly necessary to add that the authors hold no belief that the gel develops as spheres or that the pores are rectangular slits of uniform cross section. Those assumptions were made only for convenience of illustration and computation. The bodies of hardened gel could be in the form of submicroscopic plates, filaments, prisms, or of no regular form at all. However, as developed above, the evidence points to the conclusion that the gel is a solid having a characteristic porosity.

Data on relative amounts of gel-water and capillary water

The relative amounts of gel-water and capillary water in saturated samples at various stages of hydration are shown in Fig. 3-11 and 3-12, for the materials of Series 254-9. The shaded portion of each column represents gel-water and the open portion capillary water. These charts bring out again the fact that for specimens of sufficiently low water-cement ratio, prolonged curing eliminates all capillary water.

In several instances there is an indication that the ratio of capillary to gel-water *increases* after a minimum is reached. Whether this is real or the result of experimental vagaries cannot be told without further experiment. If it is real, it might be due to the leaching of soluble material from the paste during the curing period. Such leaching would be expected to increase the porosity of the paste. It might also be due to a coarsening of the gel-texture by the formation of microcrystals at the expense of colloids. If so, the change is of considerable significance. Present data warrant no conclusions on this point.

There is a rather definite indication that the gel is able to fill but a limited amount of space, regardless of the length of the curing period. This is brought out in Fig. 3-13, where w_e/V_m (w_e = evaporable water) at saturation is plotted against w/c for all specimens cured 180 days or longer. This shows again that the minimum possible evaporable water content is about $4V_m$ and further that all samples having original water-cement ratios greater than about 0.32 by weight will contain some capillary water. The empirical relationship illustrated can be represented approximately by the equation

$$\left.\frac{w_c}{V_m} = 12.2\,(w/c - 0.32)\right] \quad w/c \geqq 0.32$$

where

w_c is the capillary-water content of specimens cured 6 months or more.

SUMMARY OF PART 3

Adsorption isotherms for water on hardened portland cement pastes show the same characteristics as those for vapors on many different organic and inorganic materials.

The process of adsorption and the conditions for equilibrium are explained in terms of the Brunauer-Emmett-Teller (B.E.T.) theory and the capillary condensation theory.

The B.E.T. eq. (A) is used for representing data over the range p = 0.05 p_s to 0.45 p_s. The equation is

$$\frac{w}{V_m} = \frac{cx}{(1 - x)\,(1 - x - cx)} \qquad \text{(A)}$$

where

w = weight of evaporable water held at equilibrium with pressure p,
V_m = quantity of water required for a complete condensed layer on the solid, the layer being 1 molecule deep on the average,
C = a constant related to the heat of adsorption,
$x = p/p_s$ where p_s = saturation pressure and p the existing pressure.

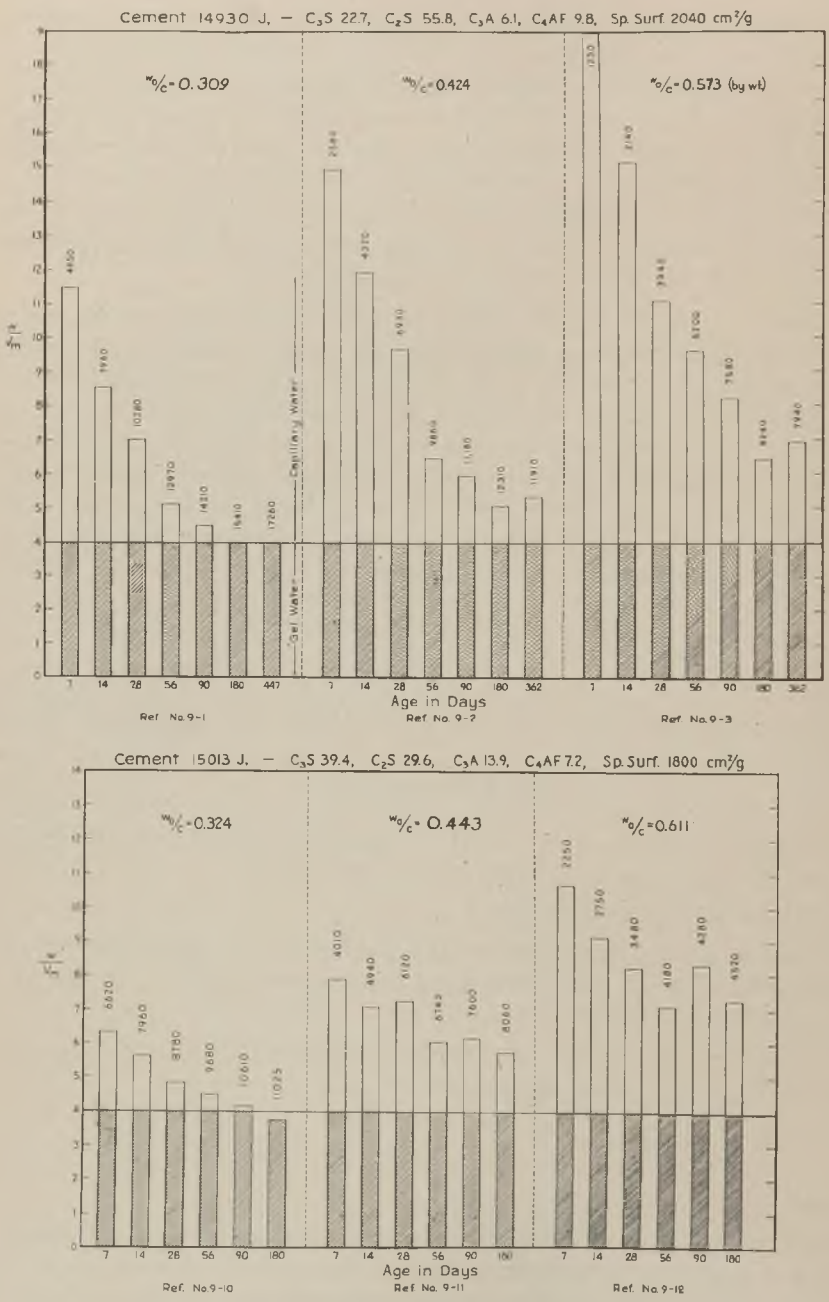


Fig. 3-11—Effect of hydration on w/V_m

Note: Figures over columns are compressive strengths of 2-in. cubes, psi.

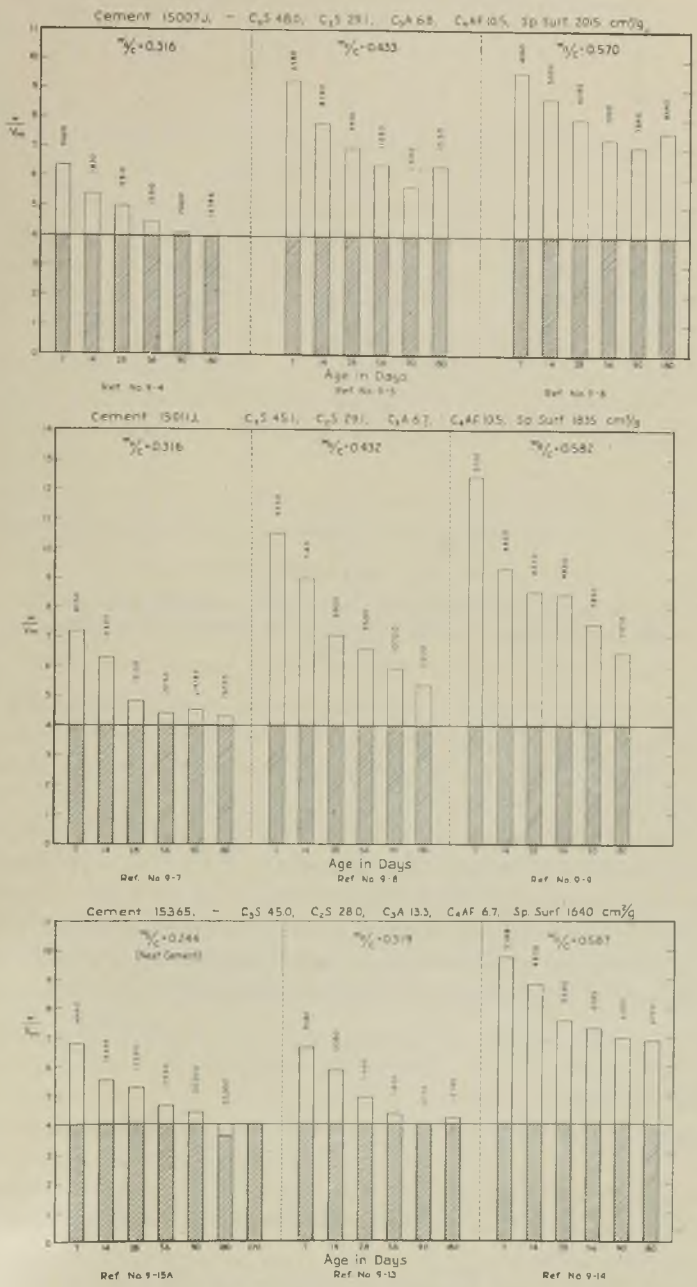


Fig. 3-12—Effect of hydration on w/V_m

Note: Figures over columns are compressive strengths of 2-in. cubes, psi.

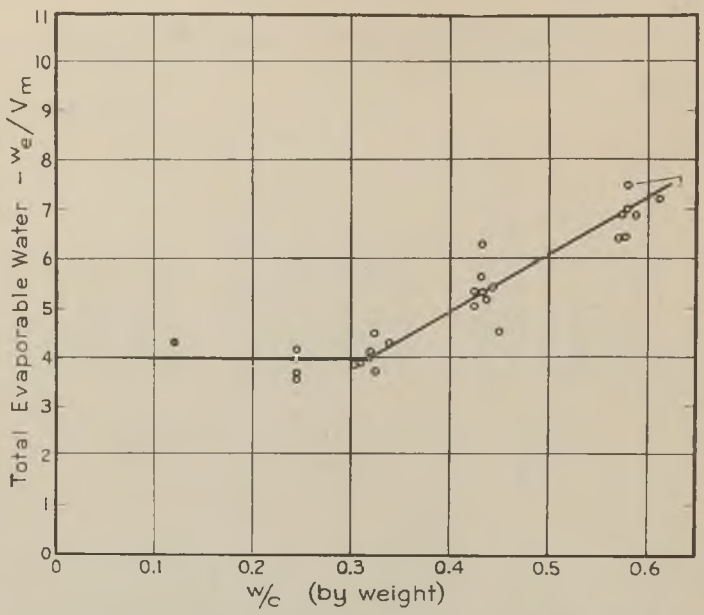


Fig. 3-13—Empirical relationship between total evaporable water per unit V_m and original water-cement ratio for samples cured 180 days or longer

V_m and C can be readily evaluated from experimental data and p_s is a constant depending on temperature. C is about the same for all pastes. Hence differences in adsorption characteristics are indicated by differences in V_m.

The non-evaporable water content, w_n, is regarded as proportional to the total amount of hydration products. Since V_m is porportional to surface area and since practically all the surface is that of the colloids, V_m is considered to be proportional to the colloidal material (gel) only.

The ratio V_m/w_n is considered to be a constant for any given cement. It is influenced by compound composition about as follows:

$$\frac{V_m}{w_n} = 0.00230\ (\%C_3S) + 0.00320\ (\%C_2S) + 0.00317\ (\%C_3A) + 0.00368\ (\%C_4AF)$$

Among the different types of cement V_m/w_n varies from about 0.24 to 0.28.

The above equation implies that the hydrate of each compound is colloidal or at least that all compounds occur as constituents of a complex colloidal hydrate.

The specific surface of hardened paste can be computed from the relationship $S = 35.7 \times 10^6\ V_m$. It increases with the period of curing and reaches about 2 million sq. cm per g of original cement.

The specific surface of the hardened paste is related to w_n as follows:

$$\frac{S}{w_n} = 35.7 \times 10^6 k,$$

where k is a constant for a given cement. Among the different types of cements, S/w_n ranges from about 8.6×10^6 to 10×10^6.

None of the relationships given above apply to paste cured at high temperature. Under steam pressure a sample cured 6 hours at 420 F showed only 0.07×10^6 sq. cm. of surface per g of cement, as compared with 1.3×10^6 for a paste cured 28 days, or about 2.0×10^6 for long curing, at room temperature.

When adsorption data are expressed in terms of w/V_m and p/p_s, the result is an isotherm based on the relative amount of gel. Such curves are virtually identical for all cement pastes over the pressure range $p = 0.05\ p_s$ to $0.45\ p_s$.

For pastes in which the total evaporable water content is about $4V_m$, the curves are identical for the whole pressure range.

For pastes having capacity for evaporable water exceeding $4V_m$, the excess is taken up over the pressure range $p = 0.45\ p_s$ to $p = p_s$.

The evaporable-water capacity is smaller the lower the original water-cement ratio and the longer the period of curing, but it cannot be reduced below about $4V_m$.

Evaporable water in excess of $4V_m$ is believed to occupy interstitial space not filled by gel or other hydration products. The water in this space is called *capillary water*. The rest of the evaporable water is held within the characteristic voids of the gel and is called *gel-water* even though some of it might have been taken up by capillary condensation.

When the total evaporable-water capacity $= 4V_m$, the specimen contains no space for capillary water.

From the surface area of the solid phase, and its characteristic porosity, the average pore in the densest possible hardened paste is estimated to be from 20 to 40Å across.

From the volume of the solid phase and its surface area, the order of particle size, expressed as sphere diameter, is estimated at 140Å.

Data on the relative amounts of gel-water and capillary water in various samples are given graphically.

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ACI News Letter

Vol. 18 No. 4 JOURNAL of the AMERICAN CONCRETE INSTITUTE December 1946

In the foregoing Proceedings pages of this Journal you will find, this month:

News Letter Contents this Month

43rd Annual Convention, February 24-26, 1947

The Cincinnati convention program under development by the Publications Committee approaches crystallization in an array of timely and important papers and reports. Some items are emerging; others are in the making; some still to be selected. An excellent program seems assured.

An important factor of "team rivalry" is again being used toward a top notch final program within time limitations. Five sessions are to be built from the results of the present exploration of a number of highly discussable subjects. The final choice of convention offerings will be made by the committee from the contributions *in hand before* mid-January. For the very earliest (the limit, *early December*), pre-convention publication may be important for the development of that discussion which adds so much to meeting-room interest. Other material too late for preview before final program decisions will of course be considered promptly for subsequent JOURNAL publication.

Thus, as this goes to the printer (November 22) for the December JOURNAL, it seems possible to announce that the Institute's publication program for the next half year (including the convention program and other papers and reports too late for the convention) will include the following:

Building code

The report of Committee 318, Standard Building Code, (p. 401 this JOURNAL) presenting proposed revision of "Building Regulations for Reinforced Concrete (ACI 318-41)", always of widespread interest because the "ACI Code" sets the pattern for so many building laws in this country and abroad, will have a prominent spot on the program. As announced by Chairman Boase and Secretary Zipprodt for Committee 318, the present proposed revisions are chiefly matters of simplification and clarification. In the current publication of the report, convention consideration is assured.

Houses

Technological advances in the application of concrete to the solution of the pressing nation-wide housing problem have spurred the present effort to bring the record to date in short, timely papers on this subject. Several are promised. It is hoped some will be available in time to be scheduled for the Cincinnati meeting. In any event they seem certain to find a place in JOURNALS of early 1947 and in turn inspire discussion with revelation of further facts of large public importance.

Grouting

Several aspects and applications of the techniques of grouting have the current attention of considerable talent in writing papers on foundation

grouting, contraction grouting in large concrete dams, oil well grouting, tunnel grouting, and possibly, railroad track grouting.

Bond

Two papers seem to be assured on bond of reinforcing steel, with another one or two in prospect.

Destructive agents

Several prospective papers are to cover such aspects of durability of concrete as: durability of concrete as influenced by cement, aggregates as a destructive agent, destructive agents as contained in sewers and water supplies, sulfate resistant cement, and possibly, the effect of sea water on portland cement concrete.

Concrete surfaces

A session is proposed on concrete surfaces with papers on cracks, erosion studies and their corrective measures, abrasion, painting, precast ornamental concrete, finishing air-entraining concrete, and floors for hard wear.

Precast concrete

For a proposed session on precast concrete, papers are expected on precast concrete units, precast concrete in building construction and air entrainment in concrete masonry units, and possibly a paper on a concrete block designed to be used like sheet piling.

Research

Based on its popularity in recent years, the Research Session organized by Prof. S. J. Chamberlin, secretary, Committee 115, is considered a "Must."

* * * * *

Although Cincinnati hotel facilities are believed to be ample, and the headquarters hotel, the Netherland Plaza, promises full cooperation toward individual accommodations, Members and others planning to attend the convention should do their part with early requests for reservations. This is the first meeting in Cincinnati since 1923 and it is expected that many will take advantage of its central location to attend the meetings; become acquainted with the Institute and its long-time regulars. Members from the mid-south especially are urged to attend, those who have not always had as convenient an opportunity to attend an ACI convention without a long trip.

More positive announcements of the convention program in the January ACI News Letter.

ACI Awards Announced

Acting on the report of its Medals Award Committee, the Board of Direction announces:

To Morton O. Withey, Dean of the College of Engineering, University of Wisconsin:* the Henry C. Turner Medal (founded in 1927 by Past President Turner) "to be awarded annually but not more often, for notable achievement in or service to" the field of concrete.† The Medal is of gold and is accompanied by a certificate of award which, in the present instance will bear this citation:

> *"in recognition of forty years of outstanding contributions to our knowledge and understanding of concrete and reinforced concrete."*

To Gerald Pickett, Professor of Applied Mechanics, Kansas State College*, the Leonard C. Wason Medal for the year's "most meritorious paper" on the basis of his

"Shrinkage Stresses in Concrete"

published in the ACI JOURNAL January and February 1946. The bronze medal, suitable inscribed, is accompanied in its presentation by a certificate of the award.

To Charles E. Wuerpel, Chief of the Concrete Research Division, U. S. Waterways Experiment Station, Clinton, Miss.*, the Leonard C. Wason Medal for noteworthy research as reported in his paper

> *"Laboratory Studies of Concrete Containing Air-Entraining Admixtures"*

published ACI JOURNAL, February 1946. The bronze medal, suitably inscribed, is accompanied in its presentation by a certificate of the award.

The Wason Medals were founded by Past President Wason (the Institute's second President 1915-16) in 1917.

To J. W. Kelly* and B. D. Keatts*, associate professor of civil engineering, University of California and engineer, Intrusion-Prepackt, Inc., respectively, the second American Concrete Institute Construction Practice Award, founded in 1944 "for a paper of outstanding merit on concrete construction practice." The award is based on the paper entitled

> *"Two Special Methods of Restoring and Strengthening Masonry Structures"*

ACI JOURNAL, February, 1946. Messrs. Kelly and Keatts will each receive a certificate of award and will share equally in U. S. Series E bonds of a maturity value of $300.00.

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*See brief biographical sketch in pages which follow.

†Previous Turner Medalists: Arthur N. Talbot 1928; William K. Hatt 1929; Frederick E. Turneaure, 1930; Duff A. Adams 1932; John J. Early 1934; Phaon H. Bates 1939; Ben Moreell 1943; John L. Savage 1946.

The presentation of these awards will be from the hands of President Gonnerman at the Institute's 43rd Annual Convention at Cincinnati, February 24 to 26, 1947.

The Medalists

Morton O. Withey—Turner Medal

Morton O. Withey was born in Meridian, Conn. October 25, 1882. In 1904 he received his B.S. degree from Dartmouth College and in 1905 his civil engineering degree from Thayer School of Civil Engineering (connected with Dartmouth College).

During his senior and post-graduate years at Dartmouth, he was an assistant in graphics and in surveying and for four months in the summer of 1904 an apprentice at the North Works of the Illinois Steel Co., Chicago.

Dean Withey has been associated continuously with the University of Wisconsin since September 1905, beginning as an instructor and holding successively positions of assistant professor, associate professor, professor, chairman of the department of mechanics and since June 29, 1946, Dean of the College of Engineering.

Aside from his teaching Dean Withey has been active in research in the fields of masonry materials, cement, concrete, reinforced brickwork, masonry and mortar. From 1924 to 1931 he was in charge of the research program on structural steel columns for the steel column research committee of the American Society of Civil Engineers. During the years 1926 to 1937 he served as chairman of the committee on yield points and structural steel of the A.S.T.M. and from 1932 to 1940 he conducted research on properties of reinforced brick masonry and masonry mortar suitable for such masonry.

More recently he has been chairman of the committee on durability of concrete of the Highway Research Board which has been engaged in an experimental program to standardize freezing and thawing procedures to be used in testing concrete.

Dean Withey has been a member of the Institute since 1921, has been a member of the Advisory Committee since 1938, member of the Board of Direction since 1937 as Director, 1937-38, Director-at-Large, 1939-40, Vice-President 1941-42, and President in 1943, serving as a past president member of the Board since 1944.

Dean Withey holds membership in several technical organizations in addition to the ACI, including Wisconsin Society of Professional Engineers (past president), American Society for Testing Materials, Society for the Promotion of Engineering Education, and the National Society of Professional Engineers.

Gerald Pickett—Wason Medalist

Gerald Pickett was born August 29, 1901; was graduated as an electrical engineer from Oklahoma A. & M. in 1927, awarded a PhD in Mechanics by the University of Michigan in 1938. Except for two years in the testing laboratory of the Brooklyn Navy Yard, he taught in the Department of Mechanics, Kansas State College, until 1940. In that period he taught advance courses in mechanics to graduate students in addition to the regular courses to the undergraduates.

In March 1940 he joined the Basic Research Bureau of the Portland Cement Association's Research Laboratory in Chicago as a research physicist. In this capacity his studies centered around the mechanics of portland cement concrete, particularly its behavior under sustained stress. On this subject he was the author of a paper appearing in the February 1942 ACI JOURNAL "The Effect of Change in Moisture Content on the Creep of Concrete under a Sustained Load."

He appeared in the ACI JOURNAL April 1945, with his "Dynamic Testing of Concrete." The paper which won him the Wason Medal for the "most meritorious paper" in Vol. 42 of the ACI *Proceedings* appeared in two parts, January and February 1946 ACI JOURNAL and was entitled "Shrinkage Stresses in Concrete."

In September 1945 Doctor Pickett returned to the faculty of Kansas State College, Manhattan, Kansas, where he is Professor of Applied Mechanics.

Charles E. Wuerpel—Wason Medalist

Charles E. Wuerpel was born in Louisiana, July 26, 1906. He began his specialization in concrete when given responsible charge of the construction of the concrete section of Bonnet Carre Spillway in 1929. Following its completion he had charge of concrete and concrete testing on the New Harvey Lock of the Intracoastal Canal at New Orleans. This was followed by a resident engineership at the Vermilion Lock Project on the Intracoastal Canal in southwestern Louisiana.

While in charge of concrete during the construction of Lock 26 at Alton, Ill., he conducted a research program dealing with the effect of fineness of cement upon the temperature rise in semi-mass concrete. He was later placed in charge of the Concrete Research Laboratory in connection with the proposed Passamaquoddy Tidal Power Project during the comprehensive research on resistance of concrete to sea water and severe frost action.

Subsequently he was in charge of the Central Concrete Laboratory of the North Atlantic Division of the U. S. Engineer Department, U. S. Military Academy, West Point, N. Y. where in addition to testing the concrete placed by the U. S. Engineer Department in northeastern United States, Mr. Wuerpel assisted in the instruction of the graduating class in concrete testing and laboratory design.

He was the author of an ACI paper entitled "Tests of the Potential Durability of Horizontal Construction Joints" which appeared in the January 1939 JOURNAL. In recent years he has done considerable research work with air entrainment and is the author of "Field Use of Cement Containing Vinsol Resin" (Sept. JL. 1945) as well as the paper which earned him the Wason Medal.

The Central Concrete Laboratory was recently moved to the U. S. Waterways Experiment Station in Clinton, Miss. and is now known as the Concrete Research Division, U. S. Waterways Experiment Station. Mr. Wuerpel is chief of the new Division.

J. W. Kelly and B. D. Keatts—ACI Construction Practice Award

This is not the first time J. W. Kelly has received an ACI award. In 1934 he received the Wason Medal as co-author with R. E. Davis, R. W. Carlson and G. E. Troxell for the paper "Cement Investigations for Boulder Dam With Results Up to the Age of One Year."

He was graduated from Purdue University in 1921 following three years of sub-professional work in construction

and two years as a first lieutenant of infantry in World War I. For two years he was engaged in waterworks engineering, making valuations and superintending construction. After a brief term as assistant in the testing materials laboratory of Purdue University, he joined the staff of the Portland Cement Association, where he remained for seven years. His work with the Association consisted largely in popularizing the intelligent design and field control of concrete mixtures by conducting "short courses" for engineers, architects, and contractors throughout the United States and Canada. Following a summer of field research and inspection for the National Sand and Gravel Association, he returned to Purdue University where as concrete specialist of the engineering extension department he prepared a practical booklet on concrete making for the small user of cement. In May 1931 he joined the staff of the University of California.

With Prof. R. E. Davis and other members of the staff, he is co-author of several papers on concrete, including the subject of volume changes, heat of cement hydration, cement investigations for Boulder Dam, and high-silica cements. As chairman of ACI Committee 611, he had major responsibility for the report published as "ACI Manual of Concrete Inspection." He is a member of the Institute and an associate member of the American Society of Civil Engineers.

B. D. Keatts was graduated from the University of Illinois in 1924 with the degree of BS in General Civil Engineering. His first work was in the planning and specification's office of the Illinois State Highway Department at Springfield for one year, and then with the bridge department of the Missouri Pacific Railroad at St. Louis where a small part of his time was spent in preparing plans for concrete structures; most of it was spent as resident engineer on bridge construction under contract.

He also had charge of maintaining the railroad's two river ferries for freight train service at St. Louis, Mo. and at Natchez, Miss. After four and a half years with the Missouri Pacific he went to Stone & Webster Engineering Corp. at Chicago for four years on the construction of equipment which was manufactured in the Chicago territory for installation in dams and industrial plants.

In the depression, he found haven with other engineers at the Century of Progress world's fair at Chicago, three years during the construction period, two years of operation, and part of the demolition period.

Next he was with the Portland Cement Associaton in the States of Illinois and Wisconsin making contacts with and instructing engineers, technicians, and builders in the construction field.

He has been with Intrusion-Prepakt, Inc. for six years in the field, locating proper sources of materials and getting them to the various jobs, establishing new field, laboratory-developed methods, and working with customers' engineers on special reconstruction problems. On January 1, 1946 he was given charge of the company's activities in the Eastern States.

Mr. Keatts has been an ACI member since 1942.

Mark your Calendar—

ACI's 43rd Annual Convention at Cincinnati February 24-26, 1947

New Members

The Board of Direction approved 86 applications for Membership (62 Individual, 4 Corporation, 5 Junior, 15 Student) received in October.

The Membership total on November 1, 1946, after adjustment for a few losses by death, resignation and for non-payment of dues, is 2927.

Individual Members

Allen, M. H., Structural Clay Products Institute, 120½ Welch Ave., Ames, Ia.

Basta, Rud F., 1 Queen Ave., Swanwyck, New Castle, Del.

Bellows, K. C., 10 S. Linden Ave., Sheridan, Wyo.

Berry N. K., c/o U. S. Bureau of Reclamation, Customs Bldg., Denver 2, Colo.

Bierman, Sidney, 7244 Shaftesbury, University City 5, Mo.

Bonilla, Petrus Manzano, Calle Seybo No. 21, C. Trujillo, Dominican Republic

Booth, James J., 1600-15th St., Denver, Colo.

Boyce, Vincent M., 619 W. 26th St., Wilmington 276, Del.

Braunbock, Ernst, Peter Jordanstr., 17/1, Vienna 19, Austria

Brewer, A. H., c/o Holabird & Root, Archts., 333 N. Michigan Ave., Chicago 1, Ill.

Brooks, Boyd S., 4827 Willett Pkwy., Chevy Chase 15, Md.

Castillo, Rene M., Calle Hostos No. 16, Ciudad, Trujillo, Dominican Republic, W. I.

Coornvelt, Harlan, 4648 N. Mervine St., Philadelphia 40, Pa.

Di Berto, Edward T., 132 Sharon St., W. Medford, Mass.

Downey, Paul W., 17 Gallatin St., N. W., Washington 11, D. C.

Duke, C. Martin, Pacific Islands Engineers, Contract NOj-13626, P. O. Box 33, c/o F. P. O., San Francisco, Calif.

Estelle, C., Jr., American Viscose Corp., 35 S. Ninth St., Philadelphia 7, Pa.

Estevez, Carlos Santos, Cerrado 356, Rpto Batista, Havana, Cuba

Field, William T., 20 Flower Bldg., Watertown, N. Y.

Foss, Ray J., Civil Engineering Dept., University of New Mexico, Albuquerque N. M.

Geymayr, Guido, c/o Sika Ltda., Casilla 1467, Santiago de Chile

Gelotte, Ernest N., 70 Alton Rd., Quincy 69, Mass.

Giardina, Anthony J., 206 De Mott Ave., Baldwin, L. I., N. Y.

Gomien, Randall L., 5401 Hetzel Ave., Cincinnati 27, Ohio

Gross, Morris H., Concrete Curing Corp., 8725 Puritan, Detroit 21, Mich.

Grout, Nathan M., Leo Elliott & Associates, Penthouse-Citizens Bldg., Tampa, Fla.

Hands, Stanley M., 501-5th Ave., Oakland 6, Calif.

Hanlon, J. R. G., c/o Reinforcing & Structural Steel Co. Ltd., Dunedin, New Zealand

Harpold, Allan E., 925 Frick Bldg., Pittsburgh 19, Pa.

Heres, Harold, 630 Louisiana Ave., Baton Rouge 10, La.

Hertz, A. L., U. S. Engineers, 2301 Grant St., Mobile, Ala.

Holley, Myle J., Jr., Room 1-251, M. I. T., Cambridge, Mass.

Holmes, W. H., P. O. Box 1079, Sacramento 5, Calif.

Horn, George P., Cementos Guadalajara, S. A., Apartado 1404, Guadalajara, Jal., Mexico

Kosman, Victor, c/o U. S. Bureau of Reclamation, Redding, Calif.

Lain, J. S., Lain Surfacing Co., 5057 Chatham St., Vancouver, B. C., Canada

Massey, Maurice Longfield, 85 Eddy Rd., Chatswood, N. S. W., Australia

McIver, A. W., P. O. Box 1945, Great Falls, Mont.

Medema, Melvin R., 301 Keller Bldg., Grand Rapids, Mich.

Mellor, Donald M., W. Va. Pulp & Paper Co., Piedmont, W. Va.

Merrill, B. S., 284 Shrine Bldg., Memphis, Tenn.

Moore, William, 110 Forsyth St., Boston, Mass.

Morris, Lloyd M., Physical Laboratory, Penn. R. R. Test Dept., Altoona, Pa.

Napp, Samuel, 110 W. 40th St., New York 18, N. Y.

Newhard, E. P., Pennsylvania-Dixie Cement Corp., Nazareth, Pa.

Porter, O. James, O. J. Porter & Co., 516 Ninth St., Sacramento 14, Calif.

Runyan, Damon O., Runyan & Slee, Longmont, Colo.

Sanders, A. L. R., 53 W. Jackson Blvd., Chicago 4, Ill.

Sen, B. R., Dept. of Civil Engineering, Iowa State College, Ames, Ia.

Sergeant, John E., Henger Construction Co., 1600 Dallas Nat'l. Bank Bldg., Dallas 1, Texas

Sperry, William C., 407 E. St., Copeland Park, Newport News, Va.

Starkmann, A., Nesher Cement Works, Haifa, Palestine

Steinbrugge, Karl V., 1304 Josephine St., Berkeley 3, Calif.

Sturlesi, Benjamin, c/o Neumann Bros., P. O. B. 1367, Jerusalem, Palestine

Summer, W. B., 534 Bridgeboro St., Riverside, N. J.

Thomas, T. W., Dept. of Highways Laboratory, Experimental Engr. Bldg., University of Minnesota, Minneapolis 14, Minn.

Torres, Ricardo G., 12 C. O. No. 3, Guatemala, C. A.

Towne, Arthur W. H., 222 Bedford Park Blvd., New York 58, N. Y.

Troeger, Maurice L., 1928—3rd St., N. E. Washington 2, D. C.

Weikel, S. F., Sandlass, Wieman & Associates, 107 S. Saratoga St., Baltimore 1, Md.

Weinberger, M. X. C., 701 Seventh Ave., New York 19, N. Y.

Williams, A. W., P. O. Box 314, Mobile, Ala.

Corporation Members

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Long, Forrest L. Jr., U. S. Engineer Dept., 1209 Eighth St., Sacramento, Calif.

Mendoza, Francisco, Villada 44, Toluca, Mexico, Mexico

Nettrour, B. F., 125 Ridge Ave., Pittsburgh 2, Pa.

Student Members

Arnold, Mack A., Jr., Syme Hall, Box 3579, N. C. State College, Raleigh, N.C.

Belgin, Adil, c/o 301 W. Engineering Bldg., Ann Arbor, Mich.

Brandley, Reinard W., Harvard University, Graduate School of Engr., Pierce Hall, Cambridge 38, Mass.

Bustamante, Guillermo Lara, P. O. Box 1644, San Jose, Costa Rica

Cartier, Morgan E. Jr., 254 Alumni Hall, Notre Dame, Ind.

Coyle, Robert W., 102 Sorin Hall, University of Notre Dame, Notre Dame, Ind.

Figuls, Jorge, Box No. 407, San Jose, Costa Rica

Fitzgerald, J. Edmund, 23 Prospect Ave., Revere, Mass.

Grippi, Vincent, 201 S. 9th Ave., Mt. Vernon, N. Y.

McCarthy, Francis Finney, 6154 Washington Ave., St. Louis, Mo.

Makzaumi, M. H., 621 S. Division, Ann Arbor, Mich.

Meltzer S., Abraham, Apartado 1770, San Jose, Costa Rica

Mueller, Edward A., 119 Morrissey Hall, Notre Dame, Ind.

Schoen, James M., 102 Sorin Hall, Notre Dame, Ind.

Williams, Dameron H. Jr., The Citadel, Charleston, N. C.

WHO'S WHO in this JOURNAL

Frank E. Richart

ACI past president who is the author of a paper in the JOURNAL, "The Structural Efficiency of Protection Shells on Reinforcement Concrete Columns" (p. 353) has been active in ACI since 1917 and needs no introduction here.

A. Amirikian

author of the paper, "Precast Concrete Structures," which appears on p. 365 of this JOURNAL has been an ACI member since 1945 and is chairman of the new ACI committee 324, Precast Reinforced Concrete Structures. Mr. Amirikian was graduated from Cornell University in 1923. After five years experience with various fabricating shops, he entered government service in the Bureau of Yards and Docks, Navy Department. Presently he is Head Designing Engineer, and for the past ten years he has been in charge of the special design section on buildings, welding, bombproofing, and floating structures. In this capacity he has been responsible for the design of some of the Bureau's most outstanding structures ashore and afloat. He is author of the design treatise "Analysis of Rigid Frames."

Arthur P. Clark

an ACI Member since 1924 and a member of ACI Committee 208, Bond Stress, is the author of the paper "Comparative Bond Efficiency of Deformed Concrete Reinforcing Bars" which appears on p. 381 of this JOURNAL.

Mr. Clark was graduated from the University of Michigan in 1903 with the degree BS in CE. Following graduation he spent three and one-half years in the Bridge Department of the Pere Marquette Railroad at Detroit and since that time has been continuously identified with the engineering problems of reinforced concrete and the distribution of concrete reinforcing bars. He joined the organization of the Corrugated Bar Co. at St. Louis in 1906 and has continued this association, through the consolidation of Corrugated Bar Co. with Kalman Steel Co. and the purchase of Kalman by Bethlehem Steel in 1931, until 1944. Since then he has been Research Associate for the American Iron and Steel Institute at the National Bureau of Standards.

In addition to the ACI, Mr. Clark holds membership in the A.S.C.E. and A.S.T.M., is an associate of the Highway Research Board and is serving on several industry committees.

T. C. Powers and T. L. Brownyard

are the authors of "Studies of the Physical Properties of Hardened Portland Cement Paste," Part 3 of which appears on p. 469 of this JOURNAL. See p. 8 of the October News Letter for biographical sketches of these authors.

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Vol. 18 which are currently available. Unless otherwise noted separate prints are 25 cents each. Starred ★ items are 50 cents or more, as indicated. Please order by title and title number.

REINFORCED CONCRETE COLUMNS UNDER COMBINED COMPRESSION AND BENDING..... 43-1

HAROLD E. WESSMAN—Sept. 1946, pp. 1-8 (V. 43)

Algebraic methods available heretofore for the analysis of the reinforced concrete column subject to combined compression and bending have usually involved the solution of a complex cubic equation and have taken considerable time when applied to particular problems. A new method of successive approximations converging rapidly to an exact answer and avoiding the use of the cubic equation is presented in this paper. The key to the method is the reciprocal relationship existing between the load axis and the neutral axis of the transformed section. The method may be applied to any shape of cross section and any arrangement of reinforcing steel, providing there is one axis of symmetry and the plane of bending coincides with this axis. The theory behind the method is presented and illustrated with three typical problems.

EFFECT OF MOISTURE ON THERMAL CONDUCTIVITY OF LIMEROCK CONCRETE..... 43-2

MACK TYNER—Sept. 1946, pp. 9-20 (V. 43)

The coefficient of thermal conductivity, *k*, of limerock concrete is a function of temperature, composition and density or moisture content. No attempt has been made to measure the effect of temperature on *k*. Holding the temperature reasonably constant, the effect of composition on *k* has been measured for two limerock concrete mixes (1:5 and 1:7 by volume). The 1:5 mix has a *k* that is 10 per cent larger than the *k* for the 1:7 mix.

With the temperature and composition held constant, the effect of moisture on *k* for the 1:5 and 1:7 mixes has been measured. The moisture content has a very profound effect on *k*, e.g. increases of moisture from zero to 5 per cent increases the *k* of 1:5 mix by 23 per cent and from zero to 10 per cent increases the *k* by 46 per cent. Concretes should be kept dry if their maximum heat insulation effect is desired.

★CEMENT INVESTIGATIONS FOR BOULDER DAM—RESULTS OF TESTS ON MORTARS UP TO AGE OF 10 YEARS..... 43-3

RAYMOND E. DAVIS, WILSON C. HANNA and ELWOOD H. BROWN—Sept. 1946, pp. 21-48 (V. 43)

The effects of composition and fineness of the laboratory cements employed in cement investigations for Boulder Dam upon strength, volume changes, and sulfate resistance of mortars, are reported for ages up to 10 years. For both wet and dry storage conditions, factors for each of several ages are given which indicate the contribution of each of the four major compounds present in portland cement to tensile and compressive strengths and volume changes.

★ANALYSIS AND DESIGN OF ELEMENTARY PRESTRESSED CONCRETE MEMBERS..... 43-4

HERMAN SCHORER—Sept. 1946, pp. 49-88 (Vol. 43)

The purpose of this paper is to outline the analysis and design of elementary prestressed concrete members, such as beams, columns, ties, etc., subjected to internal and external axial forces and bending moments. The internal stresses, caused by the action of the prestress forces, are combined with the stresses due to external loads in three typical loading stages. The first stage considers the stress condition resulting from the simultaneous application of all sustained loads. The second stage determines the stress changes due to normal live loads, based on a truly monolithic participation of the entire concrete area. The third stage assumes a cracked tension zone, which condition introduces the derivation of ultimate stresses and clarifies the influence of the prestress action on the type of failure. The analytical expressions are simplified by means of convenient ratios, which essentially define the sectional shape, the effective steel prestress, and the concrete fiber stresses. Numerical examples serve to illustrate the various steps.

★STUDIES OF THE PHYSICAL PROPERTIES OF HARDENED PORTLAND CEMENT PASTE

(Part 1) **Price 50 cents**.....43-5a
(Part 2 and appendix) **Price 75 cents**...43-5b
(Part 3) **Price 50 cents**.....43-5c

T. C. POWERS and T. L. BROWNYARD—Oct. 1946, pp. 101-132, Nov. 1946, pp. 249-336, Dec. 1946, pp. 469-504 (V. 43)

IN NINE PARTS

Part 1. A Review of Methods That Have Been Used for Studying the Physical Properties of Hardened Portland Cement Paste

Part 2. Studies of Water Fixation
Appendix to Part 2

Part 3. Theoretical Interpretation of Adsorption Data

Part 4. The Thermodynamics of Adsorption
Appendix to Parts 3 and 4

Part 5. Studies of the Hardened Paste by Means of Specific-Volume Measurements

Part 6. Relation of Physical Characteristics of the Paste to Compressive Strength

Part 7. Permeability and Absorptivity

Part 8. The Freezing of Water in Hardened Portland Cement Paste

Part 9. General Summary of Findings on the Properties of Hardened Portland Cement Paste

This paper deals mainly with data on water fixation in hardened portland cement paste, the properties of evaporable water, the density of the solid substance, and the porosity of the paste as a whole. The studies of the evaporable water include water-vapor-adsorption characteristics and the thermodynamics of adsorption. The discussions include the following topics:

1. Theoretical interpretation of adsorption data
2. The specific surface of hardened portland cement paste
3. Minimum porosity of hardened paste
4. Relative amounts of gel-water and capillary water
5. The thermodynamics of adsorption
6. The energy of binding of water in hardened paste
7. Swelling pressure
8. Mechanism of shrinking and swelling
9. Capillary-flow and moisture diffusion
10. Estimation of absolute volume of solid phase in hardened paste
11. Specific volumes of evaporable and non-evaporable water
12. Computation of volume of solid phase in hardened paste
13. Limit of hydration of portland cement
14. Relation of physical characteristics of paste to compressive strength
15. Permeability and absorptivity
16. Freezing of water in hardened portland cement paste

★MINIMUM STANDARD REQUIREMENTS FOR PRECAST CONCRETE FLOOR UNITS 43-6

REPORTED BY ACI COMMITTEE 711—Oct. 1946, pp. 133-148 (V. 43) In special covers

Supersedes 40-17, 42-11.

These minimum standard requirements are to be used as supplements to the ACI "Building Regulations for Reinforced Concrete" (ACI 318-41). With respect to design for strength, i. e., for bending moment, bond and shear stresses, all types shall be designed in accord with standard reinforced design theory and ACI 318-41. With respect to cover, there is in some cases departure therefrom justified by the greater refinement in the finished product when made by factory methods with factory control. Precast floor systems with I-beam type and hollow core type joists are covered. Appendix contains applicable sections of the ACI code (ACI 318-41). This report, originally published in Feb. 1944 Journal, has been revised by the committee and adopted by the Institute as an ACI Standard, Aug. 1946. The committee consists of F. N. Menefee, Chairman, Warren A. Coolidge, R. E. Copeland, Clifford G. Dunnells, H. B. Hemb, Harve Kilmer, Glenn Murphy, Gayle B. Price, John Strandberg, J. W. Warren, Roy R. Zippradt.

★RECOMMENDED PRACTICE FOR THE CONSTRUCTION OF CONCRETE FARM SILOS 43-7

REPORTED BY ACI COMMITTEE 714—Oct. 1946, pp. 149-164 (V. 43) In special covers

Supersedes 40-10, 42-12.

These recommendations describe practice for use in the design and construction of concrete silos—stave, block and monolithic, for the storage of grass or corn silage. The report is the work of the committee consisting of William W. Gurney, Chairman, J. W. Bartlett, Walter Brassert, Claude Douthett, Harry B. Emerson, William G. Kaiser, R. A. Lawrence, G. L. Lindsay, J. W. McCalmont, Dalton G. Miller, C. C. Mitchell, K. W. Paxton, B. M. Radcliffe, Charles F. Rogers, Stanley Witzel. It was adopted by the Institute as an ACI Standard Aug. 1946.

THE DURABILITY OF CONCRETE IN SERVICE 43-8

F. H. JACKSON—Oct. 1946, pp. 165-180 (V. 43)

This paper discusses the problem of concrete durability with reference primarily to highway bridge structures located in regions subject to severe frost action. Four major types of deterioration are defined and illustrated and several specific matters which have bearing on the problem, including the effect of construction variables, modern vs. old fashioned cements, air entrainment and the so-called "cement-alkali" aggregate reaction, are discussed. The report concludes with a series of recommendations indicating certain corrective measures which should be taken.

WEAR RESISTANCE TESTS ON CONCRETE FLOORS AND METHODS OF DUST PREVENTION 43-9

GEORG WASTLUND and ANDERS ERIKSSON—Oct. 1946, pp. 181-200 (V. 43)

This paper presents a description of tests made on concrete floor specimens of various types in order to determine their resistance to wear and to investigate the character of deterioration of concrete floor surfaces due to traffic. The results of these tests show that concrete floors provided with finish courses containing coarse aggregate up to about ⅛ inch in size and an excess of pea gravel are definitely superior to concrete floors with a finish course containing fine sand only which are common in Sweden at the present time. Moreover, this investigation has helped to elucidate the causes of the often very severe and detrimental dusting of concrete floors. The surface skin of concrete floors is of poor quality and is easily abraded. Dusting can be considerably reduced if the poor surface skin is removed by machine grinding provided that the concrete below the surface skin is of first-rate quality. The paper concludes by proposing a detailed tentative specification for concrete floor finish which differs in essentials from current Swedish practice.

★LINING OF THE ALVA B. ADAMS TUNNEL.....43-10

RICHARD J. WILLSON—Nov. 1946, pp. 209-240 (V. 43)

The 13.03 mile Alva B. Adams Tunnel, excavated under the Continental Divide, as a part of the transmountain water diversion plan of the Colorado-Big Thompson Project, United States Department of the Interior, Bureau of Reclamation, is now lined with concrete. Lining equipment and methods and aggregate processing are described.

REPAIRS TO SPRUCE STREET BRIDGE, SCRANTON, PENNA.....43-11

A BURTON COHEN—Nov. 1946, pp. 241-248 (V. 43)

Repairs and reinforcements of the Spruce Street Bridge built in 1893 over the Lackawanna Railroad and Roaring Brook in Scranton, Pa. are described. The effective application of the "Alpha System-Composite Floor Design" reinforced the floor system at the same time a new concrete floor slab was laid. Concrete prices are included and eleven illustrations supplement the text of the paper.

THE STRUCTURAL EFFECTIVENESS OF PROTECTIVE SHELLS ON REINFORCED CONCRETE COLUMNS....43-12

F. E. RICHART—Dec. 1946, pp. 353-364 (V. 43)

This paper presents a study of 108 plain, tied or spirally reinforced concrete columns. The columns were 7, 8 and 9 in. round or square, 45 in. ong, and the ties and spirals were 6 in. in diameter.

The columns were loaded axially, with "flat" ends. Strains were measured and close observations were made of the initial failure of the protective shell.

Analyses of the test results were made to see if the column shells were fully effective. This was the case with the shells of spirally reinforced columns, but the tied columns showed a slight deficiency in the strength expected on the basis of previous tests of the 1930 ACI column investigation.

The test results lend support to the design methods prescribed in the current ACI Building Regulations for Reinforced Concrete.

PRECAST CONCRETE STRUCTURES..43-13

A. AMIRIKIAN—Dec. 1946, pp. 365-380 (V. 43)

Precasting is becoming a major factor in the choice of reinforced concrete as a construction material because of ever-rising cost of labor and materials. The advantages of precasting are not however confined to savings in cost and materials. Since it is a planned method of construction, comparable to factory production, its use also assures a better control of quality and speedier completion of the project. This article is an attempt to show how precasting can be utilized to provide the framing of a great variety of structures. The first part deals with bent type of framing as used in buildings, the second describes a novel type of framing consisting of precast cells, particularly suitable for floating structures.

COMPARATIVE BOND EFFICIENCY OF DEFORMED CONCRETE REINFORCING BARS.....43-14

ARTHUR P. CLARK—Dec. 1946, pp. 381-400 (V. 43)

The purpose of the tests described was to determine the resistance to slip in concrete of 17 different designs of deformed reinforcing bars.

The tests were of the pull-out type in which the bars were cast in a horizontal position; the depth of concrete under the bars and the length of embedment were varied. The slip of the bar was measured at the loaded and free end. Three tests were made of each variable for each design of deformation.

It was established that a certain group of the bars was definitely superior to the others, in the sense that their average rating was significantly higher than the average of the others. Bars cast in the top position were much less effective than those cast in the bottom position.

★PROPOSED REVISION OF BUILDING REGULATIONS FOR REINFORCED CONCRETE (ACI 318-41)..43-15

REPORTED BY ACI COMMITTEE 318—Dec. 1946, pp. 401-468 (V. 43)

The report with its proposed changes has been released by the Standard Committee for convention action. The contents are fully explained in the title. The current "code" appears in full in larger type; the proposed changes in smaller type. Published for information and study prior to convention consideration.

Honor Roll

February 1 to Oct. 31, 1946

The Honor Roll for the period February 1, to Oct. 31, 1946 finds T. E. Stanton still in the lead with 24½ members and J. L. Savage second with 21.

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December JOURNAL in two parts

The annual "JOURNAL Supplement", as it has been known for many years, is issued this month as Part 2 of the December JOURNAL. This is to comply with a ruling of the Third Assistant Postmaster General with regard to mailing in accordance with postal laws and regulations.

As the Supplement did in the past, Part 2 does now--completes the *Proceedings* volume year with its contents of title page, table of contents, closing discussion and indexes for Vol. 42, enabling the member or subscriber who so desires to assemble his own bound volume.

Sources of Equipment, Materials, and Services

A reference list of advertisers who participated in the Fifth Annual Technical Progress Issue of the ACI JOURNAL—the pages indicated will be found in the February 1946 issue and (when it is completed) in V. 42, ACI Proceedings. **Watch for the 6th Annual Technical Progress Section in the February 1947 JOURNAL.**

Concrete Products Plant Equipment

page

Construction Equipment and Accessories

Contractors, Engineers and Special Services

Materials

Testing Equipment

ACI publications in large current demand

Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures (1946)

Reported by ACI Committee 315, Detailing Reinforced Concrete Structures, Arthur J. Boase, Chairman, this book reached the top of the ACI "best seller" list within one month of its distribution to all ACI members in good standing in July 1946. It is a large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$2.50; to ACI Members—$1.50.

ACI Standards—1946

180 pages, 6x9 reprinting ACI current standards: Building Regulations for Reinforced Concrete (ACI 318-41); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46); four 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); Construction of Concrete Farm Silos (ACI 714-46); 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.

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

AMERICAN CONCRETE INSTITUTE New Center Building Detroit 2, Michigan

Recent ACI Standards

Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)

16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)

16 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)

24 pages in covers: 50 cents per copy (40 cents to ACI Members)

Specifications for Cast Stone (ACI 704-44)

4 pages: 25 cents per copy

Specifications for Concrete Pavements and Bases (ACI 617-44)

30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)

30 pages in covers: 50 cents per copy (40 cents to ACI Members)

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)

4 pages: 25 cents per copy

Building Regulations for Reinforced Concrete (ACI 318-41)

63 pages in covers: 50 cents per copy. (40 cents to ACI Members)

Proposed Standards

Proposed Manual of Standard Practice for Detailing Reinforced Concrete Structures

Reported by ACI Committee 315. It is a separate publication of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. 55 pages; $2.50 per copy. $1.50 to ACI Members. (Distributed to ACI Members in July 1946)

The Nature of Portland Cement Paints and Proposed Recommended Practice for Their Application to Concrete Surfaces

Reported by Committee 616 as information and for discussion only. 20 pages, 25 cents per copy (Reprint from ACI JOURNAL, June 1942)

Proposed Recommended Stresses for Unreinforced Concrete

Reported by Committee 322 as information and for discussion only. 4 pages, 25 cents per copy. (Reprint from ACI JOURNAL, Nov. 1942)

The AMERICAN CONCRETE INSTITUTE

is a non-profit, non-partisan organization of engineers, scientists, builders, manufacturers and representatives of industries associated in their technical interest with the field of concrete. The Institute is dedicated to the public service. Its primary objective is to assist its members and the engineering profession generally, by gathering and disseminating information about the properties and applications of concrete and reinforced concrete and their constituent materials.

For four decades that primary objective has been achieved by the combined membership effort. Individually and through committees, and with the cooperation of many public and private agencies, members have correlated the results of research, from both field and laboratory, and of practices in design, construction and manufacture.

The work of the Institute has become available to the engineering profession in annual volumes of ACI Proceedings since 1905. Beginning 1929 the Proceedings have first appeared periodically in the Journal of the American Concrete Institute and in many separate publications.

Pamphlets presenting brief synopses of Journal papers and reports of recent years, most of them available at nominal prices in separate prints, and information about ACI membership and special publications in considerable demand are available for the asking.

New Center Building, Detroit 2, Michigan

Many papers and discussions are submitted for consideration of the Publications Committee in a single copy of the manuscript. Three copies are required.

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.