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TESTS OF EIGHTEEN CONCRETE COLUMNS REINFORCED WITH CAST IRON

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Bureau of Standards

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I. INTRODUCTION

This paper is a description and discussion of the tests of 18 reinforced concrete columns, tested to determine the suitability of cast iron as a reinforcing material.[1] The originator of this method

[1] The columns were made by L. J. Mensch, contracting engineer, of Chicago, at the yard of the American Building Foundry Co., in that city. They were submitted by him to the Bureau of Standards for the purpose of testing. Approximately one month after being made they were shipped in a gondola car to the Pittsburgh Laboratory of the Bureau, where they were tested.

of column construction is Dr. F. von Emperger, of Austria, and columns of this type have been in use abroad for many years. Small columns of similar construction have been tested both in Germany and Austria.[2]

The number of columns tested was rather small when the number of variables introduced in their construction is considered, and therefore the conclusions drawn as to the relative merits of the various types employed in the test are not so precise and reliable as ultimately to be desired. From a much larger group of columns, with a variation of both length and diameter for each type of construction, and variation of percentage both of cast iron, steel, and spiral reinforcement, a more complete insight into the behavior of the component parts would have been afforded and rational formulas for designs readily obtained. It is to be hoped that additional tests will give this information and extend our knowledge of the behavior of cast-iron reinforced columns. From this set of columns, however, one important formula was obtained, that for the variation of strength with length.

The columns were not built entirely according to specified dimensions, the cast-iron core being off center in several cases, the spiral irregularly pitched, and at times not concentric with the axis of the column. In some cases the vertical rods were not parallel with the axis and frequently did not touch the spiral but were deeper in the column, as much as an inch in some instances. However, these same variations occur in practice, and from close observation of the results obtained in the present investigation it may safely be concluded that none of the above-noted variations had sufficient effect to produce abnormal results. Thus possibly more valuable information was obtained from this test than had the columns been built with greater care. The actual structure which is to bear the loads is built under difficulties which cause variations, and these variations must be fully taken into account in the design. The smaller the effect of any probable variations the better for safety and economy in design. This fact is of no little importance, yet is frequently overlooked.

II. DESCRIPTION OF THE COLUMNS

1. STRUCTURE

In Fig. 1 and Table 1 are given the details of construction of the columns. All columns (except Nos. 9 and 10, which were plain cast-iron tubes) were 12 inches in diameter, with 1⅛-inch pitch

[2] See Beton und Eisen, 1912, p. 118, for a description of these tests.

spiral reinforcements of No. 8 gage wire. The concrete core (that is, within the wire spiral) was 12 inches in diameter, and the protective shell was nominally ⅜ of an inch thick, but, as previously stated, this depth or thickness varied greatly, due to irregularity and eccentricity in the spiral. Some variation in thickness was also due to the fact that the outer surface of the column was composed of flat faces, since the forms used in casting the columns consisted of a number of 2¾-inch vertical boards placed edge to edge.

Columns 1 and 2, identical in structure, were representative specimens of the "Emperger" type of reinforcement. They were 6 feet long, reinforced with a cast-iron tube, steel rods, and spiral, as noted above. The cast-iron tube was ¾ of an inch thick, 6 inches inside diameter. There were four ⅜-inch square steel rods and two ¼-inch round steel rods as longitudinal reinforcement, all equally spaced around the inner circumference of the wire spiral.

Column 3 was 8 feet long; in all other respects the same as 1 and 2.

The space inside the cast iron of column 4 was filled with concrete. All dimensions of this column were the same as for column 3.

Column 5 contained a 5-inch outside diameter cast-iron tube with walls 1 inch thick. The length, spiral, and rods were identical with those of column 3.

Column 6 had a 7-inch cast-iron tube with walls ⅝ inch thick, and in other details was the same as column 3.

Column 4 was made in order to determine the effect of filling the cast iron with concrete, and columns 5 and 6 to determine the effect of the variation of the radius of gyration of the cast iron.

Columns 7 and 8 were reinforced concrete columns of the ordinary type, common in contemporary building construction. They were 10 feet long, having a No. 8 wire spiral, two ¼-inch circular spacing rods, and four ⅜-inch square rods as reinforcement.

Columns 9 and 10 were representative specimens of the cast-iron tubes as used for reinforcement in columns 1 and 2, etc. They were 6 inches outside diameter, 4½ inches inside diameter, and 10 feet in length as received at the Bureau of Standards.

Two-foot sections were cut from these columns as received and numbered 9–a and 10–a. One unnumbered section of the same dimensions as 9–a and 10–a was received and tested.

Columns 11 and 12 were identical in cross section with columns 1 and 2, but were 10 feet long.

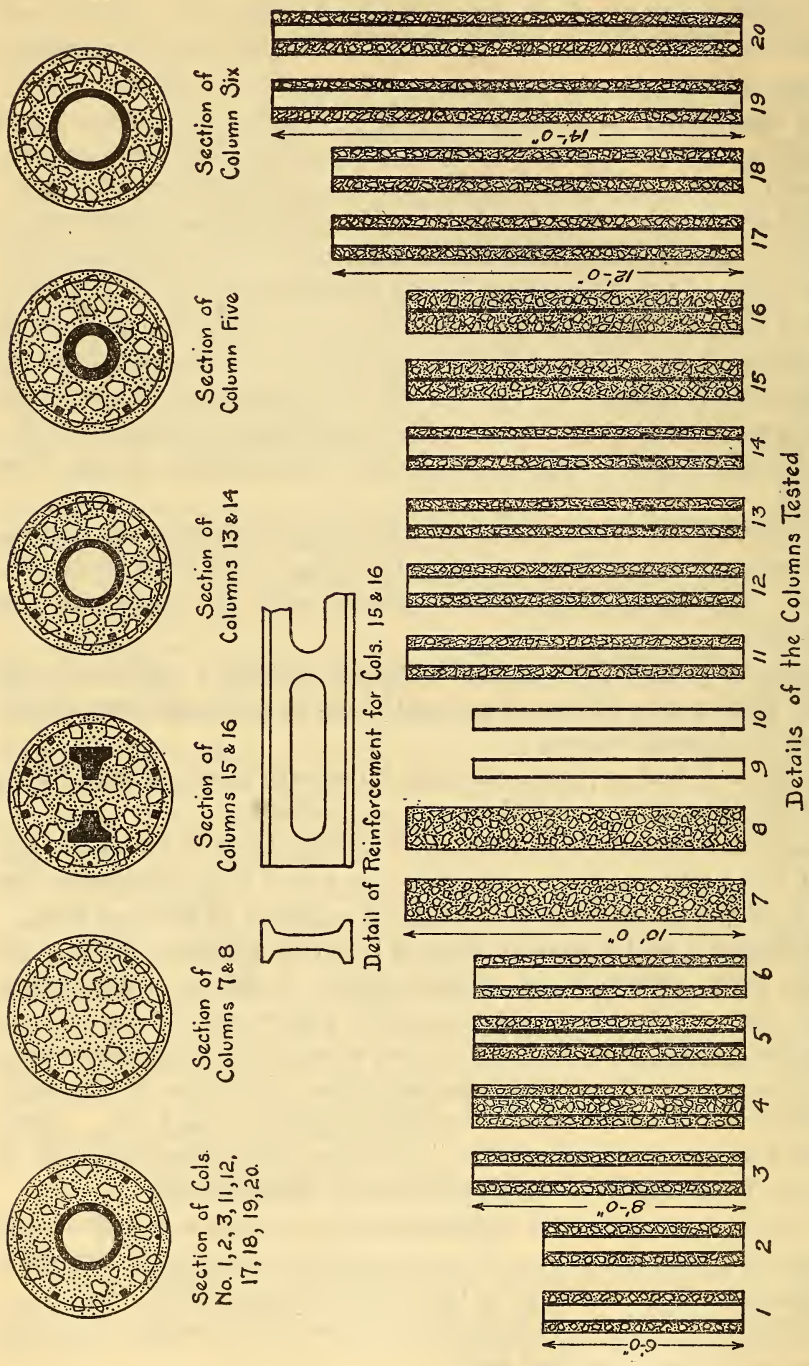

FIG. 1.—*Cross sections of columns tested, showing types and arrangement of reinforcing materials*

TABLE 1.—Summary of Data on Columns Tested

Column No.	Length	Reinforcement (cast iron)[a]		Cross-sectional areas				Per cent areas of net column area within spiral				Load carried by the column		Stress at maximum load	Per cent of maximum load at which the first signs of failure show	Mean for like columns	Strength of concrete as shown in the 6 by 12 inch cylinders
		Outer diameter	Thickness	Steel spiral (equivalent)	Longitudinal steel rods	Cast iron	Concrete	Spiral	Rods	Cast iron	Concrete	At first signs of failure	Maximum				
	Feet	Inches	Inch	Inch²	Inches²	Inches²	Inches²					Pounds	Pounds	Lbs./in.²		Per cent	Lbs./in.²
1	6	6	¾	0.69	0.66	12.33	84.2	0.71	0.68	12.7	86.6	436 500	1 057 000	10 880	41.5	58.5	4380
2	6	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	776 000	1 026 000	10 560	75.5		4380
3	8	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	436 500	940 000	9680	46.3	.....	3720
4	8	6	¾	.69	.66	12.33	100.1	.61	.58	10.9	88.5	570 000	1 031 500	9110	55.2	.....	3720
5	8	5	1	.69	.66	12.57	92.2	.65	.62	11.9	87.0	630 500	925 000	8730	68.1	.....	4150
6	8	7	⅝	.69	.66	12.51	73.9	.79	.76	14.4	84.9	565 500	920 000	10 560	61.3	.....	4150
7	10	None	None	.69	.66	None	113.1	.61	.58	.....	99.4	490 000	518 500	4580	94.7	85.9	3890
8	10	None	None	.69	.66	None	113.1	.61	.58	.....	99.4	440 000	562 200	4970	77.2		3390
11	10	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	643 500	911 000	9380	70.5	68.5	3750
12	10	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	625 000	940 000	9680	66.5		3750
13	10	6	¾	.69	1.22	12.33	83.7	.71	1.26	12.7	86.2	776 000	896 000	9230	86.7	77.5	4300
14	10	6	¾	.69	1.22	12.33	83.7	.71	1.26	12.7	86.2	727 500	1 066 000	10 980	68.2		4300
15	10	(b)	(b)	.69	1.22	13.4	98.5	.61	1.08	11.8	87.0	850 000	1 071 800	9480	79.2	68.5	4300
16	10	(b)	(b)	.69	1.22	13.4	98.5	.61	1.08	11.8	87.0	576 000	996 000	8800	47.8		4300
17	12	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	732 500	951 500	9800	77.0	73.2	4250
18	12	6	¾	.66	.66	12.33	84.2	.71	.68	12.7	86.6	630 500	909 000	9360	69.3		3900
19	14	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	630 500	888 500	9150	70.2	76.1	4250
20	14	6	¾	.69	.66	12.33	84.2	.71	.68	12.7	86.6	679 000	837 500	8520	82.0		4400
9	8	6	¾	.....	.....	12.33	.....	100	.....	.....	.....	480 000	39 000	.....	.....	.....	.....
10	8	6	¾	.....	.....	12.33	.....	100	.....	.....	.....	540 000	43 800	.....	.....	.....	.....

[a] Spiral reinforcement consisted of No. 8 gage wire, 1⅛-inch pitch, around core 12 inches in diameter, except columns 9 and 10, which were plain cast-iron tubes. The longitudinal reinforcement for columns 1 to 12 and 17 to 20 consisted of four ⅜-inch square steel rods and two ¼-inch round steel rods. For columns 13 to 16 eight ⅜-inch square steel rods and two ¼-inch round steel rods were used.

[b] Special I section.

Columns 13 and 14 had the same cross section as columns 1 and 2, with the unfortunate exception of having eight instead of four ⅜-inch square longitudinal reinforcing rods. The cast iron in each of these columns was made in two pieces 3 feet and 7 feet long, respectively. These two pieces were fitted together by a kind of a bell and spigot joint (see Table 3), both ends of the joint being machined and the joint set with neat cement before the section as a whole was used in reinforcing the column. Both ends of the section were enlarged, as shown by the details in Table 3, and in the photograph, Fig. 12. These columns, 13 and 14, were made with the object of determining the effect of such a splice or joint upon the strength and behavior of the columns under load.

Columns 15 and 16, for cast-iron reinforcement, had a casting with an **I** cross section and large longitudinal holes in the web (see Table 3) instead of the tubes used as reinforcement in the other columns. The steel spiral and longitudinal rod reinforcement was the same as that used in columns 13 and 14, namely eight ¾-inch square rods, two ¼-inch circular rods as longitudinal, and No. 8 wire as spiral reinforcement. The object of the **I** cross section is to permit other structural members to pass nearer the center of the column in the direction of the long axis of the **I**, and also to give greater strength in the other direction—a useful property in columns on the face of a building.

Columns 17 and 18 were 12 feet long; columns 19 and 20 were 14 feet long; all four of the same cross section as columns 1 and 2. The ⅜-inch square steel rods in these four columns, 17, 18, 19, and 20, were twisted. In all other columns these rods were plain, with a slight transverse corrugation, which can be clearly seen in Fig. 11.

2. MATERIALS

(*a*) *Cement, Sand, and Gravel.*—The concrete used in the columns was proportioned by volume and was composed of one part cement, one part sand, and two parts gravel. The materials used were: Universal Portland cement received from the warehouse of the Wisconsin Lime & Cement Co., and "good clean-washed torpedo sand and gravel, the standard as used in the city of Chicago," the gravel to pass 1 inch but not a ⅜ inch ring.

(*b*) *Reinforcing Steel.*—Standard high-carbon No. 8 gage wire in the form of a spiral 12 inches in diameter and diametrically secured by ¼-inch round rods and ⅜-inch square plain and twisted rods

for longitudinal reinforcement were furnished by the American System of Reinforcing Co. Coupons were cut from these rods for test, but were, unfortunately, lost in transit.

(*c*) *Cast Iron.*—The cast-iron reinforcements or columns were made and furnished by the American Building Foundry Co., of Chicago. They were cast horizontally from a mixture of 50 per cent gray pig iron and 50 per cent scrap. The tubes used in the columns were machined at both ends, the other surfaces of the columns receiving no treatment except a sand blast.

As was to be expected in commercial castings made in this manner, there were large variations (about ¼ inch) in thickness, although the cross-sectional area remained fairly constant. No determination of the actual cross-sectional area of the cast iron was made, for the large amount of time and labor necessary to saw the reinforcement into a sufficient number of sections would not have been commensurate with the information gained. After the columns had been tested the cast iron was so badly injured by the compression that either no cross-sectional area could be obtained or, if obtained, would not represent the original area.

One-inch cast-iron cubes from the same pour as the columns were tested by Mr. Duff A. Abrams at the Lewis Institute of Chicago, and showed strengths of from 100 000 to 150 000 pounds, much in excess of the maximum stress developed by the actual cast-iron reinforcement used in the columns.

3. FABRICATION

(*a*) *Forms.*—All of the test columns were made and stored in the open air. The foundation for the form was made by setting a 2-foot square iron plate firmly and level into the natural soil. Upon the iron plate an 18-inch square base made of 1⅝-inch dressed lumber nailed and cleated together was placed and carefully leveled. Upon this wooden base circles for the location of the iron columns and forms were drawn. The base of the cast-iron reinforcement was kept in the desired position by means of nails driven in the wooden base. The cylindrical forms were made of 2¾ by ⅞ inch dressed boards running longitudinally, thus giving to the finished column a series of flat faces instead of the circular cross section, as shown in the drawings. The reinforcing rods were from ¼ inch to ⅜ inch shorter than the columns.

(*b*) *Concrete.*—All columns were made with a 1 : 1 : 2 mix. The concrete was hand mixed by two experienced laborers in a mortar

box 4 by 6 feet by 8 inches deep. Each batch consisted of 3 bags of cement, 3 cubic feet of sand, and 6 cubic feet of gravel.

(*c*) *Mixing.*—The method of mixing may be divided into the following operations:

1. Three cubic feet of sand was placed in the mixing box and leveled and three bags of cement evenly spread over the sand. The whole was turned over twice and again leveled.

2. Six cubic feet of gravel was next evenly spread over the sand and cement mixture. Then the whole was turned over twice and again leveled.

3. Approximately six pails of water were slowly poured over the surface and the concrete turned over twice. The consistency was termed "mushy" by Mr. Mensch.

(*d*) *Filling the Forms.*—The forms were slowly filled from pails of the concrete and tamped thoroughly with a 1 by 2 inch rod. Observation of the surface of the column and the interior concrete after breaking showed the material to have been well mixed and well tamped.

The air temperature during making varied from 36° to 76° F, with a mean of about 60°, and for the period of one month after making did not go below 40°.

One day after making, the tops of the columns were finished with a 1 : 1 mixture of cement and sand, a glass plate being placed on top to secure a level surface for bearing. This treatment, however, was not entirely successful, for in several cases the columns were received at the testing laboratory without this cap, it having loosened in transit. The columns without caps were set on end in the laboratory, a form of heavy paper secured with wire around the column, and a 1 : 2 mixture of Universal Portland cement and Standard Ottawa sand troweled to a smooth surface flush with the end of the cast-iron reinforcement. These fresh caps were sprinkled with water twice a day until the columns were tested. None of the caps failed under the test.

(*e*) *Test Cylinders.*—One 6 by 12 inch test cylinder was made from each batch of concrete and numbered the same as the columns made from that batch.

III. PREPARATION FOR TESTING

1. OBSERVATIONS

Data from the test of the columns were obtained from (1) a general log kept during the test, in which the behavior of the column was noted; (2) observations upon the total longitudinal com-

pression of the columns at the various increments in load; (3) observations upon the longitudinal compression of the steel reinforcing rods and adjacent concrete; and (4) observation of the tilting of the base of the testing machine.

The total compression of the columns was obtained directly from Ames gages, these gages (three in number, equally spaced around the column) indicating the change in distance between two yokes attached near each end of the column. The compressometer consisted of two wrought-iron ⅛ by 1 inch straps bent to a radius 1 inch larger than that of the columns and

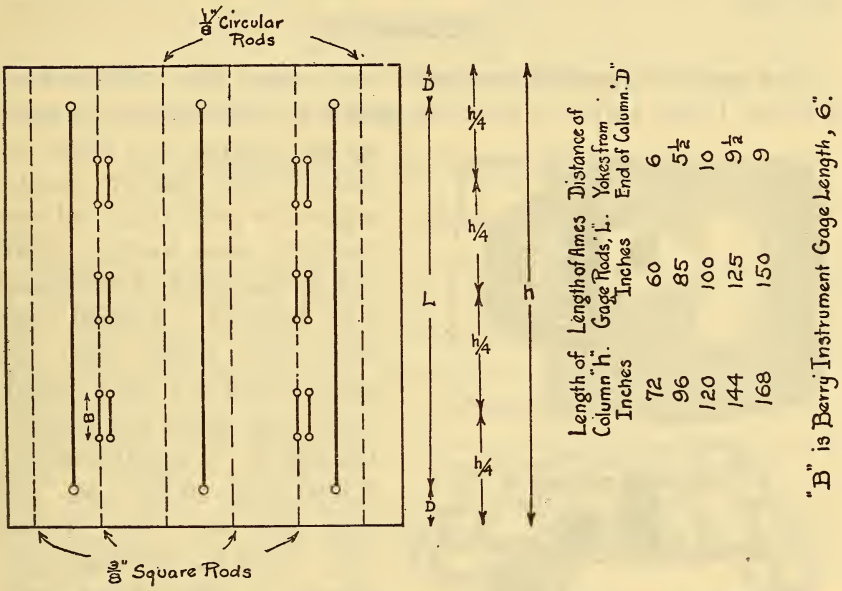

FIG. 2.—*Showing developed surface of column with location of long gage compressometer and Berry strain gage stations*

bolted together around the column. Through these yokes at points, 120° apart, holes were tapped for ½-inch bolts with lock nuts, these bolts being used to fix the yokes rigidly to the column. The yokes were placed on the column at a predetermined distance apart, each approximately 9 inches from the ends of the columns. Brackets attached to the lower yoke at the bolts held the three Ames gages. Three brackets on the upper yoke held the spherical bearings, from which were hung hollow extensible brass rods ending in conical steel tips. The rods were adjusted to the proper length, the tips set into depressions in the spindles of the Ames gages, and the spherical bearing clamped. The compression of the column between the two yokes was then read directly on the

Ames gages. Fig. 6 shows column 19 with compressometer attached, ready for testing.

The deformation observations on the steel rods were made with a portable 6-inch Berry strain gage. These readings were taken at quarter points of the column length, on stations which were diametrically opposite, making a total of six groups of observations for each column. At each of these points readings were made of the compression of the longitudinal steel rod and on the concrete immediately adjacent to the rod. (See Fig. 2 for a diagrammatic location of the gage lengths upon the column surface.)

2. PREPARATION

The points at which readings were made were prepared as follows: Using care, the concrete shell was first chipped off with an air hammer and chisel at one of the quarter points, where the reinforcing rod apparently was located, until one of the wires of the spiral was exposed. Lateral cuts were then made until the longitudinal rod was exposed, care being taken not to injure the spiral. A small hole was drilled close to the rod and into this a ⅜-inch round plug, ¾ of an inch deep, was placed, an effort being made to drill the hole so that the plug could be hammered into it and firmly fixed in this manner. Plaster of Paris was poured around the plugs as an additional measure to secure firmness and prevent motion. Holes 0.046 inch in diameter were drilled in the rods and plugs. The drilled holes for each reading of the Berry gage were 6 inches apart. Fig. 3 gives a diagrammatic representation of the prepared rod and plug.

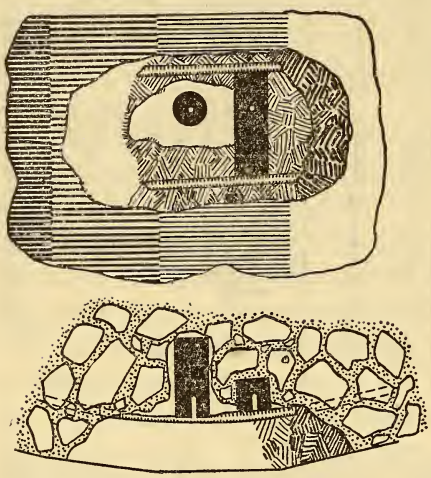

Fig. 3.—*Details of setting steel plug and locating drill holes for Berry strain gage readings on concrete and longitudinal rods*

A sensitive machinist's level was placed upon the lower platen of the 10 000 000-pound testing machine and readings of the position of the bubble were made to determine whether it tilted at any time during the test.

3. PLACING COLUMNS IN TESTING MACHINE

A bed of fresh mixed plaster of Paris being placed upon the center of the lower platen of the testing machine, the column was lowered into it, the plaster being of sufficient stiffness so that any unevenness in the base of the column was filled. After the plaster had set, the column was plumbed by tilting the lower platen of the machine. This platen is mounted on a spherical bearing, and is capable of a universal motion. A cap of plaster of Paris being placed on top of the columns, the crosshead or upper platen of the testing machine was lowered, squeezing out the superfluous plaster. When the head was approximately an eighth of an inch from the top of the column it was stopped and the plaster permitted to harden. By thus capping both ends of the column in the machine, a very good bearing was obtained.

IV. DESCRIPTION OF TESTS, GENERAL

1. SMALL CYLINDERS

The 6 by 12-inch concrete cylinders were tested to failure in a 200 000-pound Olsen testing machine. Both ends of the cylinders were capped with plaster of Paris, and a spherical bearing block was used on the lower platen of the machine in order to insure a good bearing. These cylinders were tested as near at possible on the days on which the same numbered columns were tested. Compression readings were made with an electrical contact micrometer compressometer on an 8-inch gage length upon all cylinders. The micrometer used in this compression work read directly to 0.0001 inch.

The cylinders were well made, but the small number does not warrant the drawing of any very definite conclusions as to the relative merit of the concrete in the various columns. The stress-strain diagrams for all cylinders is given in Fig. 4.

2. CAST IRON

Tests were made of five cylindrical cast-iron specimens 6 inches outside diameter, 4½ inches inside diameter (the same section used in the cast-iron reinforcement in 10 of the 18 columns), and of the following lengths:

(*a*) One special test specimen 2 feet long.

(*b*) Two 2-foot sections, the same dimensions as (*a*), cut from the ends of columns 9 and 10 at the Bureau of Standards, numbered 9–a and 10–a.

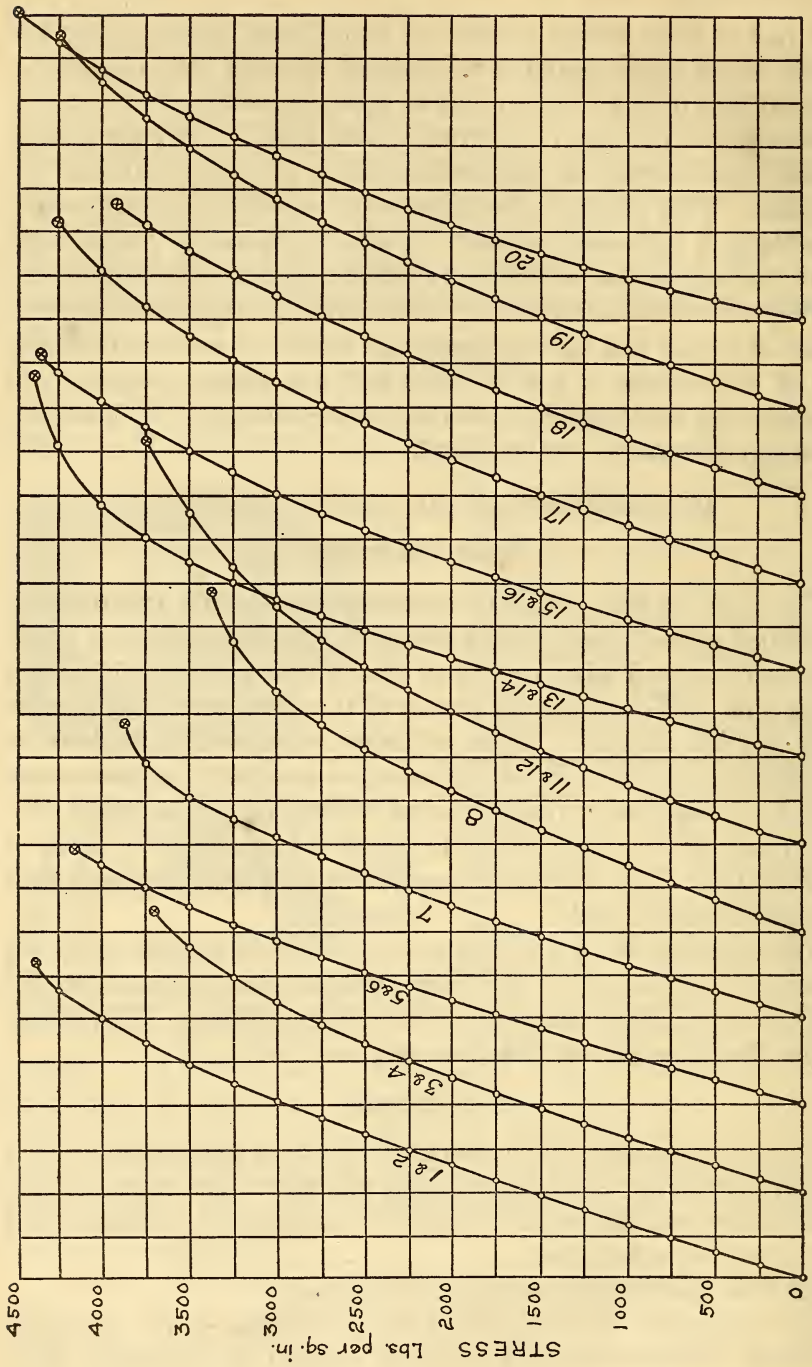

FIG. 4.—*Stress strain diagram for concrete cylinders. Numbers refer to columns of same batch*

(*c*) The remaining 8-foot sections of columns 9 and 10 after sections 9–a and 10–a had been cut from these columns as received at the Bureau.

Both ends of all specimens were very accurately and finely machined. Cast-iron plates 1 inch thick were used as bearings during the test of these specimens. Stress was applied and strain readings taken on 8-inch gage length at increments of 5000 pounds per square inch, until the large strains indicated danger from failure. At this point readings were discontinued and the load continuously increased until failure occurred. Fig. 5 gives the

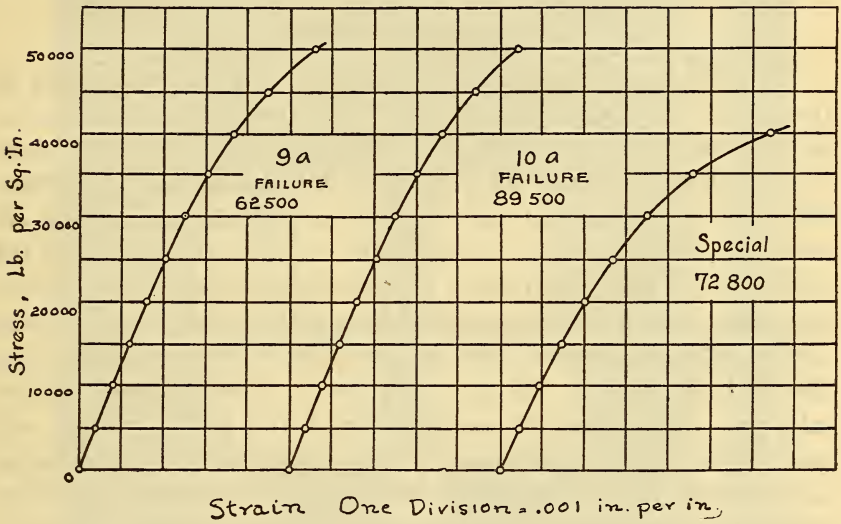

FIG. 5.—*Stress strain diagram for 2-foot cast-iron specimens*

stress-strain curves for the average strain of three gage stations of each specimen.

The failure in all five specimens was violent, care being taken to protect all parts of the testing machine from injury by flying fragments. Some fragments weighing several pounds were thrown as much as 40 feet. Fig. 13 shows the fragments of the specimens after testing, the maximum stress being marked on each specimen. No compression readings were made on the 8-foot columns 9 and 10. The average maximum stress withstood by the 8-foot specimens was only 62 per cent of the average of the three 24-inch sections. This is a much greater loss in strength than that alone due to the length of the columns, the formula $P = 10\ 000 - 60\frac{L}{R}$ giving a reduction of 24 per cent in

strength.[3] Hence the only reasonable conclusion as to the large reduction in strength is that it is caused by imperfections in the structure and materials in the columns. As could be plainly observed, column 10 failed at a chaplet, and (after breaking) the metal around the stem of this chaplet showed a "cold shut."

The cast-iron bearing plates used to protect the base and head of the testing machines from being marked had permanent impressions left in them by the force on the ends of the cast-iron columns. The evenness in depth of these impressions clearly showed that the stress was uniformly distributed.

3. COLUMNS PROPER

The reinforced columns were all tested to a maximum load at the age of 61 to 62 days in the 10 000 000-pound Olsen compression machine. (See Fig. 6.) The method of placing the reinforced columns in the testing machine has already been described. The load was applied in increments so that the stress applied (in pounds per square inch of net column cross section inside of the spiral) was in even multiples of 500 pounds per square inch. Readings were made as rapidly as consistent with accuracy to eliminate any possible small error due to flow in the concrete, the points always being read in the same sequence. The time required to take one complete set of readings varied, but on the 14-foot columns (the longest and therefore the most difficult to read owing to the observer having to climb about) a complete set of readings could be taken and a new load applied in less than five minutes. When spalling of the concrete outside the spiral spread to such an extent that any further readings of the columns would have been unreliable, they were discontinued, and the instruments removed to prevent their being ruined by the falling concrete.

(*a*) *Stress Evenly Distributed.*—To determine the angular movement of the base of the testing machine, and consequently any bending of the column or unequal distribution of compression stress a sensitive machinist's level was placed upon the testing machine base and read during the test at the same time as the compression readings. Since only one level was available readings could not be taken in two directions, which would have been preferable. The positions of each end of the bubble in this level were read, thus enabling the location of the center of the

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[3] Johnson's formula $34\ 000-88\frac{L}{R}$ gives a reduction of 10 per cent. (Johnson's Materials of Construction.)

Prof. Bauschinger's formula $80\ 000-700\frac{L}{R}$ gives a reduction of 38 per cent, evidently allowing for the imperfections mentioned above.

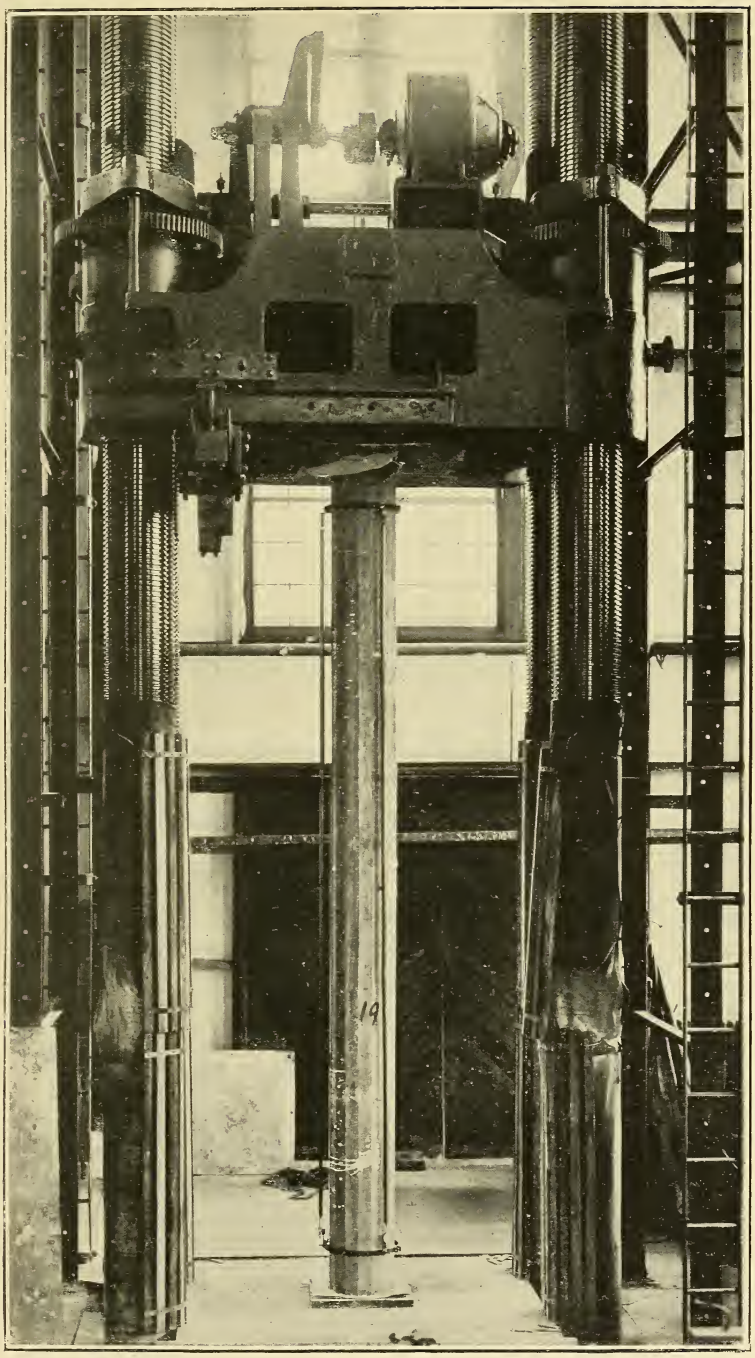

FIG. 6.—*Showing column 19 ready for testing*

bubble to be found for each load. The location of the center of bubble was plotted against applied load for all columns. The curves thus obtained were rather irregular and showed no inclination of the platen to move in any definite manner. The largest maximum deviation showed the base to be turned through an angle of 2.75 minutes, which is extremely small, or a movement of 0.096 inch in 10 feet. An increase in the angle of deviation at the greatest load at which readings were taken might reasonably have been expected, since the strain ordinate of the stress-strain diagram commences to increase more rapidly at this point. No such indication was evidenced, thus showing that any bend in the stress-strain curves was not due to bending or flexure of the columns.

(*b*) *Behavior of Columns Under Load.*—The method of failure of all columns was identical. The spalling of small flat particles, the largest being ¼ of an inch square, was the first indication of failure. These particles were arranged in lines usually inclined approximately at 45° to the horizontal. Cracks, usually vertical or sometimes horizontal, also appeared shortly after the first signs of spalling. The column shell (that is, the concrete outside of the spiral) next commenced to scale off in large sections, leaving a very even surface on the remaining concrete. Any portion of the coarse aggregate that extended through the cylindrical surface containing the spiral was sheared in the plane of that cylinder. As can be seen from the photograph of the tested columns, the shell was never completely sheared off, even after the maximum load had been applied. The plain reinforced columns 7 and 8 show the covering in much better condition than those reinforced with cast iron. No general tendency was shown by the columns to begin to lose the shell or to have the shell come off more completely in any particular section of the length of the column.

At maximum load the wire spirals commenced to break in tension. The relative locations and times of failure of these wires were quite irregular. Several wires might fail at approximately the same instant or a long interval might elapse between failures. Adjacent wires were no more likely to fail in sequence than wires in entirely different parts of the column. Usually after several spirals had failed at one point the column would begin to bend perceptibly at that point, the vertical rods buckling outward where the spiral had failed. The spirals failed only at places where

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the shell was completely sheared off. It is quite probable that the spalling of the shell at these points was augmented by incipient failure of the spiral.

At points where several spirals had failed the concrete would gradually fall out and expose the cast-iron reinforcement. The cast iron thus exposed would eventually fail violently, but due to the surrounding concrete and spiral, without doing any damage. In some cases, as in columns 14, 15, and 16, with the **I** section, the cast-iron reinforcement broke without being exposed, the failure being quite perceptible, evidenced by a dull, heavy thud. The apparently anomalous condition of the cast iron alone bearing the load in the machine after the surrounding concrete and spirals had failed is readily explained by the fact that the spirals and concrete fail at a much smaller strain than that which cast iron will undergo before failing.

Figs. 7 to 11 show the columns after test, and Figs. 12 and 13 the cast-iron reinforcement which was taken out of the columns after their being tested. Fig. 13 also shows the plain cast-iron specimens of 2 feet and 8 feet lengths after test.

Table 1 gives the various maximum strengths and other data from the tests and the general data of the design of the columns.

The Berry gage readings gave little or no information of value, except that there was no relation between the strain at any two points whether steel or concrete. The curves were not smooth but irregular, due to both uneven distribution of stress in the column, and also to difficulty in obtaining sufficiently accurate readings with the instrument on a 6-inch length.

The strains used in the diagrams in this paper are the averages of the three obtained from the long-gage lengths and measured by the Ames gage.

(*c*) *Stress-Strain Relations.*—Fig. 14 gives the stress-strain and Fig. 15 the load-strain diagram for all the columns tested. Both of these diagrams give useful information. The stress-strain diagram, showing the relative stiffness per unit area of cross section, gives a means of comparison in economy of material. The load-strain diagram gives the stiffness under the total load, showing the behavior of the column as a unit, and gives a means of comparing the individual strength of each column, and hence of columns occupying the same floor space.

FIG. 7.—*Showing 8-foot columns after failure*

FIG. 8.—*Showing 10-foot columns after failure*

Cast-iron I shapes were used in columns 15 and 16. The hollow cylindrical shapes used in columns 13 and 14 were spliced

FIG. 9.—*Showing columns 17, 18, 19, and 20 after failure*

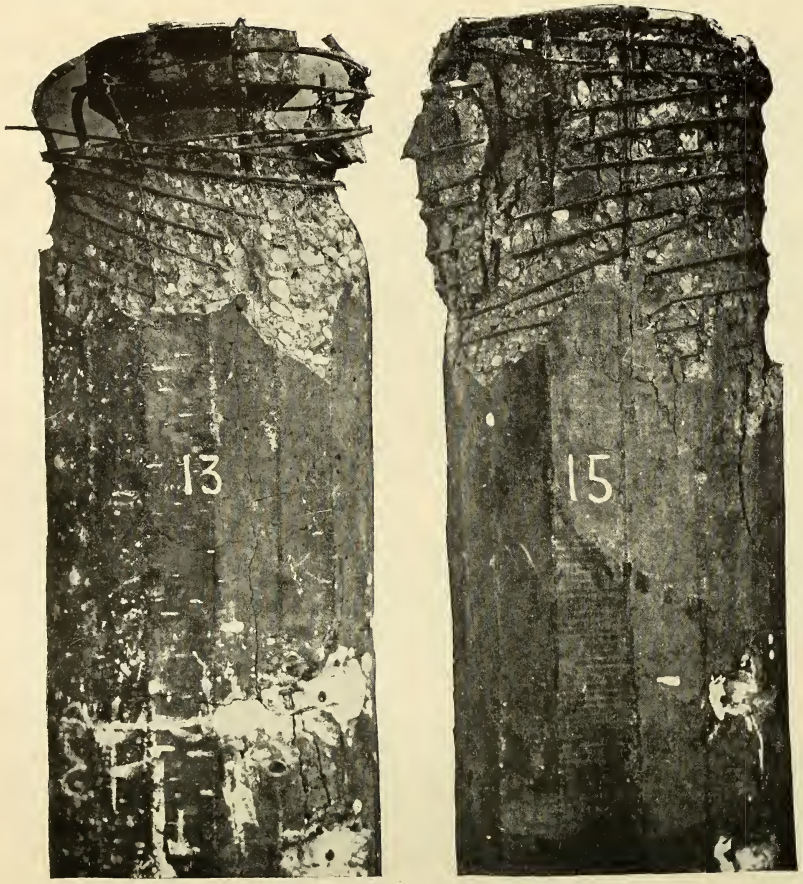


FIG. 10.—*Showing end failures in columns 13 and 15*

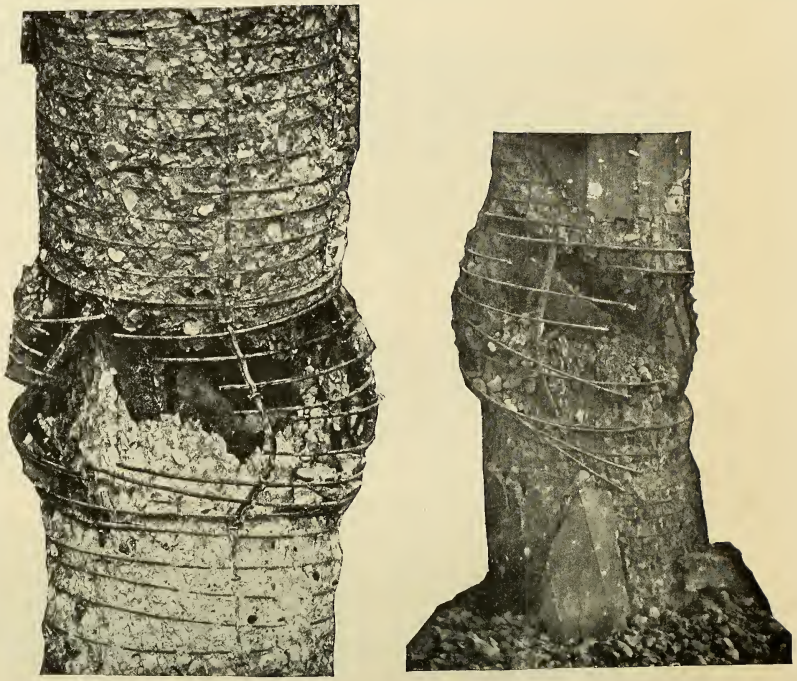
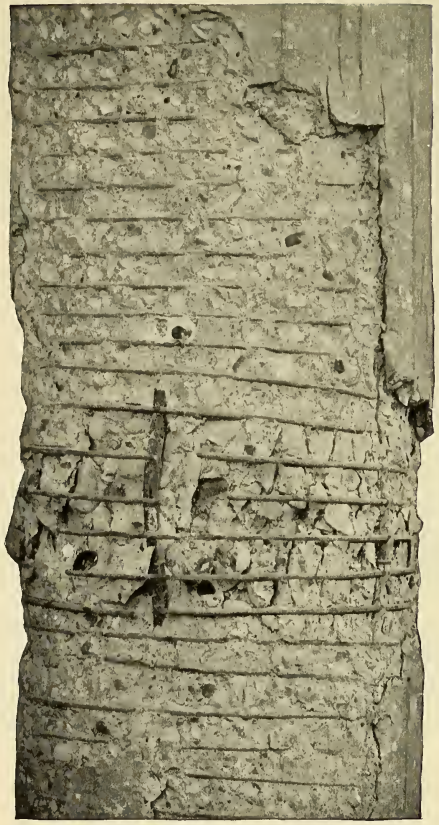


FIG. 10a.—*Showing failure of cast-iron reinforcement after spirals failed in columns 3 and 6*



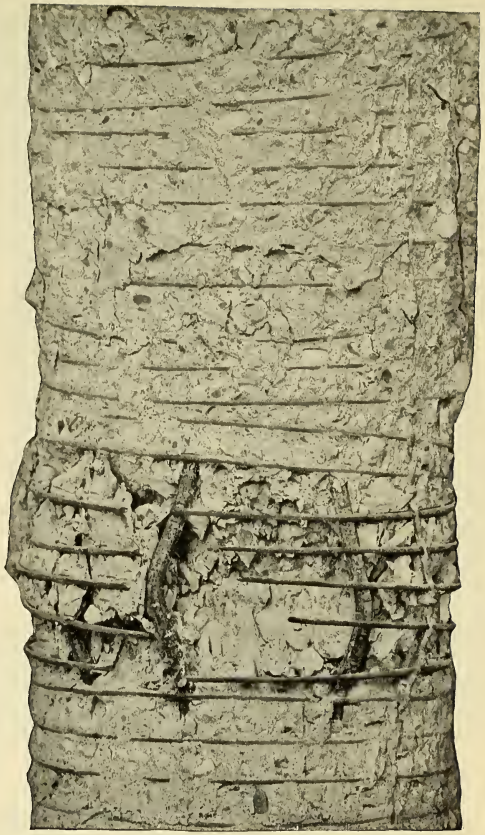


FIG. 11.—*Typical failures at maximum load, showing broken spirals and buckling of vertical steel. This figure also shows the shearing of the coarse concrete aggregate along the surface of the spiral reinforcement*

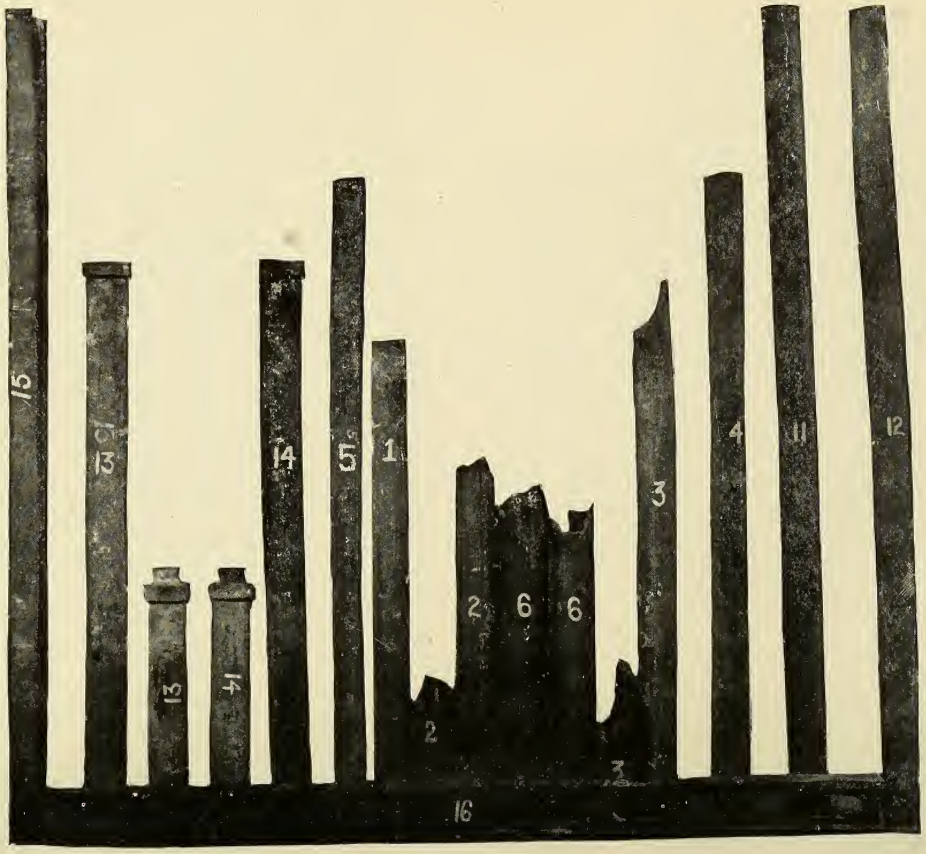


FIG. 12.—*Cast-iron reinforcing taken from columns after testing*

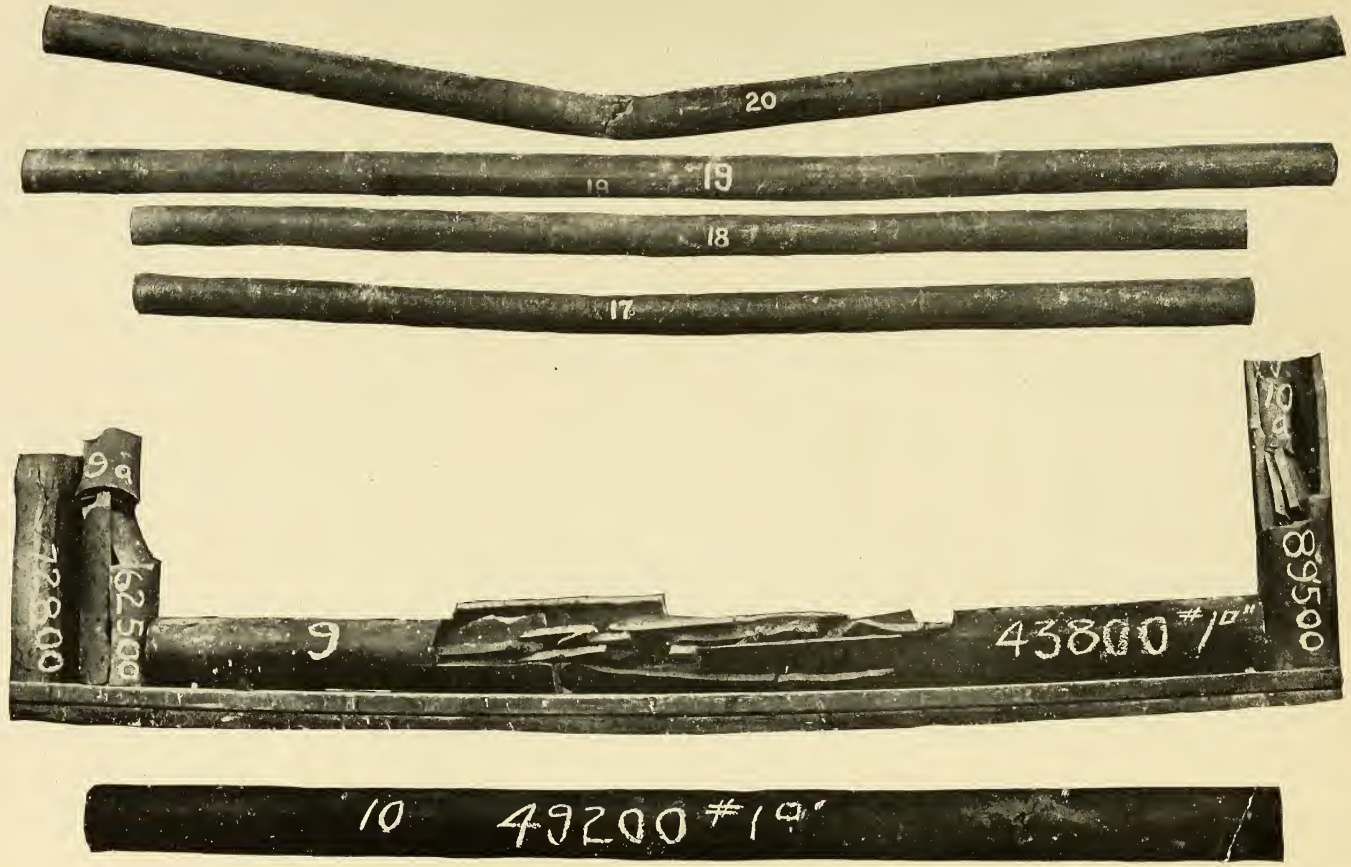


FIG. 13.—*Cast-iron reinforcing from columns 17, 18, 19, and 20; also plain cast-iron specimens after failure*

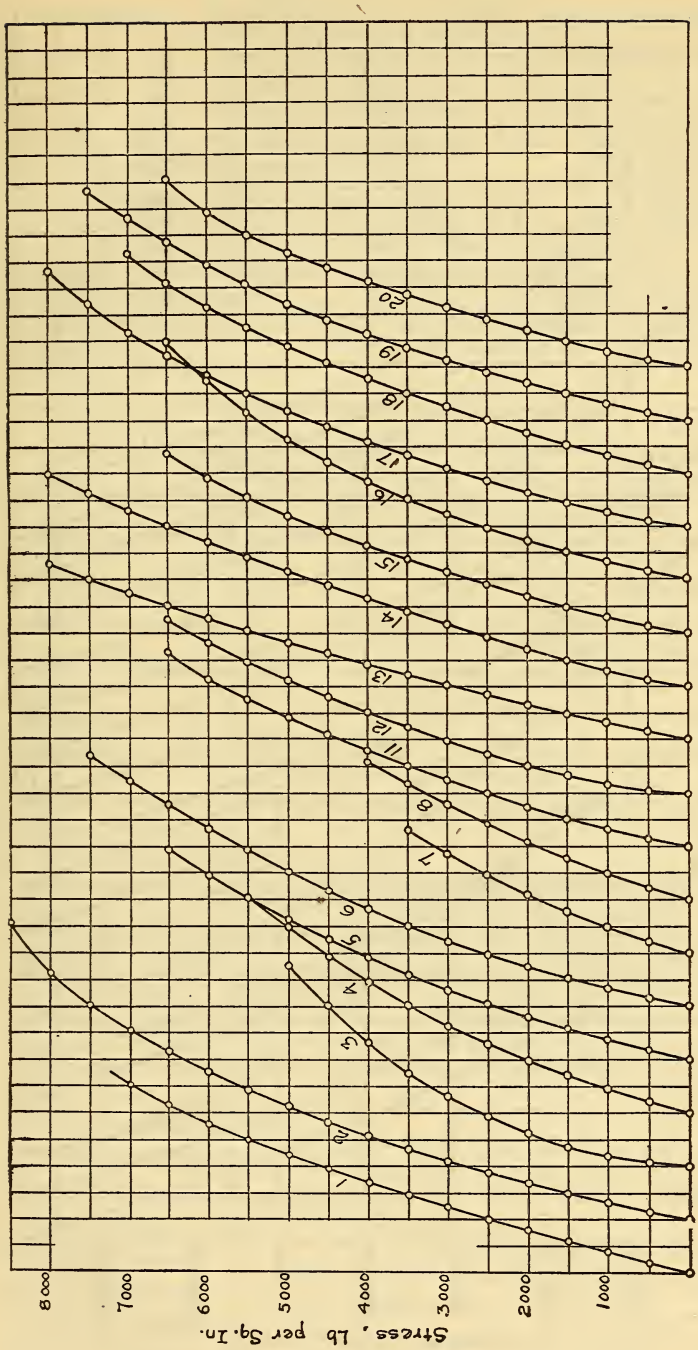


FIG. 14.—*Stress strain diagram for all concrete columns*

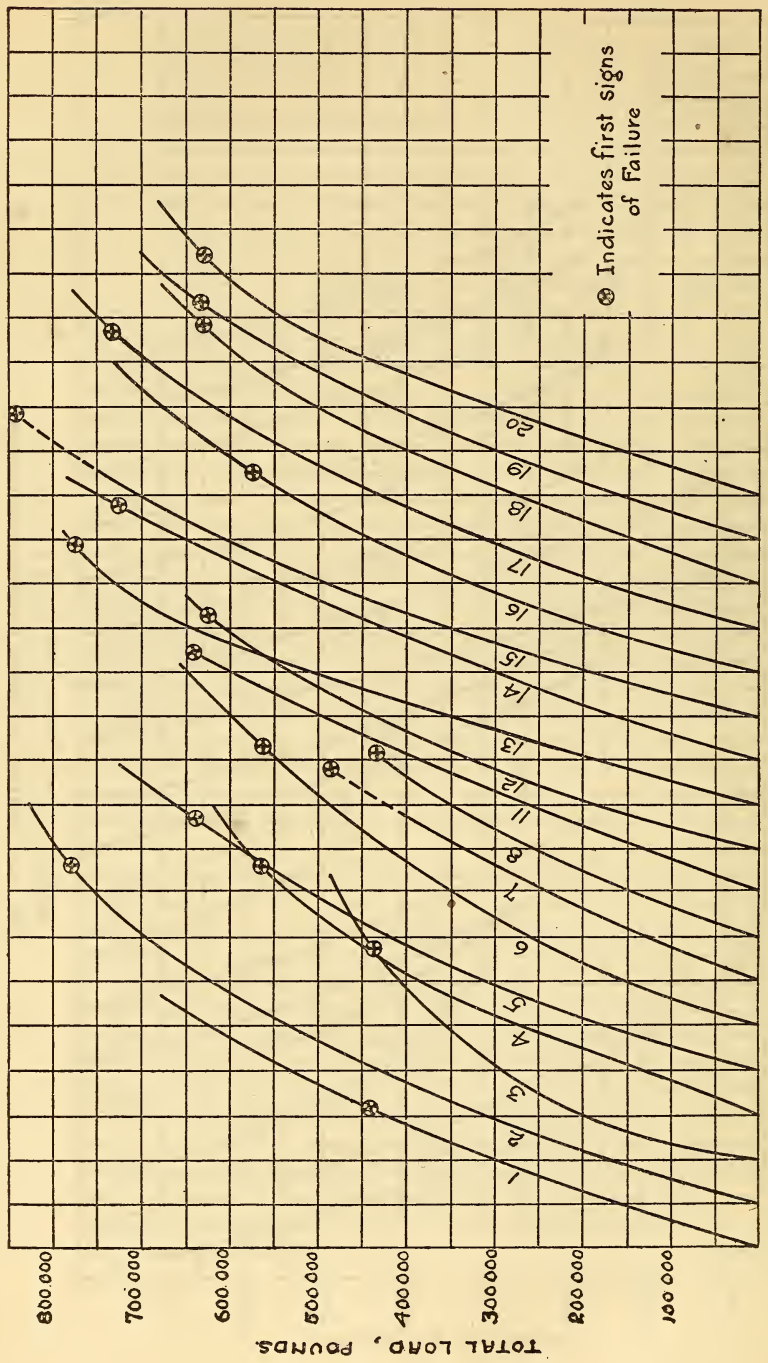


FIG. 15.—*Load strain diagram for all concrete columns*

V. DETAILED DESCRIPTION OF TESTS.

1. COLUMNS OF THE SAME LENGTH AND VARIOUS CROSS SECTIONS

(*a*) *Columns 8 Feet Long.*—For convenience of reference, Fig. 1 and Table 2 give the details of construction and the results of the tests of the four 8-foot columns In Fig. 16*B* is given the stress strain and load strain for this set of columns.

Column 3 was probably defective in some manner, as can be seen by the relatively weak stress-strain curve.

The initial stiffness of this group of columns for points previous to failure, with the exception of column 3, is proportional, or at least in the same order as the values of E calculated from the formula on page 31.

From these four columns it appears that variation of amount of concrete within the spiral and outside the cast-iron reinforcement has no appreciable effect. The concrete within the cast-iron core appears to increase the strength of the column 6500 pounds for each square inch of this concrete filling. This increase is due to two actions: (1) At places in the cast iron that are not properly welded and where instead of metal being homogeneous from the inner to outer surface it is in layers (as at a chaplet), the concrete prevents this layer from buckling and failing in the manner of a long column; (2) It prevents the cast iron from expanding unrestrainedly in a longitudinal direction, the strain resulting from a given stress being, therefore, less. The concrete between the core and the spiral adds strength to the cast iron in the same manner.

Columns 3 and 6 had cast-iron reinforcement of the same cross-sectional area, but with different radii of gyration. Apparently difference in radius of gyration did not affect the maximum strength of the column, a result to be expected in columns of such a short length. Considering the stress in the columns developed at the maximum load, we see that column 6 is the strongest, carrying 21 per cent more stress than column 5, the weakest. Apparently, then, column 6 is the most economical for the distribution of the cast-iron reinforcement in a tube of large diameter, giving a greater stress and permitting the hollow inside bore to be used for ventilation or heating, etc.

(*b*) *Columns 10 Feet Long.*—The second group of columns of the same length (10 feet) and of variable details in cross section was productive of more reliable results, since there were two identical specimens of each type of cross section tested. This group contained the two columns 7 and 8, with the type of cross section in common use to-day, having wire spiral and longitudinal steel rods, but not cast iron, as reinforcement.

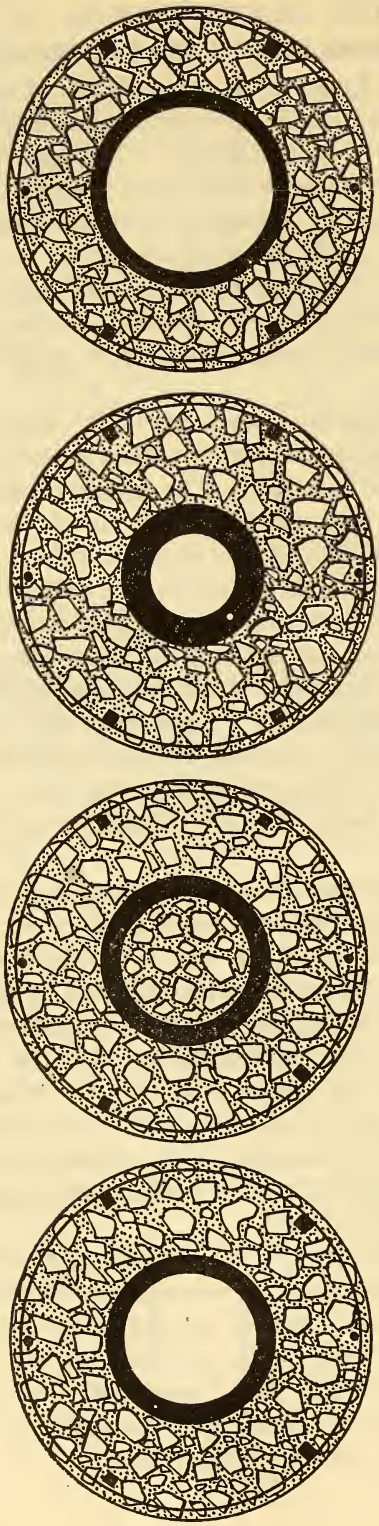


FIG. 16A.—*Transverse sections of columns (see Table 2)*

TABLE 2.—Comparison of Results of Tests on 8-Foot Columns

Column number	Construction data										Test data		
	Reinforcement (cast iron)[a]		Area				Per cent area				Load		Stress at maximum load
	Outer diameter	Thickness	Spiral equivalent	Longitudinal steel rods	Cast iron	Concrete	Spiral	Longitudinal steel rods	Cast iron	Concrete	At first sign of failure	Maximum	
	Inches	Inch	Inch²	Inch²	Inches²	Inches²					Pounds	Pounds	Lbs/in.²
3	6	¾	0.69	0.66	12.33	84.2	0.71	0.68	12.7	86.6	436 500	940 000	9 680
4	6	¾	0.69	0.66	12.33	100.1	.61	.58	10.9	88.5	570 000	1031 000	9 110
5	5	1	0.69	0.66	12.57	92.2	.65	.62	11.9	87.0	630 500	925 000	8 730
6	7	⅝	0.69	0.66	12.51	73.9	.79	.76	14.4	84.9	565 500	920 000	10 560
9	6	¾	.....	.....	12.33	.....	.....	.....	100	.....	.....	480 000	39 000
10	6	¾	.....	.....	12.33	.....	.....	.....	100	.....	.....	540 000	43 800

[a] Spiral reinforcement consisted of No. 8 gage wire, 1⅛-inch pitch, around core 12 inches in diameter, and the longitudinal reinforcement consisted of four ¾-inch square rods and two ¼-inch circular rods.

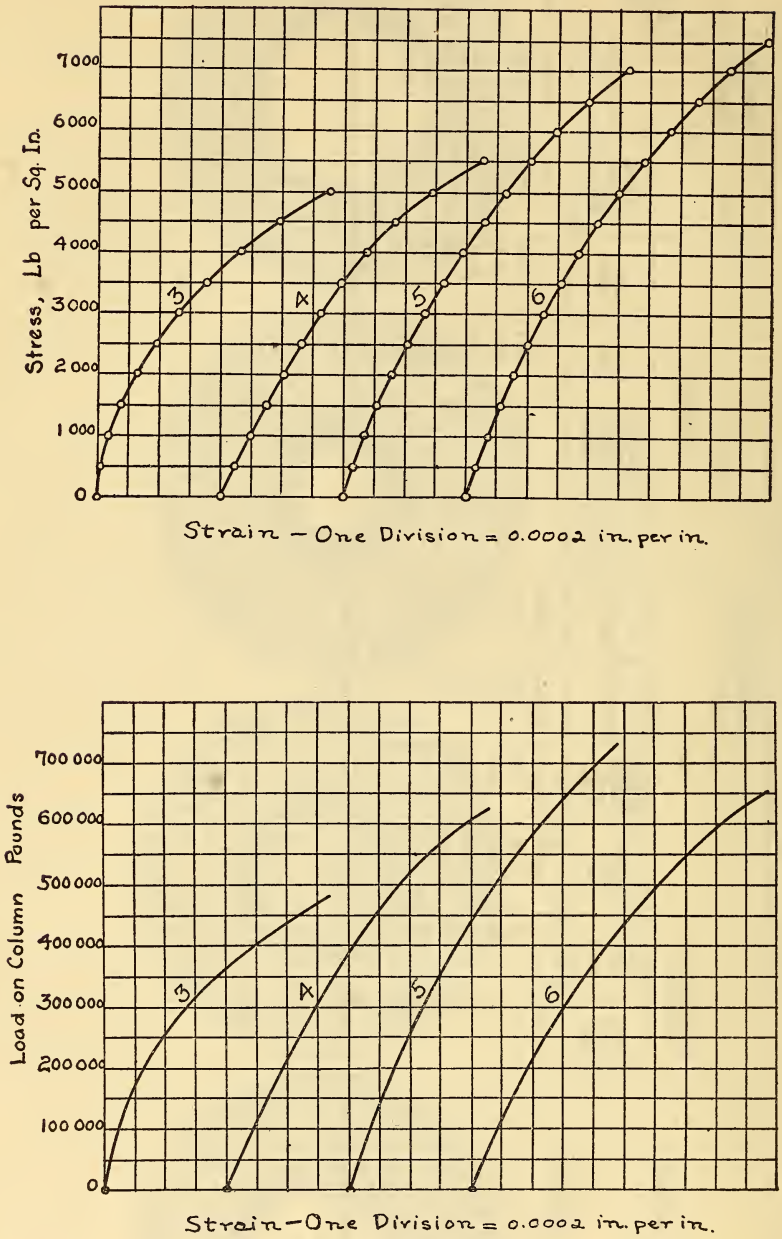


FIG. 16B.—*Stress strain and load strain diagrams for columns 8 feet long*

In order of stiffness these columns range 13–14, 15–16, 11–12, 7–8. With the exception of 13–14, which should follow instead of precede 15–16, this is the order calculated from the value of E as given by the formula on page 31. The great stiffness obtained in columns 13–14 may perhaps be explained by (1) the more careful placing and tamping of the concrete around the cast-iron reinforcement at the splice and at the ends; (2) the extra metal in the collars at the splices and at the ends of the cast-iron tubes; and (3) the better and stronger castings which were obtained for this type of reinforcement because the sections were shorter and had enlarged ends.

Evidently a joint such as used in these columns does not appreciably effect the strength of a 10-foot column, and could be used with safety. However, such a joint appears offhand to be too difficult to fabricate in a first-class manner in the field, and would not have strength to withstand any accidental blows upon the reinforcement previous to pouring and setting the concrete. A type of flanged and bolted joint would be an improvement, in that it would allow the cast-iron skeleton to be erected independently of and previous to the forms. Mr. Mensch has already built spliced columns of an improved type to which this criticism does not apply.

In this 10-foot length an I section was stronger than the plain tube and showed no signs of failure by buckling around the short axis of the I until after maximum load had been passed and a dozen spirals had failed.

Considering the stress-strain diagram and the maximum stress carried by the four types (Fig. 17*A*, *B*), the columns with joints appear to be the best, as the strength is greater, respectively, than that developed in the columns with the plain cast-iron tube and the columns with the I section. The three types reinforced with cast iron withstood for the plain tube 91.5, for the tube with joint 112, and the I section 91 per cent more than the reinforced columns (7 and 8) without cast iron.

2. COLUMNS OF THE SAME CROSS SECTION AND DIFFERENT LENGTHS

The stress-strain and load-strain diagrams are exactly similar, all the cross sections being the same (Fig. 18). As previously seen, column 3 is apparently defective in some way. From the others nothing of much value can be concluded. Apparently the length (within the limits of the columns constructed) does not affect the initial modulus of elasticity, as would be expected from theory.

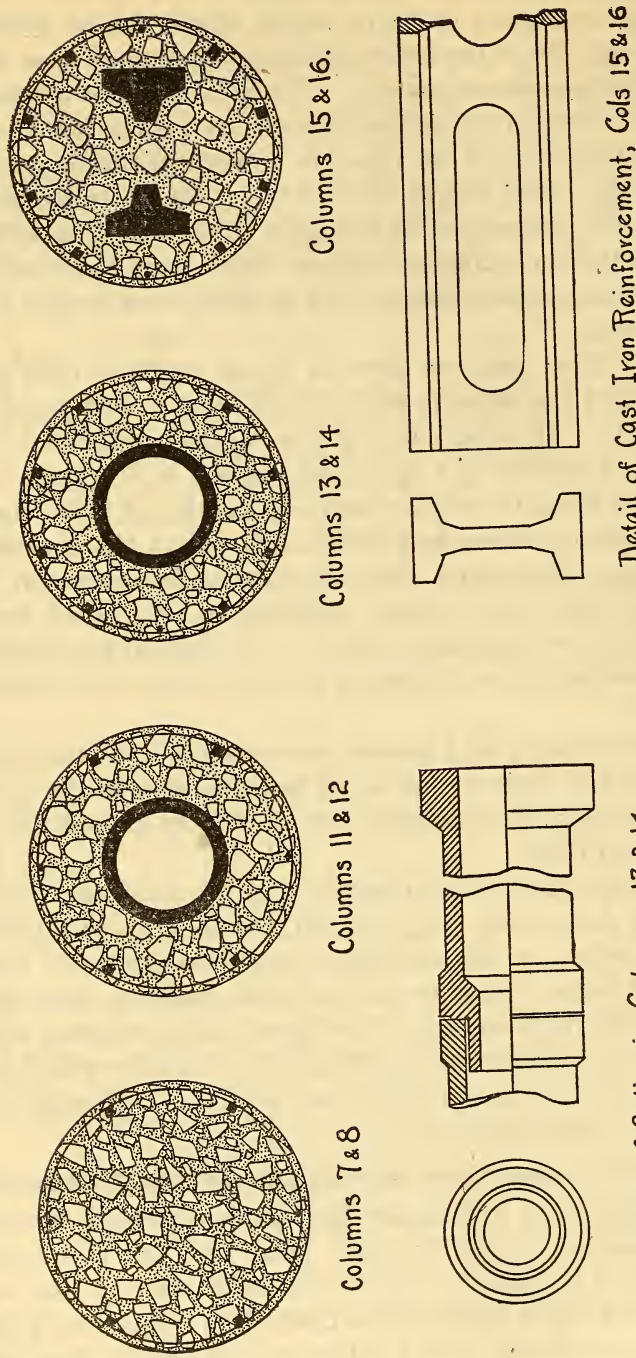


FIG. 17A.—*Sections of 10-foot columns transverse and longitudinal (see Table 3)*

TABLE 3.—Comparison of Results of Tests on 10-Foot Columns

Column number	Reinforcement (cast iron)[a]		Area				Per cent area				Load on columns		Stress
	Outer diameter	Thickness	Spiral equivalent	Longitudinal steel rods	Cast iron	Concrete	Spiral	Rods	Cast iron	Concrete	At first signs of failure	Maximum	Maximum load
	Inches	Inch	Inch²	Inch²	Inches²	Inches²					Pounds	Pounds	Lbs/in.²
7	None.	.....	0.69	0.66	None.	113.1	0.61	0.58	None.	99.4	490 000	518 500	4580
8	None.	.....									440 000	562 200	4970
Mean	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	465 000	540 350	4775
11	6	¾	0.69	0.66	12.33	84.2	.71	.68	12.7	86.6	643 500	911 000	9380
12	6	¾									625 000	940 000	9680
Mean	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	634 250	925 500	9430
13	([b])	([b])	0.69	0.66	[c]12.33	83.7	.71	1.26	12.7	.....	776 000	896 000	9230
14	([b])	([b])									727 500	1066 000	10 980
Mean	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	751 750	981 000	10 105
15	([b])	([b])	0.69	0.66	13.4	98.5	.61	1.08	11.8	87.0	850 000	1071 800	9480
16	([b])	([b])									576 000	996 000	8800
Mean	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	713 000	1033 900	9140

[a] Spiral reinforcement consisted of No. 8 gage wire, 1⅛-inch pitch, with core 12 inches in diameter. Longitudinal reinforcement consisted of four ⅜-inch square steel rods and two ¼-inch circular steel rods for columns 7, 8, 11, and 12, and eight ¾-inch square steel rods and two ¼-inch circular steel rods for column 13 to 16.

[b] Same as columns 11 and 12, but with joints and enlarged ends I section.

[c] Variable due to enlarged section at ends and joint.

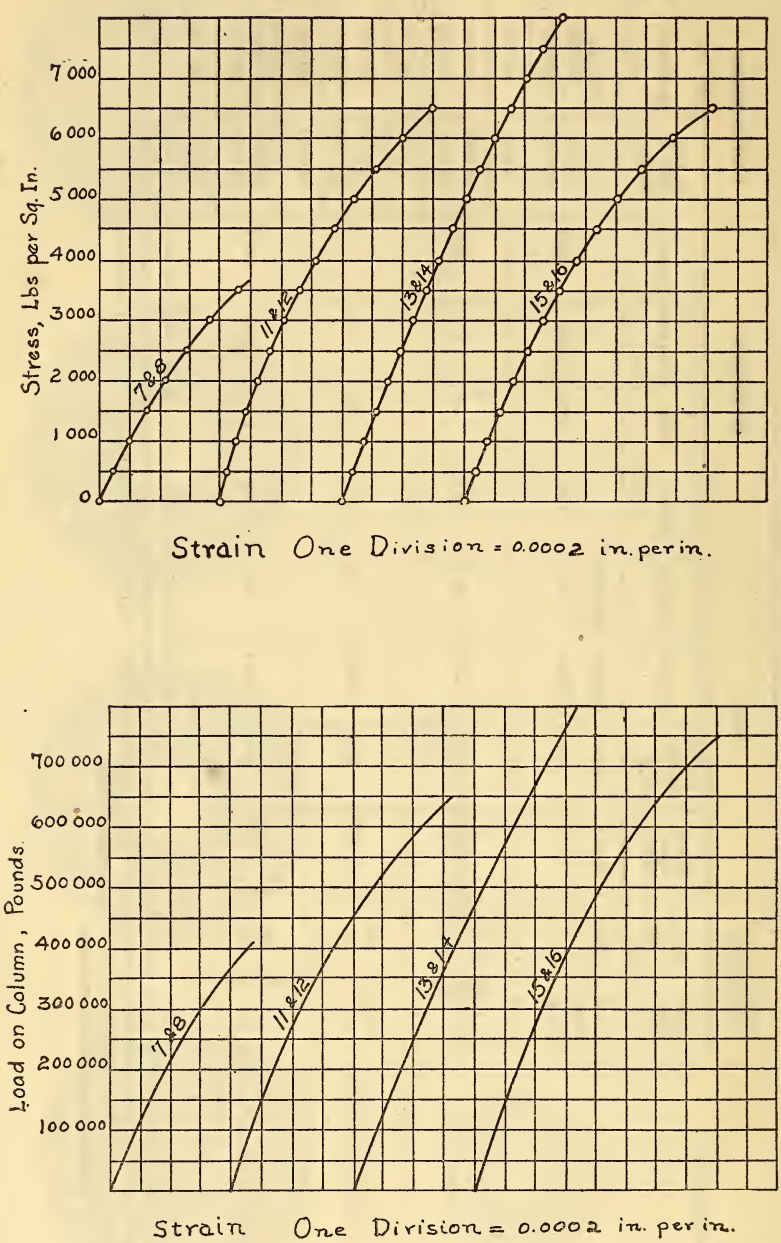


FIG. 17B.—*Stress strain and load strain diagrams for columns 10 feet long*

TABLE 4.—Comparison of Results of Tests on Columns of the Same Cross Section and Varied Lengths

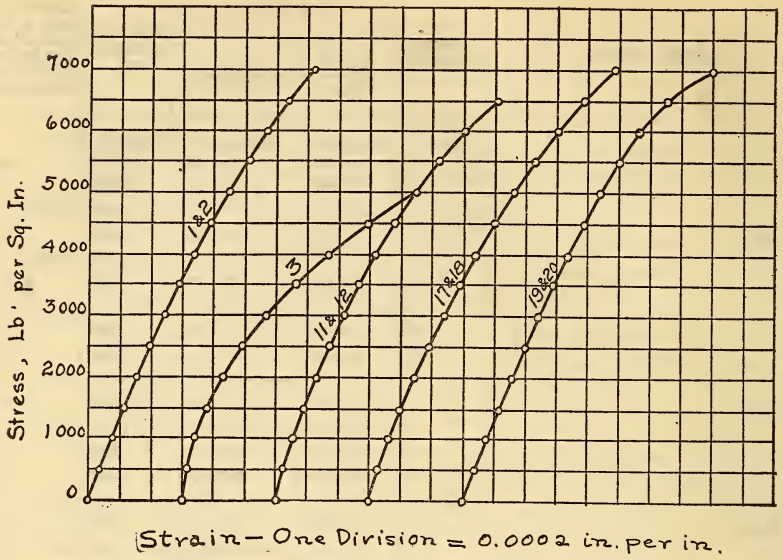
Column number	Length of columns	Load on columns		Stress at maximum load	Load on column at first signs of failure in per cent of maximum load
		At first signs of failure	Maximum		
	Feet	Pounds	Pounds	Lbs./in.²	
1	6	436 500	1 057 000	10 880	41.5
2	6	776 000	1 026 000	10 560	75.5
Mean		606 250	1 041 500	10 720	58.5
3	8	436 500	940 000	9680	46.3
11	10	643 500	911 000	9380	70.5
12	10	625 000	940 000	9680	66.5
Mean		634 250	925 500	9530	68.5
17	12	732 500	951 500	9800	77.0
18	12	630 500	909 000	9360	69.3
Mean		681 500	930 150	9580	73.2
19	14	630 500	888 500	9150	70.2
20	14	679 000	827 500	8520	82.0
Mean		654 750	858 000	8835	76.1

Reinforcement	Area	
	Square inches	Per cent
Spiral: No 8 gage wire, 12-inch diameter, 1⅛-inch pitch	0.69	0.71
Rods: Four ⅜-inch square, two ¾-inch circular	.66	.68
Cast iron: 6-inch outside diameter, ¾-inch thick	12.33	12.7
Concrete	84.2	86.6

Studying the maximum load carried by these columns and their relation to the length as plotted in Fig. 19, we find an apparently very definite relation extant. The straight line drawn represents the plotted points very closely, the average deviation of the points from the curve being but 2.6 per cent, the maximum difference of any result from the derived line only 5 per cent.

The equation of these lines is $P = 1\ 180\ 000 - 1930l$, or dividing by the cross-sectional area of the column inside of the spiral (the concrete of the shell being totally destroyed and valueless as to strength at this point) we obtain the equation $\frac{P}{A} = 12\ 150 - 201$.[4]

[4] P=load on column (in pounds).
A=area of column within spiral.
l=length of column in inches.



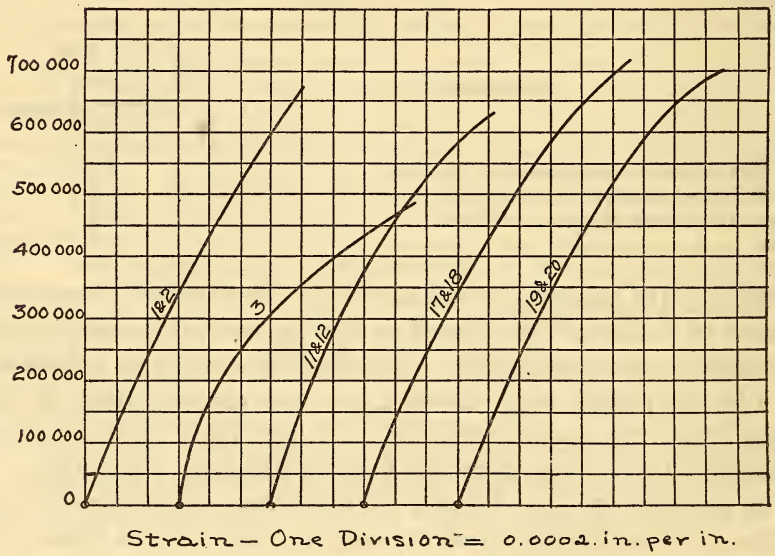


FIG. 18.—*Stress strain and load strain diagrams for columns of different lengths and the same cross section*

The average strength of the concrete as shown by the small concrete cylinders was 4060 pounds per square inch, and the increased strength of spirally reinforced concrete is given by the formula $S = (1 + 0.61\ p_s C)$, in which

S = the maximum stress withstood by the reinforced concrete,

p_s = per cent spiral reinforcement,

C = maximum stress withstood by the unreinforced concrete.

We can therefore write the probable equation:

$$\frac{P}{A} = 4060\ (1 - p_c)\ (1 + 0.61\ p_s + 63\ 000\ p_c - 20\ Z, \text{ in which}$$

p_c = per cent cast-iron reinforcement.

p_s = per cent spiral reinforcement.

This general formula is purely hypothetical, being derived from one set of columns only, all having the same percentage of cast-

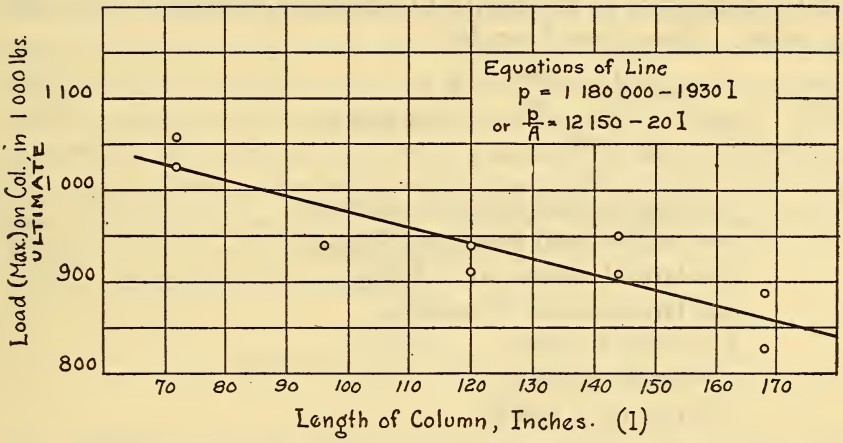


FIG. 19.—*Relation of length to strength. Columns of same cross section*

iron reinforcement. However, with data upon the tests of columns with other percentages of cast-iron reinforcement, a check will be afforded. In all probability the form of this formula will be correct and a change of the constants 4060 and 63 000 all that will be necessary.

VI. THEORY OF THE DESIGN

1. THEORETICAL MODULUS OF ELASTICITY [5]

As a rational basis for the computation of the initial modulus of elasticity or stiffness of the columns, we can write the simple f rmula:

$$E = \frac{P_i}{100} + \frac{P_c}{100} E_c + \frac{P_s}{100} E_s, \text{ or}$$

$$E = 10 \times 10^4 \times P_i + 3 \times 10^4 \times P_c + 30 \times 10^4 \times P_s$$

[5] It is to be noticed here that the initial modulus of elasticity bears no relation to the ultimate strength of the column. The modulus is discussed here because it is frequently valuable, when the compression of the column is to be found for a calculated load.

The symbols having the following significance:

E_i = modulus of elasticity for cast iron,
E_c = modulus of elasticity for concrete,
E_s = modulus of elasticity for steel.
P_i = per cent of total column area cast iron,
P_c = per cent of total column area concrete,
P_s = per cent of total column area column steel.

The effect of the spiral has not been taken into consideration, it being very small, less than 1 per cent.

2. STRESS-STRAIN RELATIONS FOR REINFORCED COLUMNS

The maximum strength is obtained in the columns tested in this group at the point at which the spiral commences to fail. For the purpose of this discussion we assume the internal behavior of these columns to be identical to columns reinforced only with a spiral. From the formula[6]

$$p = \frac{meE_c\left(m - \frac{2m}{nr} - 1\right)}{m^2 - \frac{2m^2}{nr} - m - 2}, \text{ in which}$$

e = longitudinal strain in column.
p = longitudinal stress in column.
p_i = lateral stress in column.
p_l = lateral strain in column.
p_s = stress in steel.
e_s = strain in steel.
r = per cent spiral.
$\frac{l}{m}$ = Poisson's ratio (approximately 1/8 for concrete).

---

[6] This and the following formula derived as follows:

Equations.	Unknowns.	Known.
1. $e = \frac{p-2p}{E_c\ ME_c}$	p	e
2. $e_1 = \frac{pl}{E_c} = \frac{l}{m}\ \frac{pl-p}{E_c}$	p_1	m
3. $p_s = \frac{pl}{r/2}\ 2\ p = rps$	e_1	E_c
4. $p_s = e_s\ E_s$	p_s	E_s
5. $e_1 = e_s$	e_s	$\left(\frac{Es}{E_c} = n\right)$

1 and 2 are derived directly from the use of Poisson's ratio.

Considering a lateral cut through the axis of the cylinder, the compression in the concrete must equal the tension in the steel since two forces in equilibrium must be equal. Hence we obtain equation No. 3.

Eliminating the unknowns from these equations we obtain two equations, giving values of P and p_s in terms of wholly known quantities.

$$p_s = \frac{1}{r}\ \frac{2\ m\ e\ E_c}{2 + m + m^2\left(\frac{2}{rn} - 1\right)}$$

$$P = me\ E_c \frac{\left(m - \frac{2m}{rn} - 1\right)}{m^2 - 2m^2 - m - 2}$$

we see that at the strain at which ordinary unreinforced concrete would fail, the increase in stress carried by the spirally hooped column is but a fraction of 1 per cent. This formula is also valid for the Emperger type of column, the area of cast iron being considered simply as giving additional strength in proportion to its modulus of elasticity and strain.

The increased strength in spiral reinforced columns is due to the fact that the spiral restrains the concrete and permits several times the strain which would cause failure in the unrestrained concrete. The maximum stress withstood by the column will then be readily seen to depend upon the ultimate strength of the steel spiral.

When the spiral fails, the concrete, already stressed beyond what it could carry, immediately fails as unreinforced concrete. Since the failure of spirals occurs in only a small section of the length of the column, only the concrete at the places of failure of the spirals would be expected to be destroyed. The concrete within the spiral that was still sound would also be sound. This is actually the case, an opportunity of testing the soundness of the concrete being afforded when the cast-iron reinforcement was dug out of the concrete. The concrete, from the strength exhibited, was apparently uninjured.

The spirals, from the formula

$$p_s = 1 \times \frac{2mcE_{c2}}{2 + m + \frac{m(2-1)}{rn}}$$

are stressed to about 8500 pounds per square inch when the concrete shell begins to crack and still have the greatest part of their strength to call into play. Although bearing this considerable stress (of 8500 pounds per square inch), they have not perceptibly increased the strength of the columns for any strain below 0.0015. The manner whereby the spiral adds strength to the concrete, as just described, is due to the restraining effect of the spiral upon the concrete, permitting the concrete to be strained beyond 0.0015 inch per inch. The column's maximum strength is therefore regulated by the maximum strength of the spirals.

It is most probable that if a new type of column were molded, with spirals directly on the outer surface of the column, greater

accuracy in placing of the spiral would result. If some fire-resisting material, possessing the properties of lower heat conductivity, greater imperviousness when subjected to heat, and the ability to withstand a greater strain than 0.0015 before failure (as much as 0.004) were next added as a protective coating to the column, a far superior type would result.

3. LIMIT OF PER CENT OF CAST-IRON REINFORCEMENT

There is an upper limit of the percentage of cast-iron reinforcement that can be used economically in this type of column. If more than a certain definite per cent of cast iron be used, we will have the phenomena of a column having two maximum strengths—first, when the spirals fail, and, second, a higher maximum when the cast iron alone and unsupported fails.

The maximum economical per cent of cast iron will vary with the relative strengths of the concrete and cast iron, and also with the per cent of spiral reinforcement used. The two cases examined below are for a 1:1:2 mixture with 0.61 per cent spiral.

Considering a cast-iron reinforced concrete column with 16 per cent of its area of cast iron, the cast-iron reinforcement tested as a column alone will carry as its maximum load the same load that will cause the reinforced column to show initial signs of failure through the cracking and incipient scaling of its shell. In a column of the same type, but with 22½ per cent cast-iron reinforcement, we find that the cast-iron reinforcement alone can withstand as much load as the reinforced column.

This can very readily be appreciated, since the cast iron at the time of failure of the spirals is carrying, say, but 60 per cent of its maximum load. If the hooped concrete is carrying less than the extra load that the cast iron will carry, then we have the phenomenon previously predicated, a maximum at the failure of the spirals, then a second maximum attained by the cast iron alone.

However, the cast-iron column above would be far less reliable, due to probable unevenness in texture, etc., and would also not be as safe from a fire-resisting standpoint.

The maximum economical per cent of cast iron to be used can be determined by the formula:

$$\text{Per cent cast iron} = \frac{100 f_e}{f_i + f_c - f_v}$$

f_c = Maximum stress withstood by the hooped concrete. This can be computed by use of the formula: Increase of strength of hooped over unrestrained concrete $= \frac{61}{100} \times$ per cent spiral.

f_i = Maximum stress withstood by cast iron.[7]

f_v = Stress in the cast-iron reinforcement when the column is sustaining maximum loads. This value is about 32 500 pounds.

VII. CONCLUSIONS

1. The behavior of the Emperger columns tested to failure is similar to that of ordinary concrete columns reinforced with spiral and longitudinal steel rods. Scaling and cracking of the shell or protective coating is the first sign of failure. The shell continues to scale until maximum load is reached at which point the spiral reinforcement fails through tension. After this the column begins to buckle, and finally the cast-iron reinforcement fails, not violently, however, since it is protected by the surrounding concrete.

The concrete shell begins to fail at the same strain (from 0.00105 to 0.00183 with average of 0.00145 in the columns tested) at which cylinders of plain concrete fail.

At the stage of initial cracking the stresses in the various members are approximately as follows

	Lbs. per sq. in.
Concrete[8]	4060
Cast iron	150000
Rods	30000
Spiral	8500

The concrete is up to its maximum unrestrained strength as are also practically the longitudinal steel rods.

The concrete upon increase of load assumes a larger and larger strain, failure of the column being prevented by the spiral reinforcement. The ultimate (and maximum) strength of the concrete depends upon the maximum strength of the spirals. When the spirals fail the concrete itself is being strained much beyond the failing point if unrestrained, and consequently is immediately

[7] This quantity varies with the thickness of the cast iron. In thin tubes ¾ of an inch thick, as were tested in this set of columns, only 41 500 pounds per square inch was withstood, whereas in the tubes of thicker walls, say 1½ inches, a stress of 60 000 pounds per square inch can safely be expected with the same kind of iron.

[8] The average of the maximum stress withstood by the plain concrete 6 by 12 inch cylinders.

destroyed. At the period of failure the stresses in the different materials are as follows:

	Lbs. per sq. in.
Concrete[9]	5 600
Cast iron	40 000
Rods (indeterminate)	0
Spirals (maximum strength)	60 000

2. This type of column, like the spiral-reinforcement column, gives ample evidence of approaching failure by the shell scaling off.

In the testing machine the column after maximum load fails slowly and gradually, very different from cast iron or terra cotta, which fails with sudden violence. However, it would be an error to suppose that the column will take (as in the testing machine) perhaps 15 minutes for complete failure after the maximum strength has been reached. With a static load the failure would be sudden and immediate after maximum load had been reached.

3. The economical limit of cast-iron reinforcement to be used is given by the formula

Per cent cast iron [10] $= \frac{100\, f_e}{f_i + f_e - f_v}$, or 21 per cent using the values determined in this test. The increase in strength of the concrete due to spiral reinforcement is 63/100 × per cent spiral.

Where f_e = maximum stress withstood by the hooped concrete: For 1 per cent spiral and 1:1:2 concrete this value appears to be 5300 pounds, or 33 per cent increase above the unrestrained strength of concrete.

f_i = maximum stress withstood by cast iron. A fair value for this would be 60 000 pounds.

f_v = stress in the cast-iron reinforcement when the column is sustaining maximum load. About 40 000 pounds.

4. Eccentricity and variation in pitch of the spiral and variation in depth of the longitudinal steel rods of the column apparently do not affect the strength of the column.

5. The maximum strength of the columns is not quite equal to the strength of the cast-iron reinforcement tested independently, plus the strength of spiral reinforced column. The cast-iron reinforced column has, however, much less area of concrete.

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[9] The strength of the mean of the small cylinders with increase due to 0.66 per cent (from 0.61 to 0.71 per cent in columns tested) of spiral reinforcement. (See p. 35 for formula.)

[10] Of gross columns cross section.

6. The columns commence to scale at from 58.5 to 77.5 per cent of maximum load (or an average of 70.4 per cent) as against 77.2 per cent and 94.7 per cent (or an average of 89.9 per cent) for the two plain spiral-reinforced columns tested. This pronounced indication of failure is an excellent feature in the behavior of the columns, since in practice it serves as a warning of overstressing, thereby permitting a low factor of safety to be allowed in the design of such columns.

7. The average maximum total load withstood by the various types reinforced by the Emperger system and of the same length as the plain spiral-reinforced columns was 81.5 per cent greater than for these plain spiral columns.

8. *Variation of Strength with Length.*—From the nine columns of the same cross section and lengths from 6 to 14 feet with 12.5 per cent cast iron (that is, of total area of cast iron, rods, and concrete within spiral). The formula $\frac{P}{A} - 12\ 150 - 20\ l$, is derived.

P = mean stress upon total area of column within spiral.

A = total area as above.

l = length of column in inches.

9. Filling the hollow cast-iron core of the columns apparently adds approximately 6500 pounds for each square inch of cross section of the concrete thus filled in. It also increases by 20 per cent the load withstood at the first signs of failure as indicated by spalling.

10. Varying the radius of gyration of the cast-iron-tube reinforcement, the cross section of the cast iron remaining constant and the tube hollow (not filled with concrete) appears to have the following results: (1) A 10 per cent greater load in the columns with the smaller diameter pipe before the first spalling or indication of failure; (2) the same ultimate strength and the same total load; and (3) the same strain curve for all columns.

11. The splicing of the columns evidently made the spliced sections stronger than the rest of the column section. The column did not buckle at these points. The columns were stiffer (that is, had a larger E) than any of the other columns and failed at the average maximum of the other columns of the same length.

12. The columns with I sections were apparently exactly similar in their action to those of the same per cent of cast iron in form of tubes and did not show any signs of bending until after the maximum load had been withstood.

13. In view of the general behavior of spirally reinforced concrete just preceding failure, it is believed that a superior type could be developed if the protective shell outside the spiral were applied after the column had been cast and the forms removed. This shell should be of some material possessing the necessary heat-insulating qualities and the ability to withstand without failure a considerably greater strain than 0.0015 (the strain at which unreinforced concrete fails). This method of construction would also make possible a much greater accuracy in the placing of the spiral.

WASHINGTON, June 25, 1918.