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TESTS OF COLUMNS: AN INVESTIGATION OF THE VALUE OF CONCRETE AS REINFORCEMENT FOR STRUCTURAL STEEL COLUMNS

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TESTS OF COLUMNS: AN INVESTIGATION OF THE VALUE OF CONCRETE AS REINFORCEMENT FOR STRUCTURAL STEEL COLUMNS

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY ENGINEERING AND IN CHARGE OF THEORETICAL AND APPLIED MECHANICS, AND ARTHUR R. LORD, RESEARCH FELLOW IN THEORETICAL AND APPLIED MECHANICS

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TESTS OF COLUMNS: AN INVESTIGATION OF THE VALUE OF CONCRETE AS REINFORCEMENT FOR STRUCTURAL STEEL COLUMNS.

I. INTRODUCTION.

1. *Preliminary.*—In reinforced concrete building construction columns form an important element, and in the case of very high or very heavily loaded buildings, the size of the columns and the space they occupy become important considerations. Various types of reinforced columns are in use. Columns with longitudinal reinforcement and hooped columns are common. During the past few years designers have used structural steel columns encased in concrete. Sometimes the structural steel shapes form a relatively small proportion of the column section and are considered as reinforcement for the concrete. In other designs the amount of steel is much larger and the structural shapes will carry a large proportion of the load so that the column instead of being a reinforced concrete column is really a steel column reinforced with the concrete in which it is embedded. Such columns may occupy less space than the reinforced concrete column as usually designed.

Two points of view seem to exist with reference to columns having a large percentage of structural steel: (a) that the concrete surrounding the steel simply affords protection from fire and corrosion and that the additional strength afforded by the concrete is not considerable in amount and is not available for design; and (b) that if the concrete be present it must act in unison with the steel and that its strengthening effect and its effect upon the permissible deformation of the column should be taken into account. The present building codes either directly or through the relation of stresses allowed virtually occupy the first position when the steel column forms more than 8% of the column section.

The series of tests described in this bulletin was planned to throw light upon the action of columns formed of structural steel shapes by filling the space between the shapes with concrete or encasing them in concrete as exemplified in a form of column section which has been used in reinforced concrete building construction. It is hoped that the results will be helpful in discussing fundamental principles underlying the design of such columns.

2. *Scope of Bulletin.*—The investigation was planned with the view of securing information on the following principal points for the section and type of column tested: (1) the effect of length and slenderness on the strength of the plain steel columns; (2) the effect of length upon the strength of similar columns made up with a core of concrete; (3) the effect of richness of concrete in the core filling upon the strength of the column; (4) the effect of adding an exterior coat around the steel section upon the strength of the column, and the action of this coat under load; (5) the effect of spiral hooping upon the strength and stress-deformations of the column.

The tests were planned in an effort to obtain the most information on these points with the 32 columns available for the

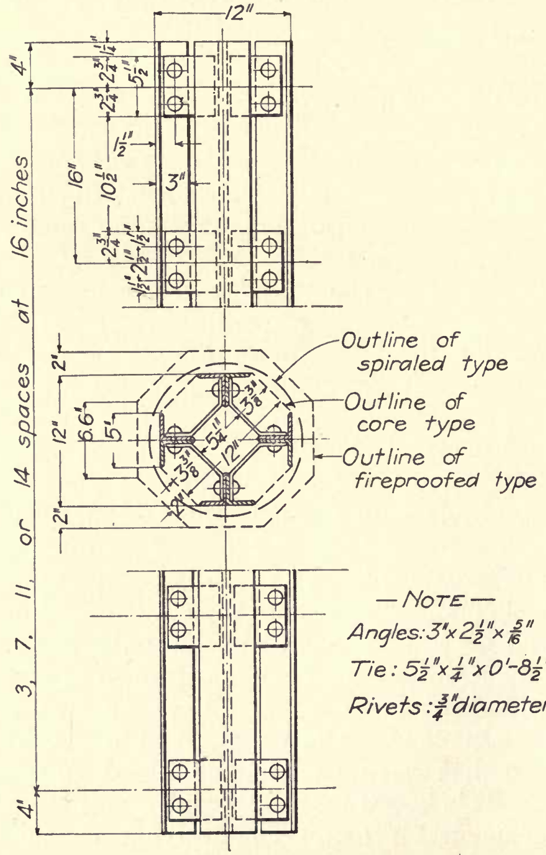

FIG. 1. DETAILS OF TEST COLUMNS.

tests. The results may not be applicable to sections or types of columns which differ from the form used.

3. *Acknowledgment.*—The steel columns used in the tests were furnished through the courtesy of the Illinois Steel Company, Mr. E. J. Buffington, President, and were made at the North Works of the Company in Chicago. The spirals for the six spiraled columns were furnished by the American System of Concrete Reinforcing, of Chicago. The work of concreting the columns was directly supervised by Mr. D. A. Abrams, Associate in the Engineering Experiment Station of the University of Illinois. The tests were made by Mr. Lord with the assistance of Mr. R. K. Steward and members of the staff of the Engineering Experiment Station. Mr. Lord is responsible largely for outlining the scope of the tests and for working up the results. Mr. W. A. Slater, First Assistant in the Engineering Experiment Station, has given helpful assistance in putting the bulletin into final form. Especial acknowledgment is made to Professor Frank P. McKibben and the Department of Civil Engineering of Lehigh University for the work of testing the four spiraled columns sent to the Fritz Engineering Laboratory and for the great interest and care so freely given.

II. Materials, Test Pieces, and Methods of Testing.

4. *The Steel Columns.*—The structural steel columns used were of the Gray type and were composed of eight angles, $3 \times 2\frac{1}{2} \times \frac{5}{16}$-in., tied at intervals of 16 inches by $5\frac{1}{2} \times \frac{1}{4}$-in. plates as shown in Fig. 1. This type of column has been used in the construction of reinforced concrete buildings and has proved very satisfactory. The size of angles, radii of gyration, dimensions of tie plates, slenderness ratio of the flanges between tie plates, radius of gyration of column and slenderness ratio of columns give relations which are in many ways comparable to column sections which have been used in reinforced concrete building construction.

The method of fabricating these columns was in no sense unusual. The ends, although milled, did not present a true bearing as they were received in the laboratory. The steel was open hearth structural steel. Tension tests of $1\frac{1}{2} \times \frac{5}{16} \times 18$-in. specimens cut from an untested flange gave an ultimate strength of 62000 lb. per sq. in. and a yield point of 39800 lb. per sq. in.

The ultimate compressive strength of flanges composed of two $3 \times 2\frac{1}{2} \times \frac{5}{16}$-in. angles riveted back to back averaged 39 700 lb. per sq. in. for two specimens 32 inches long, and 33 200 lb. per sq. in. for two specimens 7 ft. 10 in. long.

5. *Sections of Columns.*—Table 5, page 14, gives the schedule of the columns tested. Ten steel columns were tested without concrete reinforcement; these columns are called "plain steel columns" in the schedule and in the discussion. For studying the effect of concrete in connection with the steel, the space inclosed within the outline of the structural shapes, as shown in Fig. 1, was filled with concrete. This combination of structural shapes and concrete will be termed the "core type of column". It was adopted as the principal form of test piece because it was believed wisest to obtain the larger part of the data with the section which is considered to be the effective section in design. For purposes of comparison three columns were made with 2 inches of concrete outside of the steel (see Fig. 1), and the action of this outer shell under load was studied. This combination of steel and concrete is here termed the "fireproofed type." The effect of richness of concrete upon the strength of the core type was sought by the testing of columns with 1–1–2 and 1–3–6 mixtures in addition to the 1–2–4 mixture. In six columns the steel was inclosed within a wire spiral and the space filled with concrete to the outer face of the spiral. The spirals were 14 in. in diameter, were of $\frac{1}{4}$-in. steel wire with a pitch of 2 in. and $1\frac{1}{2}$ in., respectively, the percentages of spiral reinforcement used being 0.75 and 1.0. The percentage of the structural steel section in terms of the whole column area varied, being 10.8 % for the core type, 6.1 % for the fireproofed type and 8.5 % for the spiraled columns.

TABLE 1.

TENSILE STRENGTH OF CEMENT.

Ref. No.	Age 7 Days			Age 28 Days		
	Neat	1:3 Standard Sand	1:3 Sand Used in Columns	Neat	1:3 Standard Sand	1:3 Sand Used in Columns
1	589	198	265	674	278	323
2	684	227		709	283	
3	653	240		731	319	

6. *Cement and Aggregates.*—The cement used was furnished by the Universal Portland Cement Company. Tests of samples taken at times through the season and made by B. L. Bowling, Assistant in charge of the Cement Laboratory, are given in Table 1. Sample No. 1 was taken October 14, No. 2 November 22, and No. 3 January 15. In the fineness test 98.5 per cent passed No. 50 sieve, 96.5 per cent passed No. 100 sieve, and 82.5 per cent passed No. 200 sieve.

The sand used was torpedo sand from Attica, Indiana. It was of good quality, fairly sharp, clean and well-graded. It combined with the cement used in a very satisfactory manner and gave a higher briquette test than did the same cement with standard Ottawa sand. It was from the same locality and of the same quality as the sand used in making reinforced concrete test specimens for several years at the University of Illinois.

A good quality of rather hard limestone from Kankakee, Illinois, specified to pass through a 1-in. screen and over a $\frac{1}{4}$-in. screen, was used. It is representative of the stone most used in building construction of reinforced concrete in Illinois and it was of the grade which has been used in the previous experimental work of the Laboratory. In the columns tested the failure did not appear to result from the crushing or breaking of the stone in any way.

7. *Concrete.*—Table 2 gives the proportions of the materials used in the different batches of concrete from which the columns were made. The sand and stone were first measured by loose volume. A bag of cement (95 lb.) was considered as 1 cubic foot of cement. The resulting proportions by weight are given in the table.

Men skilled in mixing concrete and making test pieces were employed in the work. The foreman and the other workmen are experienced concrete workmen; they have made the specimens for the laboratory for six seasons. The mixing was done with shovels by hand. The sand and cement were first mixed dry; the stone, which had previously been thoroughly moistened, was added and the mix then turned until of a uniform appearance. Usually the first operation included five or six turnings and the second three or more. Water was added in sufficient quantity to produce a distinctly wet mixture which would run rapidly from the shovel. The whole was then turned until thoroughly mixed.

8. *Making of Columns.*—The forms for the core type were extremely simple inasmuch as the concrete was confined to the octagon determined by the edges of the steel flanges. Four planks of the correct length were placed in a vertical position directly against the flanges and were held in position by means of yokes. For the fireproofed type octagonal forms of wood were built to give the required 2 inches of clearance over all faces. For the spiraled type circular metal forms were placed directly against the wire spiral and held in place by bands as shown in Fig. 1 of Bulletin No. 20 of the University of Illinois Engineering Experiment Station.

TABLE 2.

PROPORTIONS OF CONCRETE INGREDIENTS.

Column No.	By Volume			By Weight		
	Cement	Sand	Stone	Cement	Sand	Stone
8907	1	2	4	1	2.16	3.55
8908	1	2	4	1	2.05	3.55
8912	1	2	4	1	2.16	3.55
8913	1	2	4	1	2.05	3.55
8917	1	2	4	1	2.11	3.35
8918	1	2	4	1	1.88	3.62
8922	1	2	4	1	2.09	3.42
8923	1	2	4	1	2.04	3.54
8925	1	1	2	1	1.06	1.81
8926	1	1	2	1	1.04	1.76
8927	1	3	6	1	3.14	5.43
8928	1	3	6	1	3.02	5.30
8929	1	2	4	1	2.04	3.52
8930	1	2	4	1	2.13	3.60
8931	1	2	4	1	2.07	3.60
8933	1	2	4	1	2.08	3.61
8934	1	2	4	1	2.02	3.51
8935	1	2	4	1		
8936	1	2	4	1	2.02	3.56
8937	1	2	4	1	2.04	3.52
8938	1	2	4	1	2.08	3.67

The concrete was placed by pouring it into the forms at the top of the column a bucketful at a time. It was worked around the sides and in the center by means of a pole. As an aid in securing uniform concrete successive bucketfuls were taken from different portions of the pile. The forms were filled practically level with the top of the steel.

Previous to pouring the concrete, the steel column was placed in a vertical position on a 14 x 14 x 1-in. cast-iron bearing plate (upon which it was later tested) and the forms were placed in position. The day after pouring, after the concrete in the column had had time to shrink, the top of the column was prepared for testing by setting another bearing plate upon it, using a neat cement mortar as a bed. Care was taken to have this plate rest equally upon the four flanges and also to bear on the mortar. The layer of mortar between the plate and the steel was kept as thin as possible. A film of cement over the end of the steel was necessarily present; but in most cases it was extremely thin and in no case did this thickness reach ⅛ inch. In pouring the spiraled type it was found that the concrete did not flow into the small space at the back of the flanges between the wires of the spiral, and it was necessary to grout into this space with a sand mortar. In columns of the fireproofed type the concrete outside the flanges was found to be less dense and in some cases the surface over the flanges was more or less pitted. As has always been the practice in the preparation of concrete specimens in the laboratory, the fabrication was done seriatim, one specimen of a kind well through the series, followed later with the fabrication of the second specimen seriatim. By this means any accidental variation of cement or aggregate or of temperature conditions would be likely to have the same effect on one type of specimen as on another, whereas if all the specimens of the same kind were made at the same time the presence of such variations might be mistakenly considered to be caused by variations in type. This practice is explained at some length here, because considerable variation was found in the concrete of specimens of the same type made at different times. The dates of making specimens were as follows: On October 2, 1910, No. 8907 and 8912 were made from one batch of concrete and No. 8917 from another batch. On October 29, No. 8922 was made from one batch and No. 8925 from another batch. On November 4, No. 8927 was made. On November 8, No. 8908 and 8913 were made from the same batch, No. 8918 from a batch, and No. 8923 from another batch. November 12, No. 8926 was made and November 19, No. 8928 was made.

9. *Auxiliary Test Specimens.*—From each batch of concrete used three 6-in. cubes and one 8 x 16-in. cylinder were made from which to determine the properties of the concrete in the columns. These were stored in damp sand until a few days before the corresponding column was to be tested when they were removed to the testing laboratory and two faces prepared for the test by the addition of a thin coat of plaster of paris. It was originally intended to test all these specimens at the same date as the corresponding column, but an unexpectedly low strength was found

in the cube tests of the first specimens made. One cube from each of the later batches was then reserved for test at 90 days in order to know whether the concrete was poor or whether it merely lacked curing. It was found that these later specimens gave what may be considered to be normal strength, and hence no light was thrown on the cause of the low strength of the concrete from the first batches. The results of the cube and cylinder tests are given in Table 3. These auxiliary tests show that the strength of the concrete at the time of testing was far from uniform. The possible causes of this are discussed in a later paragraph.

TABLE 3.

COMPRESSION TESTS OF CUBES AND CYLINDERS.

Loads are given in pounds per square inch.

Corresponding Column No.	6-in. Cubes 60 days	6-in. Cubes 90 days	8 x 16-in. Cylinders 60 days
8907	1790		1350
8908	2430	2650	1490
8912	1790		1350
8913	2430	2650	1490
8917	1420		1140
8918	2150	2600	1260
8922	1320		970
8923	1970	2640	1150
8825	2970		2420
8926	3280	4000	2520
8927	1320	1800	700
8928	1440	1740	660
8929	1760	2580	1370
8930	2020	2740	1330
8931	1670	2760	1280
8933	1440	2000	1160
8934	2070	2350	1340
8935	2270	2720	1330
8936	1780	2180	1110
8937	1620	2900	1460
8938	1980	2580	1120

10. *Storage and Handling.*—The columns were stored in the room where they were made. Forms were removed at the end of a week and from that time on the columns were occasionally sprinkled. The records show that the room temperature varied from 60° to 70° Fahrenheit, but it seems probable that a larger variation may have occurred in different parts of the building. It is also probable that the later columns dried out much more and attained a higher percentage of their final concrete strength than did the cubes at the same age.

Before removing the columns to the testing laboratory the bearing plates at top and bottom were connected by rods to prevent displacement, and these rods were not removed until the column was in its final position in the testing machine.

11. *Method of Measuring Deformations.*—The extensometer used and the method of attachment are shown in Fig. 2. For the determination of deformations in the steel, holes were tapped in the flanges at the lower point to receive the shaft of a wire-wound dial extensometer and at the upper point to receive a bolt; from this bolt a wire was suspended, wrapped once around the drum of the extensometer and weighted with a nut at the end. The deformation occurring in the gauge length between the bolt and the lower shaft was registered by the movement of the pointer of the dial. Measurements were made on each of the four flanges, four instruments being used for the purpose. Where measurements were desired on concrete faces, specially prepared plugs were inserted in the column during the pouring and these were tapped to receive the bolts and the shafts of additional extensometers. The wire suspended upon the upper bolts was in general 1 inch from the face of the column, and the accuracy of the observations depends upon the conservation of the plain section in the column as a whole. The results obtained indicated that this condition was not fully satisfied in the case of plain steel columns. An arrangement of instruments which gave the deformations at the center of gravity of the flanges direct was used in the test of flanges and on column No. 8914. The result indicated that the error in the first arrangement of instruments was not great. The gauge lengths used varied with the lengths of the specimens. The dials could be read to an indicated movement of 0.0002 inch. The gauge

TABLE 4.

GAUGE LENGTHS AND PRECISION OF MEASUREMENTS.

Length of Column	Gauge Length	Least Unit-deformation Measurable
2 ft.-0 in.	10 in.	.00002
4 ft.-8 in.	40 in.*	.000005
10 ft.-0 in.	100 in.	.000002
15 ft.-4 in.	150 in.	.000001
19 ft.-4 in.	200 in.	.000001

*Average.

length and the least unit-deformation obtainable for the different lengths of the columns are given in Table 4.

In the case of the plain steel columns 10 feet or more in length, in addition to the four measurements of deformation taken over the gauge lengths noted in Table 4, six other measurements of deformation were taken over gauge lengths about one-third as long. These shorter gauge lengths were located at different portions of the columns in an effort to detect local bending. The measurements did not show that material bending occurred at loads below the maximum load, and if such bending occurred, it was confined to shorter distances.

12. *Method of Testing.*—The columns were tested in the Riehle vertical 600 000-lb. screw-power testing machine in the Laboratory of Applied Mechanics of the University of Illinois. It was appreciated in advance that the full strength of the spiraled columns could not be developed in the 600 000-lb. testing machine, but it was thought that there would be a sufficient indication of their action to determine the critical yield-point. However, after testing them it was concluded that a determination of the action of such columns at higher loads would be of value, and further tests of four of the columns were made in the Riehle 800 000-lb. testing machine at Lehigh University. In the case of the plain steel columns a specimen was placed with its lower end bearing directly on the weighing table of the testing machine and centered with respect to the screws. The compression head of the machine was then lowered until the suspended spherical bearing block rested on top of the column. This block was then centered on the column. Although the ends of the columns were milled in the shops, in some cases it was necessary to file the ends to secure a satisfactory bearing and in some cases to use carefully prepared steel shims. In the case of the concreted columns the bearing plates of the columns rested directly on the weighing table. When the column was in its final position in the machine, the rods connecting the end bearing plates, used while transporting the column to the testing room, were removed and the spherical bearing block was lowered into position as noted for the plain steel columns. An initial reading of the extensometer was taken with no load on the column except the weight of the spherical bearing block. The compression head of the testing machine was then brought to bear on the bearing block and was run down at the slowest speed (0.05 inch per minute)

until a load of 25 000 lb. was registered. The machine was stopped at this load and after an interval of 30 to 45 seconds the extensometers were read. In like manner the load was increased by increments of 25 000 lb. at the speed of 0.05 inch per minute, with readings of the extensometers between applications of load, until the maximum strength of the column was passed or the capacity of the testing machine was reached. With the first five columns tested (No. 8905, 8906, 8910, 8915, 8920) no effort was made to restrain any movement of the spherical bearing block. In the remaining tests, when a load of 50 000 lb. had been applied, special wedges were inserted and adjusted in the bearing block to restrain it from a rolling movement. In the tests of many of the columns the operation of the machine was continued after the maximum load was passed, to determine the effect of a further application of load and a further shortening of the column. This was done in order that the critical section might be definitely determined and the nature of the final failure observed.

Three conditions mentioned incidentally above merit more complete discussion:

(a) It is evident that the speed at which the testing machine is run is of importance and that a column will carry a higher maximum load if the load be applied rapidly than it will under a slow application. In actual use the load is not momentarily applied but it is a dead weight and follows at once any shortening or other movement of the column. In testing, the load should be held a sufficient time to ensure that the material has attained its full deformation. In the tests herein described the slowest speed of the machine (0.05 in. per min.) was used in all cases. The instruments were read starting about 30 seconds after the indicated load had been momentarily applied.

(b) The condition of the bearing surfaces at the ends of the column has an important effect upon the load which the column will carry. In a building a column receives its load from story to story and especial attention is given to the bearing arrangement at its base, but in a testing machine the load is applied directly upon the end section. If the load is to be uniformly distributed, the bearings of the ends must be perfect. In these tests care was taken to get a fairly good bearing, but it should be noted that the load was not always uniformly distributed over the section and that some of the flanges were more highly stressed than others. The records of observed deformations of the various gauge lengths in the different flanges show this effect. When

TABLE 5.

DATA OF TEST COLUMNS.

Column No.	Description	Length ft. in.	$\frac{l}{r}$	Area of Gross Section sq. in.	Concrete Mixture	Age of Test days
8902	Plain Steel	2-0	6.1	13		
8905	Plain Steel	4-8	14.4	13		
8906	Plain Steel	4-8	14.4	13		
8907	Core Type	4-8		120	1-2-4	60
8908	Core Type	4-8		120	1-2-4	59
8910	Plain Steel	10-0	30.8	13		
8911	Plain Steel	10-0	30.8	13		
8912	Core Type	10-0		120	1-2-4	60
8913	Core Type	10-0		120	1-2-4	62
8914	Plain Steel	10-0	30.8	13		
8915	Plain Steel	15-4	47.2	13		
8916	Plain Steel	15-4	47.2	13		
8917	Core Type	15-4		120	1-2-4	61
8918	Core Type	15-4		120	1-2-4	59
8920	Plain Steel	19-4	59.5	13		
8921	Plain Steel	19-4	59.5	13		
8922	Core Type	19-4		120	1-2-4	60
8923	Core Type	19-4		120	1-2-4	60
8925	Core Type	10-0		120	1-1-2	61
8926	Core Type	10-0		120	1-1-2	60
8927	Core Type	10-0		120	1-3-6	59
8928	Core Type	10-0		120	1-3-6	60
8929	Fireproofed	10-0		213	1-2-4	60
8930	Fireproofed	10-0		213	1-2-4	60
8931	Fireproofed	10-0		213	1-2-4	60
8933	Spiraled*	10-0		153	1-2-4	60
8934	Spiraled*	10-0		153	1-2-4	59
8935	Spiraled*	10-0		153	1-2-4	59
8936	Spiraled*	10-0		153	1-2-4	60
8937	Spiraled†	10-0		153	1-2-4	60
8938	Spiraled†	10-0		153	1-2-4	59

*0.75 % of spiral reinforcement.
† 1.0 % of spiral reinforcement.

the load is unevenly distributed over the section the tendency towards bending is increased. The effect of poor end conditions is more serious in short columns than in long ones; in the 19 ft. 4-in. columns its effect is probably negligible. In studying the effect of length on column strength the effect of end conditions must be borne in mind. Columns No. 8902 and 8914 had the best end conditions of any tested. In the 2-ft. column (No. 8902) a test was found to be impracticable without carefully turning the ends in a lathe.

TABLE 6.

Maximum Loads Carried by Columns and by Steel and Concrete.

Column No.	Total Load in Pounds			Pounds per square inch of Section	
	Column Load	Load Considered as Carried by Steel	Load Considered as Carried by Concrete	Steel	Concrete
8902	487 300				
8905	440 200			33 700	
8906	449 000			34 700	
8907	577 000	444 600	132 400	34 200	1240
8908	602 000	444 600	157 400	34 200	1470
8910	410 400			31 600	
8911	425 600			32 700	
8912	510 000	418 000	92 000	32 150	860
8913	584 700	418 000	166 700	32 150	1560
8914	424 000			32 600	
8915	368 000			28 300	
8916	376 000			28 900	
8917	468 500	372 000	96 500	28 600	900
8918	532 200	372 000	160 200	28 600	1500
8920	374 200			28 800	
8921	345 000			26 500	
8922	491 400	359 600	131 800	27 650	1230
8923	495 500	359 600	135 900	27 650	1270
8925	636 000	418 000	218 000	32 150	2040
8926	655 000	418 000	237 000	32 150	2210
8927	516 000	418 000	98 000	32 150	920
8928	530 500	418 000	112 500	32 150	1050
8929	600 000*				
8930	630 700	418 000	212 700	32 150	1060
8931	635 700	418 000	217 700	32 150	1090
8933	600 000	Maximum load applied five times; not broken.			
8934	600 000	Not broken.			
8935	600 000	Maximum load applied three times; not broken.			
8936	625 000	Not broken.			
8937	600 000	Maximum load applied three times; not broken.			
8938	600 000	Not broken.			
8934	856 000	Second test; near ultimate.			
8935	830 000	Second test; near ultimate.			
8936	600 000	Second test; not broken.			
8937	830 000	Second test; not broken.			
8938	827 000	Second test; not broken.			
8937	714 000	Third test with spiral and outside concrete removed.			

*Not broken.

(c) The conditions of the end restraint will affect the curve taken by a column. The lower end, bearing on the unyielding weighing table of the testing machine, is quite firmly restrained until serious bending occurs in the column. The upper end is loaded through a spherical bearing block in order to secure adjustment with the compression head of the testing machine. With this arrangement, this adjustable bearing block may finally move on itself as the tendency to bend in the column overcomes the friction between the surfaces of the spherical bearing block, and thus only partial restraint will exist. In the first five plain steel columns tested (No. 8905, 8906, 8910, 8915, 8920) no effort was made to prevent this movement, and the column was in the condition of one end fixed and one end partly restrained. Neither the fixedness nor the freedom of the ends can be considered as in any sense absolute; they must be taken as relative terms. In all the column tests after the five noted above, special wedges and angle blocks were driven under the upper head (at a load of 50 000 lb.), the

TABLE 7.

LOADS CARRIED AT VARIOUS UNIT-DEFORMATIONS.

Loads are given in thousands of pounds for the unit-deformation given in the column caption and in the last column for the maximum load applied.

Column No.	.0002	.0004	.0006	.0008	.0010	.0012	.0014	.0016	.0018	.0020	.0030	.0040	Max. Load
8905	83	160	229	290	341	379	405	422					440
8906	83	161	235	300	355	396	421						449
8907	115	206	286	360	419	467	515	557					577
8908	120	216	311	390	465	529	565	592					602
8910	82	158	226	286	338	372	393						410
8911	82	155	221	281	335	377	402						426
8912	121	207	286	355	414	455							510
8913	123	224	319	390	461	516	548						585
8914	80	160	233	300	355	390	410	419	423				424
8915	84	154	216	271	319	352							368
8916	86	159	221	276	321	353	373						376
8617	109	193	268	336	393	432							468
8918	118	208	288	363	429	482	509						532
8920	80	150	211	262	308	349							374
8921	84	156	214	265	310								345
8922	115	199	269	331	387								491
8923	114	206	285	358	427	477							495
8925	132	240	336	426	505	568	604	624					636
8926	119	226	320	405	486	553	594	619	635	647			655
8927	107	190	265	334	397	447	480	500					516
8928	105	187	264	335	399	452	486	500					530
8929	141	257	357	452	534	594							600‡
8930	117	218	310	398	474	536	581	611					631
8931	144	254	347	424	492	546	585	610	626				636
8933	126	231	322	409	489	550	588						600‡
8934	134	235	321	394	459	511	545	568	588				600‡
8935	136	242	338	430	516	578	600	600					600‡
8936	129	234	324	406	483	541	579						625‡
8937	135	242	338	427	508	567	600						600‡
8938	124	230	326	416	491	543	581						600‡
8934*	112	220	325	430	525	625	685	715	737	755	808	835	856‡
8935*	100	200	300	410	525	640	690	720	743	761	815		830‡
8936*	110	200	290	375	460	540	592						600‡
8937*	96	194	295	404	520	628	680	710	733	753	815		830‡
8938*	110	218	322	430	540	653	705	730	750	768	820		827‡
8937†	111	225	340	453	567	656	700	712	714				714

*Second test; all second tests made at Lehigh University except column No. 8936.
†Third test; made at Lehigh University.
‡The column did not fail under maximum load applied.

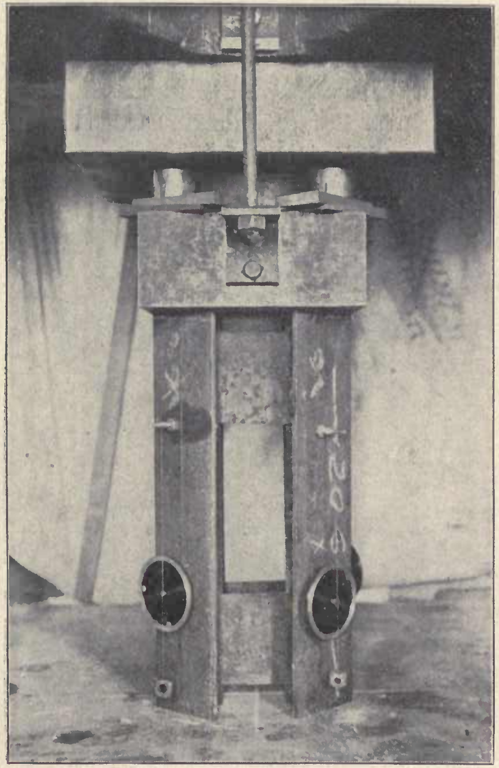

FIG. 2. VIEW OF BEARING BLOCK AND ATTACHMENT OF INSTRUMENTS.

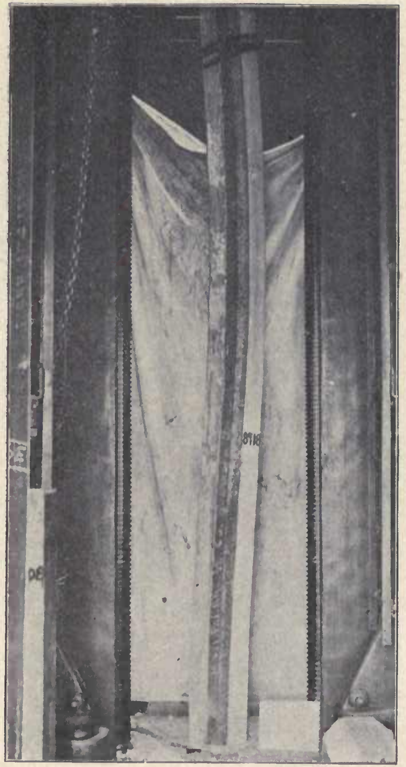

FIG. 3. VIEW SHOWING CORE TYPE OF COLUMN BEYOND THE MAXIMUM LOAD.

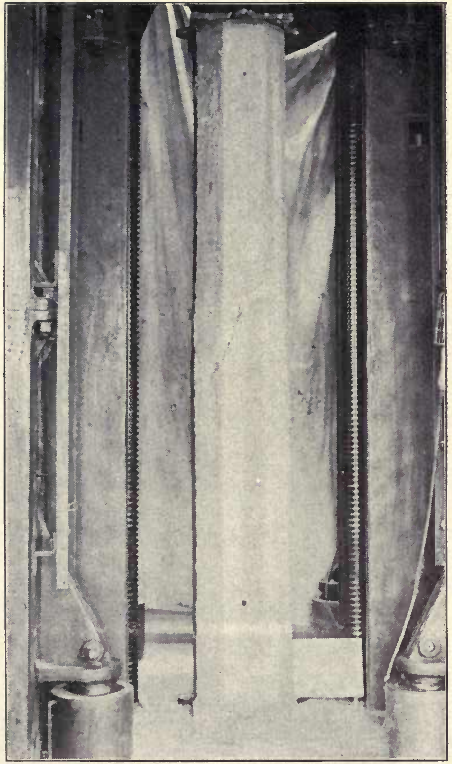

Fig. 4. View Showing Fireproofed Column beyond the Maximum Load.

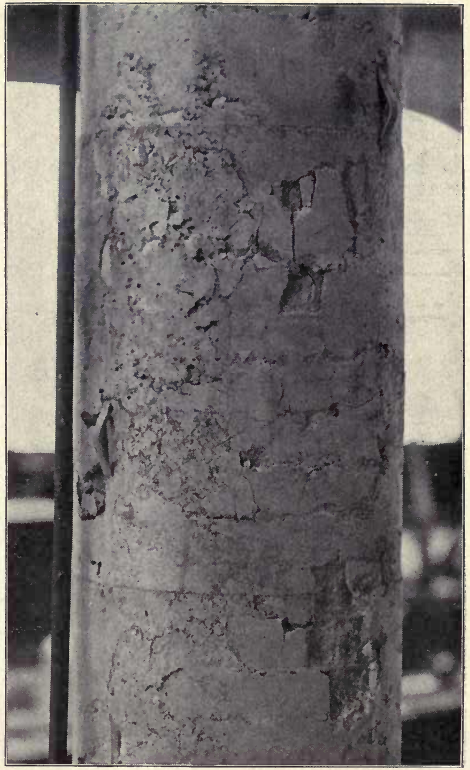

Fig. 5. View Showing Spiraled Column after Test.

spherical bearing block having had opportunity to adjust the bearing satisfactorily by this time and no appreciable tendency to bend having yet developed in the column. In this manner the spherical bearing block was restrained from further motion and became in effect as rigid a loading block as the base. Fig. 2, facing p. 16, shows the arrangement used in the later tests. That the apparatus accomplished the desired purpose is evidenced by the fact that failure in bending occurred almost invariably at or near the center of the column. It may be noted in this connection that the conditions of restraint in the testing machine are probably not as near fixed-end conditions as are those of a reinforced concrete building.

III. Experimental Data and Discussion.

13. *Data.*—In Table 6, page 15, are given the maximum loads carried by the test columns. The loads carried at various unit-deformations (interpolated between readings) are given in Table 7. The load-deformation diagrams are given in Fig. 6, 7, 11, 15, 18, 19, and 20. Complete data for all tests are on file in the Laboratory of Applied Mechanics of the University of Illinois, but as there were two hundred or more readings of deformations for most of the tests, only the summarized data are given in this bulletin.

The discussion of the tests will be made under the following heads: A. Plain Steel Columns—Effect of Length; B. Core Type—Effect of Length; C. Core Type—Value of Concrete; D. Fireproofed Columns; E. Spiraled Columns; F. Summary.

A. Plain Steel Columns—Effect of Length.

14. *Phenomena of the Tests.*—The plain steel columns showed test phenomena which were consistently uniform. At loads of from 225 000 to 250 000 pounds cracking sounds were heard and these continued intermittently throughout the remainder of the test. The time required to add the 25 000-lb. increment of load gradually became longer with the testing machine running at a uniform speed, and at the maximum load the weighing beam floated within a range of 1000 lb. for a period of 10 or 15 minutes. At maximum load no bending was visible to the eye except in the longer columns and very little was shown by the deformation readings. After the maximum load was reached and the machine head was run down to complete the failure of the column

(generally at a faster speed), bending developed very gradually. In general this bending was fairly symmetrical about the middle of the length.

15. *Stress-deformation Relations.*—The load-deformation diagrams for the plain steel columns are given in Fig. 6. It is seen that the load-deformation curves bend at low loads. This may be due partly to imperfect end bearings of the steel shapes and to the arrangement of instruments which presupposed the conservation of a plane section in the column during the test. Column No. 8914 whose ends were dressed quite carefully to a fairly true surface and for which the extensometer was arranged to avoid any effect of bending shows a straight line up to 175 000

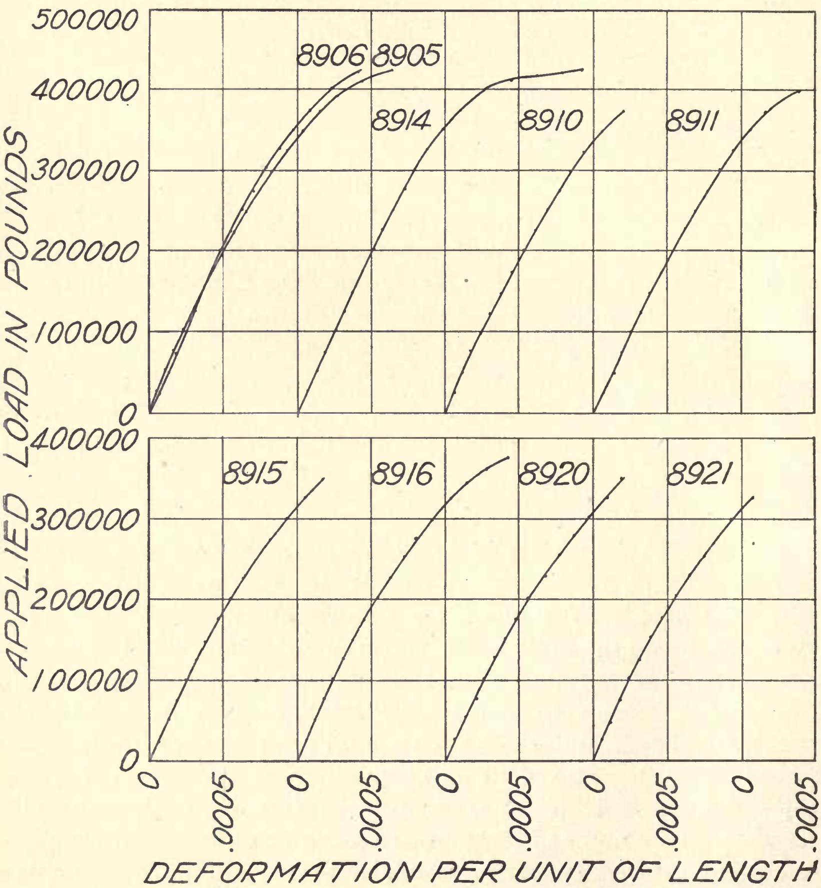

FIG. 6. LOAD-DEFORMATION DIAGRAMS FOR PLAIN STEEL COLUMNS.

lb. load and also an increase in stiffness over the other two columns of the same length. Even in this case a marked bending is present in the diagram well below the maximum load.

In Table 8, page 22, are given the secant moduli of elasticity, calculated from the curves, for unit-deformations of .0004, .0007, .0010, and for a point near the ultimate load. From these computations, it would appear that the modulus of elasticity for low deformations may be considered as lying between 30 000 000 and 31 500 000 lb. per sq. in. In Fig. 7 the average load-deformation curves for the several lengths of column are also

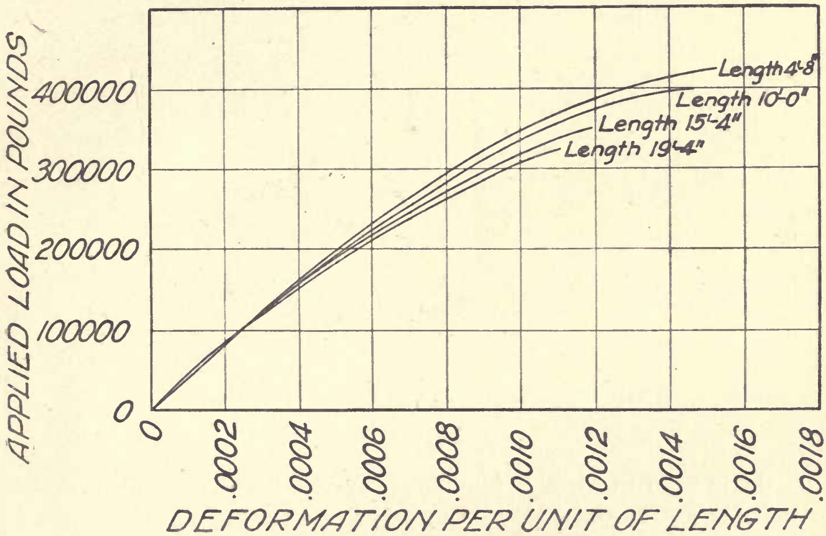

FIG. 7. AVERAGE DEFORMATION DIAGRAMS FOR PLAIN STEEL COLUMNS OF VARIOUS LENGTHS.

shown. It is apparent that the modulus of elasticity of the column as a whole decreases as the length of the column increases. It is also seen that the unit-deformation in the column as a whole at the maximum load becomes less as the length of the column increases. This is an indication of localized high stresses in the column.

16. *Relation of Strength to Length.*—In Table 6, page 15, are given the maximum loads carried by the plain steel columns of the several lengths.

Tests which have been made in the past on relatively small columns indicate that for columns having a ratio of length (l) to radius of gyration (r) less than, say, 100, the results may be expected to fall along a line slightly inclined to the $\frac{l}{r}$ axis. In arriving at an expression for the effect of length upon strength it may add to our clearness of perception to trace the development of the effect of length upon strength at various stages of the test. In

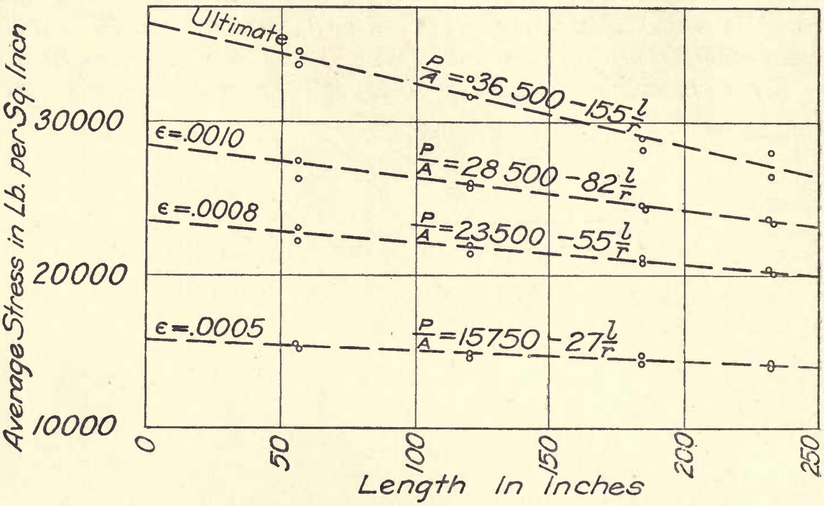

FIG. 8. LOAD-LENGTH DIAGRAMS FOR PLAIN STEEL COLUMNS.

Fig. 8 the results for the four lengths of column are plotted for unit-deformations of .0005, .0008, .0010, and for ultimate load. The four equations give the relation between $\frac{l}{r}$ and unit-stress at these deformations. The equations show that the effect of length upon stress is small at low deformations and is relatively much higher at the higher deformations. In a similar manner results were plotted for other unit-deformations from .0003 to .0012, and from these data Fig. 9 has been prepared. In this figure values of f and k for the column formula $\frac{P}{A}=f-k\frac{l}{r}$ are given for a range of values of the unit-deformation. The lower curve also shows the increasing effect of the slenderness ratio on the strength as the test progresses. If we should produce the tangent back to the horizontal axis the intersection is at .0004, and it is not far from the facts to say that up to a unit-deformation of .0004 the slenderness ratio has no effect and that beyond

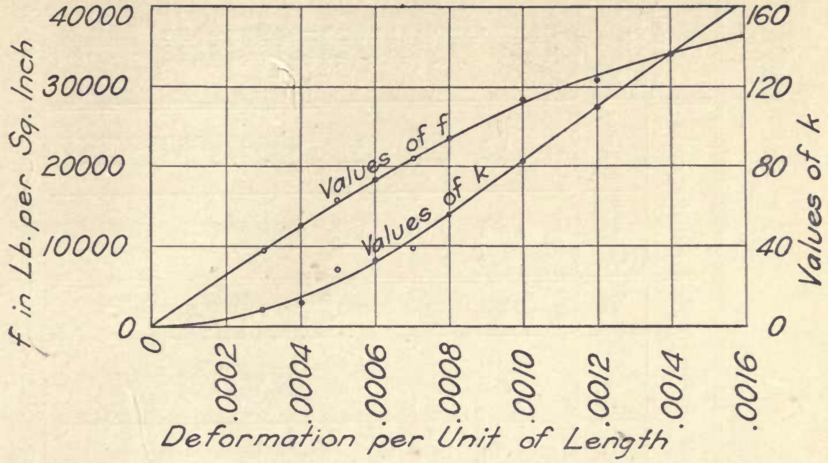

FIG. 9. VALUE OF TERMS IN STRAIGHT-LINE FORMULA FOR PLAIN STEEL COLUMNS. Column formula: $\frac{P}{A}=f-k\frac{l}{r}$

this deformation it increases in a constant ratio to the increase in deformation. From these diagrams it appears that a straight line will represent the results very satisfactorily for any given unit-deformation.

For the ultimate load, the nature of the strength-length relation is somewhat affected by the fact that the ultimate general unit-deformation of a column at its maximum load is smaller for the greater lengths of column, as is shown in Fig. 7. While for deformations less than the ultimate the straight-line equation

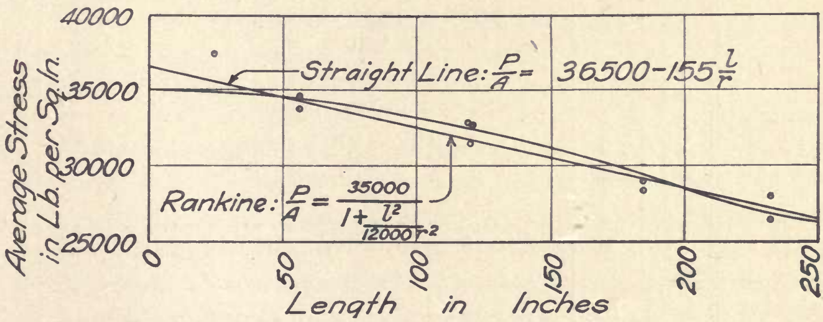

FIG. 10. COLUMN FORMULAS FOR MAXIMUM LOAD.

TABLE 8.

Moduli of Elasticity.

The values of the moduli are given in millions of pounds per square inch.

Column No.	Secant Modulus at Unit-deformation or .0004			Secant Modulus at Unit-deformation of .0007			Secant Modulus at Unit-deformation of .0010			Secant Modulus near Ultimate Strength			Initial Modulus		Last Unit-deformation*
	Steel	Concrete	Ratio n	Steel	Concrete	Ratio n	Steel	Concrete	Ratio n	Steel	Concrete	Ratio n	Column Concrete	Cylinder	
8902	34.2			32.5			30.6			26.5					.0017
8905	31.3			28.7			26.4			22.8					.0016
8906	31.1			29.6			27.3			25.1					.0014
8907	31.2	1.07	29	29.1	0.80	35	26.8	0.70	38	23.8	0.64	37	2.00	2.00	.0018
8908	31.2	1.33	23	29.1	1.20	24	26.8	1.11	24	23.8	1.03	23	1.67	2.08	.0015
8910	30.6			28.3			26.0			20.3					.0015
8911	29.8			27.8			25.7			22.3					.0014
8912	30.2	1.21	25	28.0	0.92	30	25.9	0.73	35	21.3	0.56	37	2.00	2.00	.0017
8913	30.2	1.46	21	28.0	1.32	21	25.9	1.19	22	21.3	1.03	21	2.30	2.08	.0014
8914	31.3			29.4			27.4			24.2					.0013
8915	29.8			26.9			24.4			23.0					.0012
8916	30.8			27.5			24.6			20.5					.0014
8917	30.3	0.86	35	27.2	0.75	36	24.5	0.67	36	21.8	0.63	34	1.46	1.85	.0012
8918	30.3	0.98	31	27.2	1.07	25	24.5	1.03	24	21.8	0.90	24	1.79	1.55	.0013
8920	28.9			26.0			23.5			22.4					.0012
8921	30.0			26.5			23.7			23.1					.0011
8922	29.4	0.94	31	26.3	0.84	31	23.6	0.74	32	22.8	0.74	31	1.81	1.30	.0010
8923	29.4	1.23	24	26.3	1.13	23	23.6	1.11	21	22.8	1.05	22	1.87	1.33	.0012
8925	30.2	1.98	15	28.0	1.72	16	25.9	1.61	16	21 3	1.40	15	2.33	2.84	.0016
8926	30.2	1.63	18	28.0	1.47	19	25.9	1.43	18	21.3	1.36	16	1.85	3.15	.0020
8927	30.2	0.79	38	28.0	0.63	44	25.9	0.52	45	21.3	0.58	37	1.30	1.25	.0016
8928	30.2	0.74	40	28.0	0.63	44	25.9	0.60	43	21.3	0.60	35	1.06	1.25	.0015
8929	30.2	1.27	24	28.0	1.09	26	25.9	1.00	26	21.3	0.93	23	1.60	2.00	.0012
8930	30.2	0.76	40	28.0	0.73	38	25.9	0.69	37	21.3	0.66	32	1.00	1.67	.0018
8931	30.2	1.22	25	28.0	0.95	29	25.9	0.78	33	21.3	0.67	32	1.85	2.00	.0018
8933	30.2	1.34	23	28.0	1.16	24	25.9	1.11	23				1.67	1.37	
8934	30.2	1.41	21	28.0	1.06	26	25.9	0.88	29				2.00	1.25	
8935	30.2	1.52	20	28.0	1.34	21	25.9	1.29	20				2.00	2.00	
8936	30.2	1.36	22	28.0	1.14	25	25.9	1.06	24				1.70	2.00	
8937	30.2	1.55	19	28.0	1.33	21	25.9	1.22	21				2.00	2 00	
8938	30.2	1.30	23	28.0	1.18	24	25.9	1.11	23				1.80	2.00	

* This column gives the unit-deformation observed immediately before the removal of the extensometers from the test columns, which was at loads varying from 80 % to 100 % of maximum load.

alone appears to apply, at the ultimate it is possible to construct an equation of the Rankine type which will represent the result quite closely. In Fig. 10 such an equation and a straight-line equation are plotted. The equations are as follows:

$$\text{Straight-line: } \frac{P}{A} = 36\,500 - 155\frac{l}{r}$$

$$\text{Rankine type: } \frac{P}{A} = \frac{35\,000}{1+\dfrac{l^2}{12000\,r^2}}$$

It appears that the straight-line equation represents the test results as closely as the Rankine equation. The ultimate load for $\frac{l}{r} = 0$ is higher for the straight-line equation than by the Rankine equation, and both are lower than the stress of 37 500 lb. per sq. in. carried by the test column two feet long and the stress of 39 700 lb. per sq. in. carried by the compression test pieces taken from the flanges. After making a study of the data it is believed that the straight-line equation given above may be considered best to represent the effect of length upon ultimate load for the plain steel columns tested.

B. Core Type—Effect of Length.

17. *Phenomena of the Tests.*—The columns in which the core only was filled with concrete acted in much the same way as the plain steel columns. The concrete was somewhat restrained by the structural shapes, but the capacity for carrying an increasing load accompanied by the development of very high deformations which has been found in hooped concrete columns was not present. The columns exhibited much toughness and gave slow failures.

All the columns were tested with the upper bearing block restrained from motion, and in all cases the bending was symmetrical about the center or it occurred below the center. Very little bending was apparent at the maximum load. The bending shown in the view in Fig. 3, facing p. 16, occurred some time after the maximum strength of the column had been developed. In the final failure of the column, at deformations well beyond the maximum load, the crushing of the concrete was frequently more marked at the top of the column than elsewhere, due probably to the smaller density of the concrete at this place. In the columns 4 ft. 8 in. long practically no bending occurred, and the columns failed by general crushing which was more marked over the upper half. The columns 10 feet long, in the continuation of the test beyond the maximum load, failed finally in bending, No 8912 bending about the center and No. 8913 bending sharply near the base. The columns 15 feet 4 in. long and 19 feet 4 in. long, after passing the maximum load, bent symmetrically about the center. In the latter part of the test, the concrete crushed on one side of

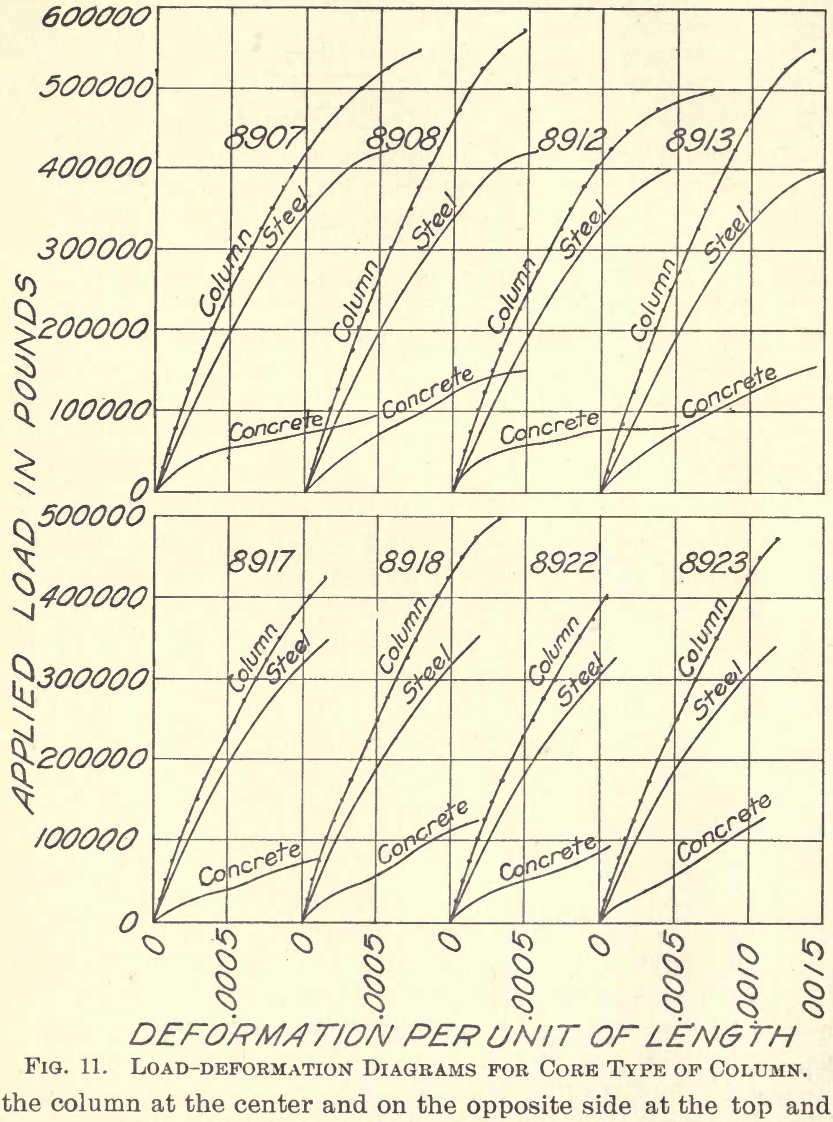

FIG. 11. LOAD-DEFORMATION DIAGRAMS FOR CORE TYPE OF COLUMN.

the column at the center and on the opposite side at the top and bottom. The varying density of the concrete caused by the obstruction to settlement and shrinkage in setting offered by the steel was shown in the final crushing. In all cases the crushing was more marked at points at or immediately above the tie plates in the steel columns than at points between these plates. Apparently the concrete was less dense at the first-named places. As noted above, and as was to be expected, the concrete at the top of the column was weaker and less dense than that lower down.

18. *Stress-deformation Relations.*—Fig. 11, page 24, gives load-deformation diagrams for these columns. It was evident from the test results that in these columns the conservation of plane section was well maintained during the test, and this condition has been assumed in the interpretation of the data. The load-deformation curves are similar to those of the plain steel columns. In the diagrams in Fig. 11 the average load-deformation curve for the two plain steel columns of the same length has also been drawn.

19. *Effect of Test Conditions.*—The thickness of the mortar cushion between the end of the steel column at the top and the bearing plate may have exerted an influence on the strength of the columns, but it is difficult to arrive at any estimate of the amount of this. Where the joint was extremely thin, as was generally the case, its effect was undoubtedly negligible, but in one case where this joint was nearly $\frac{1}{8}$ inch thick it may have exerted an appreciable influence on the strength developed by the column.

20. *Effect of Length.*—The maximum loads carried by the columns are given in Table 6, page 15.

In the discussion of the effect of length of column the slenderness function may be expressed in terms of $\frac{l}{d}$ where l is the length of the column and d is the short diameter of the column section. In a following paragraph the effect of the length will be expressed in terms of $\frac{l}{r}$, where r is the radius of gyration of the steel section.

In Fig. 12 are plotted the loads for eight columns for unit-deformations of .0005, .0008, .0010 and also for ultimate load. Owing to variations in the concrete strength these points can not be expected to show as close agreement as did the plain steel columns. It may be noted that for each length of column the specimen showing greater strength was made on Nov. 8 and the weaker one on an earlier date, the variation in the strength agreeing with variation of strength of cubes and cylinders, as discussed elsewhere. The straight lines on this diagram express fairly well the relation between $\frac{l}{d}$ and the average unit-stress over the cross-section of the column for several unit-deformations and for the maximum loads. P represents the load on the column and A the area of the cross-section of both steel and concrete. The equation for the maximum load is seen to be $\frac{P}{A} = 5150 - 52\frac{l}{d}$.

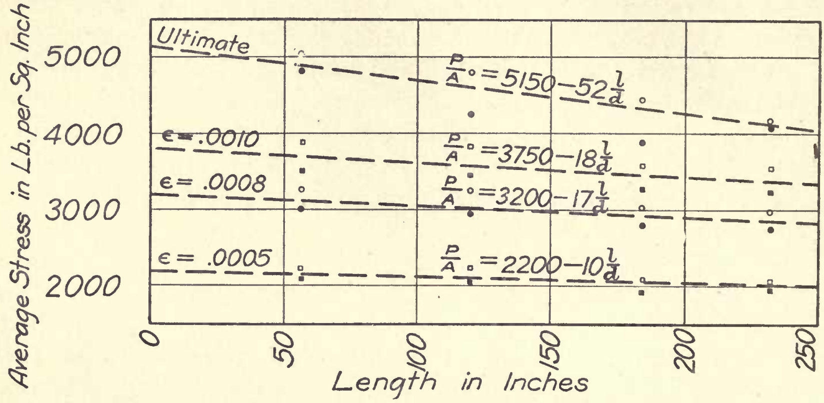

FIG. 12. LOAD-LENGTH DIAGRAM FOR CORE TYPE OF COLUMN.

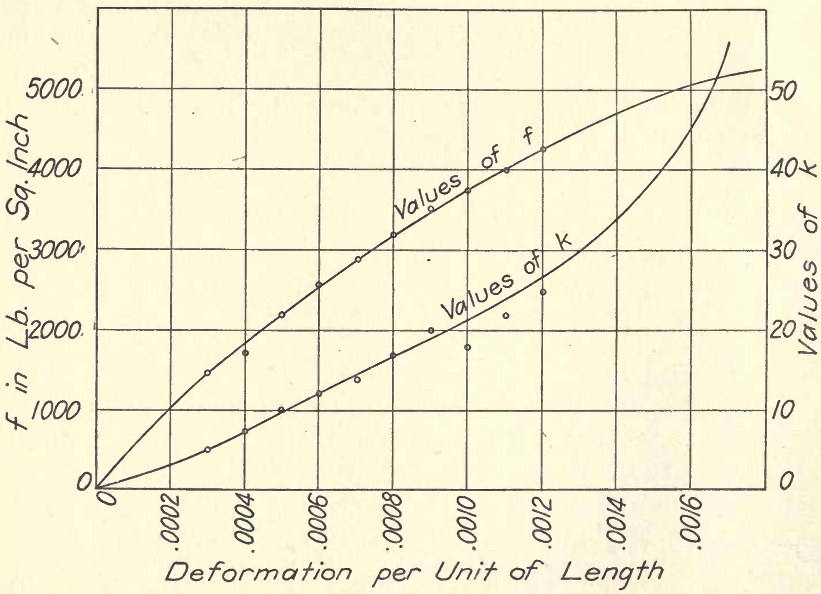

FIG. 13. VALUE OF TERMS IN STRAIGHT-LINE FORMULA FOR CORE TYPE OF COLUMN. Column formula: $\frac{P}{A} = f - k\frac{l}{d}$

Fig. 13 gives values of f, the first term of the second member in the straight-line column formula, and of k, the coefficient of $\frac{l}{d}$, for the range of unit-deformations.

21. *Comparison with Plain Steel Columns.*—In order to obtain a comparison of the effect of slenderness in columns of the core type and in plain steel columns, Fig. 14 has been prepared. In this diagram, for the purpose of comparison, the total load has

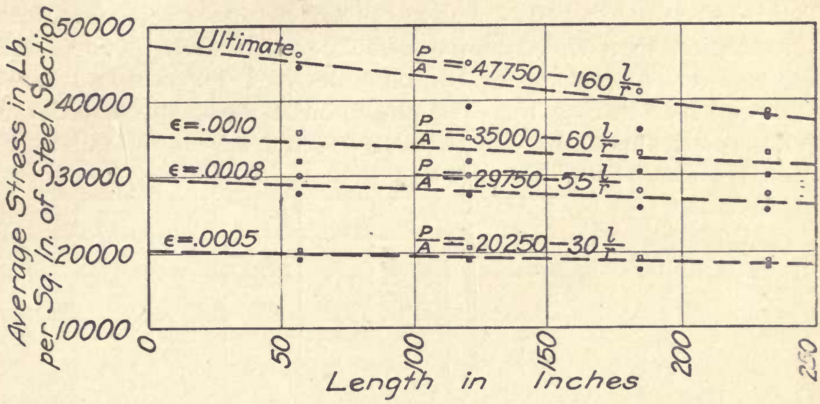

FIG. 14. LOAD-LENGTH DIAGRAM IN TERMS OF THE STEEL SECTION FOR CORE TYPE OF COLUMN.

been considered to be carried by the steel alone. A is the area of the cross-section of the steel and r is its radius of gyration. By comparing with Fig. 8 it is seen that the coefficients of $\frac{l}{r}$ agree very closely with the coefficients of $\frac{l}{r}$ found in the tests of plain steel columns at the same compression deformations. Thus at ultimate load the coefficient of $\frac{l}{r}$ is 155 for the plain steel columns and 160 for the reinforced steel columns. This would indicate that in the core type of column within the limits of length tested, the effect of length upon strength of column is a function of the slenderness ratio of the steel itself and is almost independent of the slenderness ratio of the concrete. This conclusion is in accord with results of tests of long concrete columns made in the Laboratory of Applied Mechanics, plain concrete columns 20 diameters long giving nearly as great strength as columns 10 diameters long. The discussion of the amount of stress taken by the concrete is given in a later paragraph.

C. Core Type—Value of Concrete.

22. *Test Phenomena and Stress-deformation Relations for Various Mixtures.*—All the columns of the core type showed substantially the same test phenomena for the different mixtures of concrete, a fact which may be accounted for by the presence of sufficient steel to make the concrete effect the smaller element. As in none of these columns did the total load taken by the concrete exceed one-half of that taken by the steel, it would be expected that the steel would govern the general behavior of the column under test. The 1–1–2 columns were found to sustain a greater ultimate unit-deformation than the columns of leaner mixtures. The load-deformation diagrams for the columns of 1–1–2 mix and 1–3–6 mix are given in Fig. 15. The close similarity of the load-deformation diagrams and those of the 1–2–4 mix of the same length, shown in Fig. 11 and 15, may be noted.

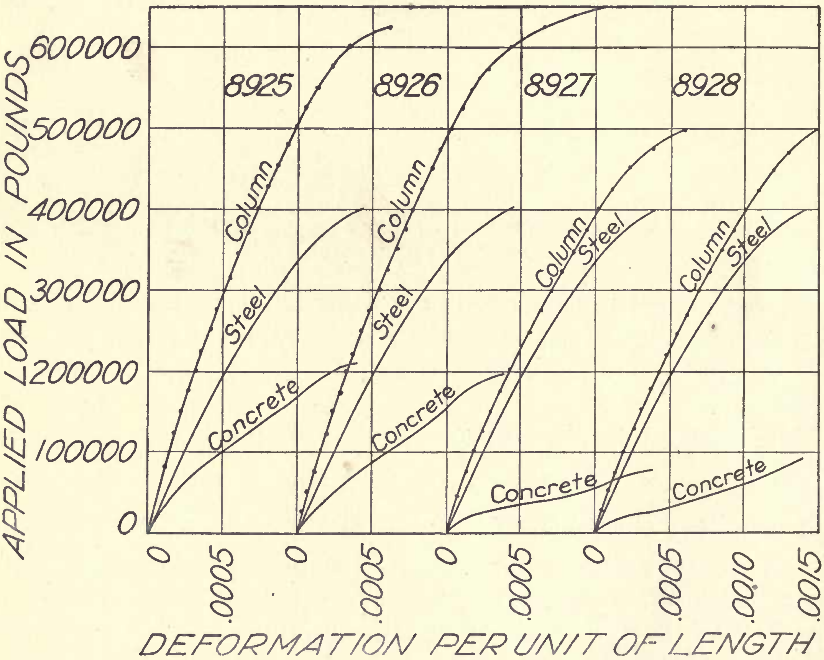

Fig. 15. Load-deformation Diagrams for Columns with Lean and Rich Concrete.

23. *Basis for Determining Load Taken by Concrete.*—In studying the strengthening effect of the concrete on the steel column it is necessary to decide upon some basis of division of the total load into the part considered to be carried by the steel and the part carried by the concrete of the columns. The load-deforma-

tion diagrams given for the tests of plain steel columns (Fig. 6, page 18) show that the compression deformations of these columns vary considerably from a straight line, which would be the form if a constant modulus of elasticity were assumed for the column. A straight-line relation does not take into account the adjustments in the bearing of the rivets and in the loading of the tie plates or lacing bars, the local flexure of the flanges, and the later changes in the stiffness of the material itself. Of course, it can not be told to what extent the presence of the core concrete in columns will overcome the agencies which cause curvature in a plain steel column before parts of the steel reach the yield point. It seems probable that there will be some such action and that the concreting of the column will add stiffness to the steel section itself. However, for the purpose of the discussion it seems best to consider that the steel section in any concreted column carried the same amount of load as was carried by a plain steel column of the same length. For this purpose the average load-deformation diagram for the plain steel columns of the same length are plotted on the load-deformation diagrams of the tests of columns of core type. (See Fig. 11 and 15). For any given unit-deformation we may then obtain the amount of load considered to be carried by the concrete by subtracting the load carried by the plain steel columns at this deformation from the total load carried by the concreted column. The results have been plotted on the line marked "Concrete". It seems possible that the values given by this line will be somewhat in excess of the part actually taken by the concrete, though of course it may be of little consequence whether this small part of the load is taken by the concrete or is carried by the steel by reason of the greater stiffness given it by the concrete. A similar method was used for determining the part of the load taken by the concrete at the maximum strength of the column. The values thus found are given in Table 6. These methods of dividing load between steel and concrete will be used in the later discussion of the ratio of steel stresses to concrete stresses.

24. *Development and Amount of Concrete Stress.*—A consideration of these derived curves for the concrete (Fig. 11 and 15) enables the unit-stress on the concrete to be determined (a comparative and not an absolute figure) and also the manner of development of this stress throughout the test. For all mixtures it was found that at the beginning of the test the concrete took the load rapidly, with constant speed in the testing machine,

much the same as does the concrete in the cube or the cylinder. In Fig. 16 the stress-deformation diagrams are shown for the concrete of several columns and for the cylinder made from the same batch of concrete. In the earlier stages of the test the curves are not dissimilar. As the deformation increases, however, the concrete in the columns takes proportionately less and less stress. In general, too, the concrete in the column may be said to be less stiff than the same concrete placed in the cylinder.

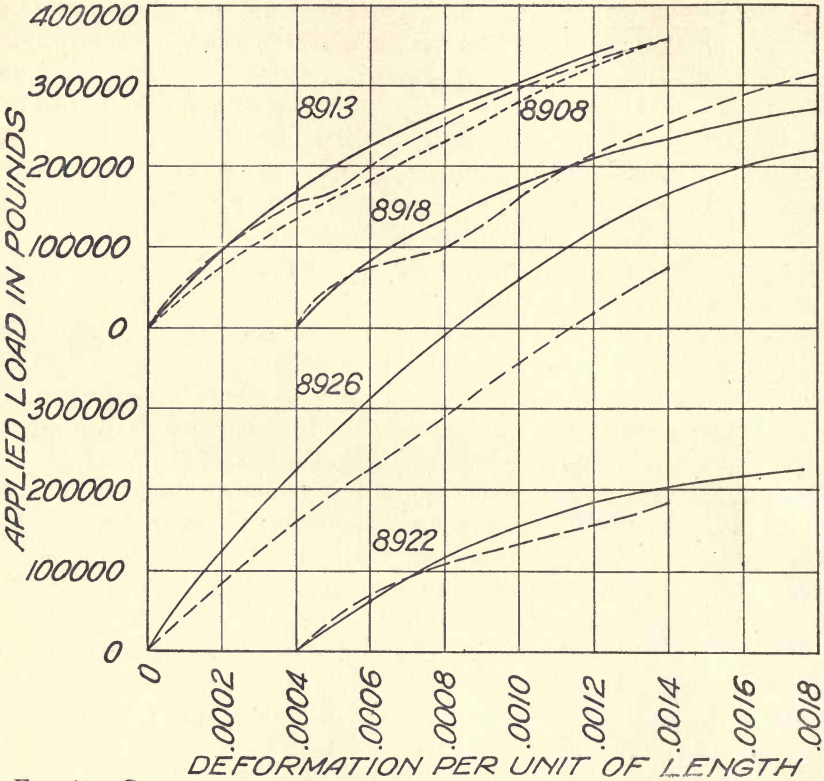

FIG. 16. COMPARISON OF STRESS-DEFORMATION IN COLUMN CONCRETE AND IN CYLINDERS.

For the 1–1–2 concrete there are no exceptions to this statement. For the 1–2–4 mixture 6 out of 8 columns follow the rule and the other two follow the rule up to a medium load. For the 1–3–6 mixture both columns show less stiffness up to a medium load and greater stiffness at the latter stages of the test than do the corresponding concrete cylinders. None of the concrete curves of Fig. 11 and 15 reaches its maximum value before the column as a whole reaches its ultimate load, but the curve becomes very flat at the higher deformations and shows a tendency for the concrete stress to become nearly constant in value over a considerable range of shortening. This tendency is one to be considered in deciding upon permissible stresses for use in designing columns of the core type.

The amount of stress considered to be taken by the concrete, upon the assumptions previously discussed, is given in Table 6, page 15. It is to be noted that the columns with 1–2–4 concrete made on October 28 and 29 give concrete stresses much lower than those made on November 8, so much so that the concrete of the columns falls into two distinct groups. The cube and cylinder tests fall into two similar groups. The materials were taken from the same lot and it is reasonably certain that the measurements and weights of materials are correct. The tests indicate that the cement used early in the season acquired its strength more slowly than that used later, (the later specimens acquired a normal amount of additional strength between the ages of 60 and 90 days) and the room temperature the last of October and the first of November was lower than later in the year. It is thought then that part of the difference in strength is due to differences in rate of hardening. Under the circumstances it will be best to treat the 1–2–4 columns as made of two grades of concrete, dividing them into two groups, those made October 28 and 29 in group (a) (No. 8907, 8912, 8917 and 8922) and those made November 8 in group (b) (No. 8908, 8913, 8918 and 8923). The columns with 1–1–2 concrete gave a similar but smaller variation. The distinctions named above have been indicated in Fig. 12 and 14, the solid symbols representing group (a) and the open symbols group (b).

25. *Comparison of Cube and Cylinder Strength with Column Strength.*—In Fig. 17 the values of the ultimate stresses taken by the concrete determined as described in a preceding paragraph are plotted as abscissas and the strengths of the corresponding cubes and cylinders made from the same batch of concrete as ordinates. The results of the two groups of 1–2–4 concrete are shown by separate symbols. The relation between the cube and the column strengths seems to be well expressed by a straight line; it indicates that the column concrete developed about two-thirds the strength of the same concrete tested in 6-in. cubes. The relation between the cylinder and the column strengths seems more uncertain. For the 1–2–4 mixtures, the ratio seems to be about 1. For the 1–1–2 mixture the cylinder shows higher strength and for the 1–3–6 mixture the core concrete shows higher

strength. The average of all the ratios of cylinder-column strength is about 1.

26. *Values of E and n.*—An accurate determination of the modulus of elasticity of the column concrete can not be made, but for the purposes of comparison it may be proper to use the stresses obtained by the method given in "23. Basis for Determining Load Taken by Concrete". Assuming these concrete stress-

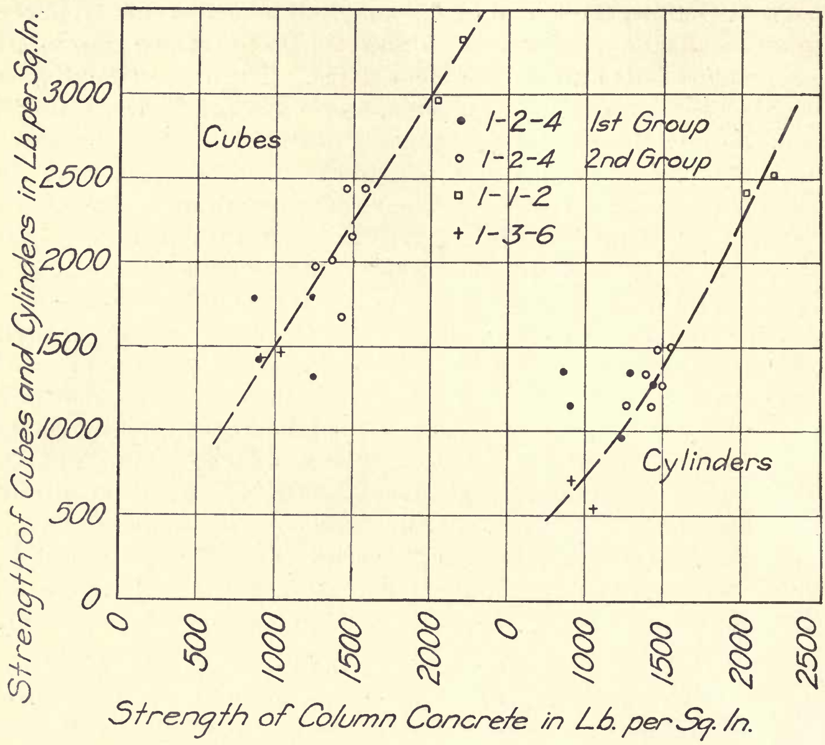

FIG. 17. COMPARISON OF STRENGTH OF CONCRETE IN CUBES, CYLINDERS AND COLUMNS.

es to be correct, the secant moduli of elasticity (E) have been calculated and are given in Table 8, page 22, for unit deformations of .0004, .0007, .0010 and for a deformation near the ultimate. The ratio between this modulus of elasticity and the modulus of elasticity of the plain steel column at the same deformation (values of n) are also given in Table 8. This ratio expresses the relation between the unit-stress taken by the steel and the unit-stress taken by the concrete. It is to be noted that these values of the ratio n are larger than ordinarily assumed in reinforced concrete design. This is probably due largely to the fact that the concrete in the column is less dense than the same concrete would be if allowed to settle and shrink freely. A striking fact is that for widely different values of the unit-deformations the value of n varies but little.

If the columns with 1–2–4 concrete are divided into the two groups noted in "24. Development and Amount of Concrete Stress", we shall obtain the values for the stresses taken by the concrete and the values of the ratio n given in Table 8. In discussing the value of n to be used in design it must be borne in mind that the concrete in the columns was not as well seasoned as would ordinarily be the case in building construction. It should

TABLE 9.

AVERAGE STRENGTH OF SPECIMENS AND VALUES OF n.

Stresses are given in pounds per square inch.

Mixture	Compressive Strength			Average Values of n	Suggested Values of n
	Cube	Cylinder	Column Concrete		
1-3-6	1400	690	990	36	35
1-2-4 (*a*)	1500	1150	1050	35	
(*b*)	2200	1300	1450	23	25
1-1-2	3100	2475	2125	16	16

also be said that, as in this type of column the steel will be used as the basis of design, the value of the ratio n to be accepted in design should be greater than the average value found, rather than less, in order to be on the safe side. In Table 9 values of n for the different mixtures of concrete are given which seem reasonable for use in design.

Another Basis for Design.—Another basis for design which seems rational is to determine the strength of the steel column for the $\frac{l}{r}$ of the steel column, taking this from the straight-line equation on page 23, and then to use as the strength of the concrete of the core section (without reference to the length of the column for the column slenderness usual in buildings) a value taken from the strength of plain concrete, say two-thirds of the cube strength, in this way combining the strength of steel and concrete. This seems to be in accord with the results of tests within the limits of the $\frac{l}{d}$ here used. Of course a suitable factor of safety would then be applied.

D. Fireproofed Columns.

27. *Phenomena of the Tests.*—Three columns in which a 2-in. shell of concrete was added to the core section were tested to determine the additional strength afforded by this covering and to study the behavior of a fireproofed column under load. During the earlier stages of the test there was no difference between the behavior for this type of column and one of the core type except that the outer shell did not take the expected proportion of the load. The concrete shell remained intact until the ultimate deformation of the column was practically reached, the unit-defor-

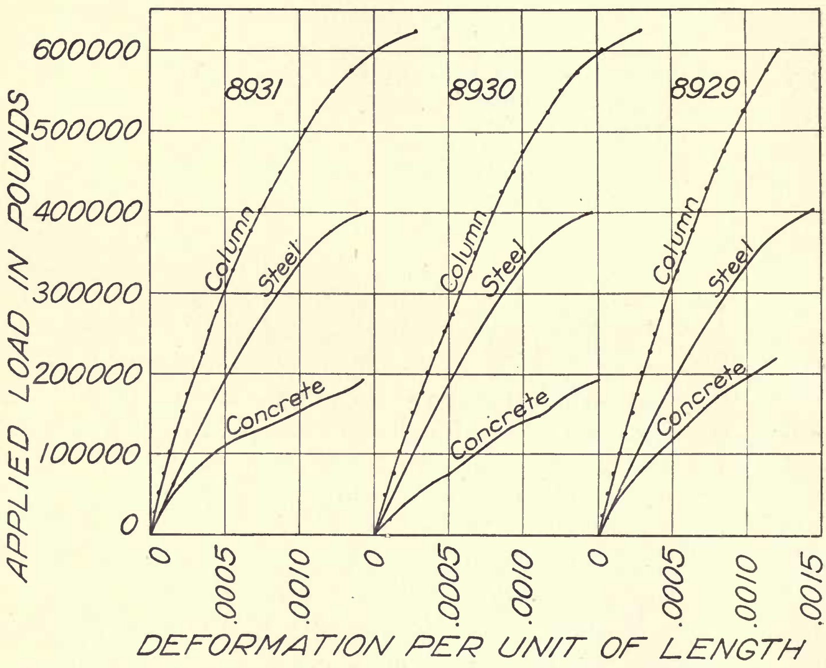

Fig. 18. Load-deformation Diagrams for Fireproofed Columns.

mation being .0018 or over at first crack. The steel was then evidently starting to yield and the concrete in the core was very close to its ultimate deformation and strength. When the shell cracked, the total load on the column dropped off about 65 000 lb. and the strength of the core itself was well evidenced by the length of time during which the load remained at this second ultimate although the machine was in operation in the meantime.

The load rose 2000 to 4000 lb. after its first drop, showing that the strength of the steel and of the enclosed concrete was not quite fully developed when the shell cracked. Fig. 4 gives a view after the maximum strength had been developed and the shell had cracked.

28. *Stress-deformation Relations.*—The stress-deformation diagrams are given in Fig. 18. The moduli of elasticity of the columns and the values of n are given in Table 8, page 22. Owing to the presence of the shell, which constituted 47% of the area of the total concrete section, and which it will be seen did not carry its full share of the load, the value of the modulus of elasticity of the column concrete is somewhat less for the fireproofed type than for the core type. Where n for the latter averaged about 23 for concrete of the same grade, it becomes more nearly 30 for the former. Since undoubtedly it is wisest not to figure on any load on the shell in designing, this value of n is significant only in showing that the use of values of n herein recommended does not threaten the safety and integrity of the shell.

29. *Comparison of Concrete Stresses on Gross Section and on Core Section.*—The maximum load carried by the fireproofed columns is given in Table 6, page 15. The division of load between steel and concrete, determined on the assumptions used in the discussion of the core type of column, is also given in this table.

Of the three fireproofed columns tested two were tested to failure. No. 8930 took a maximum load of 630 700 lb.; the load fell to 565 000 lb. when the shell failed, indicating that a load of 65 700 lb. was carried by the shell. For No. 8931 the maximum

TABLE 10.

STRESS CARRIED BY CONCRETE IN SHELL AND IN CORE OF FIREPROOFED COLUMNS.

Column No.	Stress in pounds per square inch		
	Gross Concrete	Shell Concrete	Core Concrete
8930	1 060	710	1 370
8931	1 090	680	1 440

load was 635 700 lb. and the core load was 572 000 lb., the difference, 63 700 lb., apparently being carried by the shell. Of the core load 418 000 lb. may be considered to have been carried by the steel (determined from the average strength of the plain steel columns of the same length), leaving on the core concrete 147 000 lb. for No. 8930 and 154 000 lb. for No. 8931. The area of the shell was 93 square inches and of the core concrete 107 square inches.

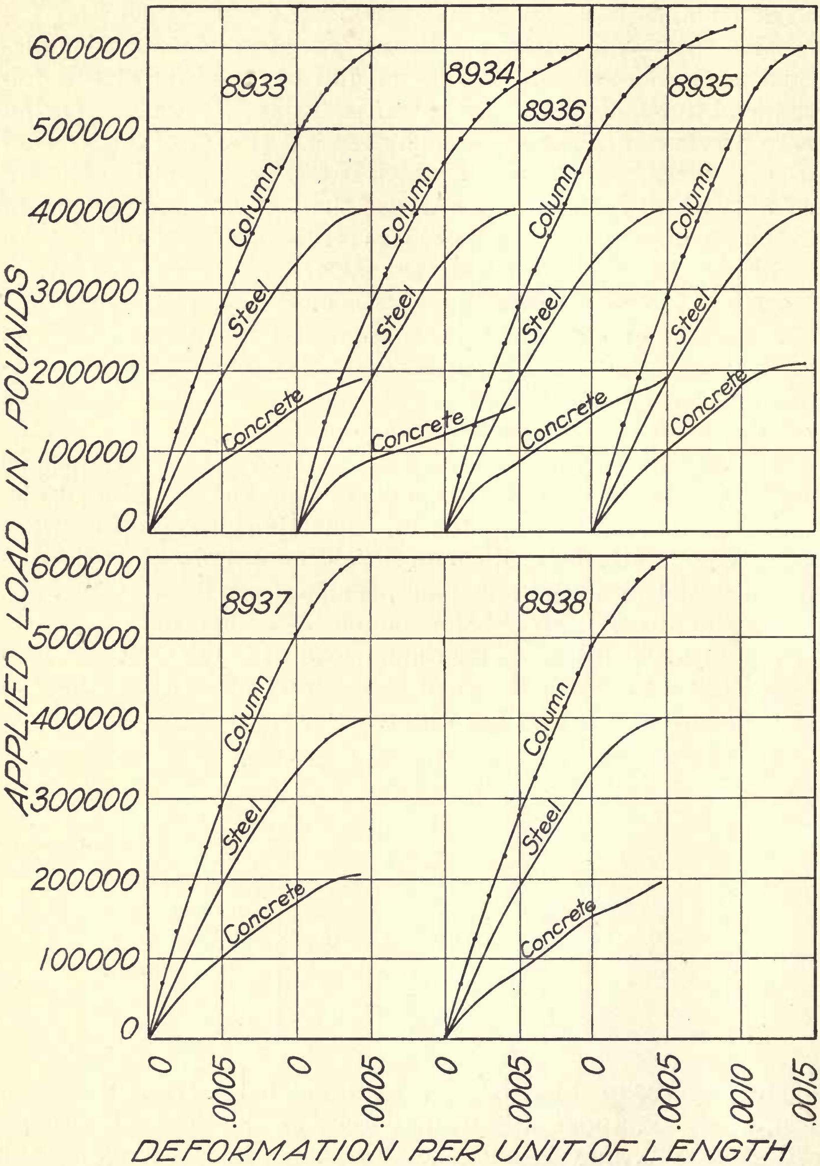

FIG. 19. LOAD-DEFORMATION DIAGRAMS FOR SPIRALED COLUMNS,—FIRST TEST.

The unit-stresses, as found by the method just outlined, are given in Table 10. These values indicate that the concrete shell carried roughly one-half as much load in pounds per square inch as the concrete of the core.

30. *Permissible Deformation as Governing Design.*—The question always arises in design as to the effect of large deformation in a column under load upon the integrity of the shell. If the shell goes to pieces at relatively low unit-deformations, it is evident that deformation rather than stress must govern in the selection of the working stresses to be used in design. As already noted, the shell did not crack or the load drop off until the unit-deformation exceeded .0018, and this is practically the deformation at the maximum load for the core type of column also. In other words, the action of the shell during the test did not seem to be such as to impose any restriction on the selection of working stresses.

31. *Need of a Tie to Prevent Stripping of Shell.*—Although the shell maintains itself intact under high unit-deformation, yet it seems that a tie of some sort (wire mesh, spiral, or other binder) should be imbedded in the outer shell to make its permanence certain. The backs of the flanges occupy one-fourth of the bonding surfaces between the shell and the core, and in practice this proportion might be even greater. It seems unwise to trust the fireproofing shell of the column to stand uninjured under collisions and accidents with so large a surface uncertainly supported. This tie would also prevent the rapid and complete failure of the shell at maximun load, as it occurred in the test of the columns, although this advantage is not very great in actual construction. The prevention of spalling away from the steel in case of a severe fire is a more important reason for requiring a metal binder to hold the exterior concrete in place. From the tests of spiraled columns it is concluded that a spiral is an excellent tie for this purpose.

E. Spiraled Columns.

32. *Phenomena of Tests and Stress-deformation Relations.*—For loads within the capacity of the University of Illinois testing machine (600 000 lb.) the load-deformation curves of the columns with $\frac{3}{4}$ % and with 1 % spiral reinforcement (see Fig. 19) are practically identical in nature with those for the core type. The tests were not carried to the point where the thin concrete coat-

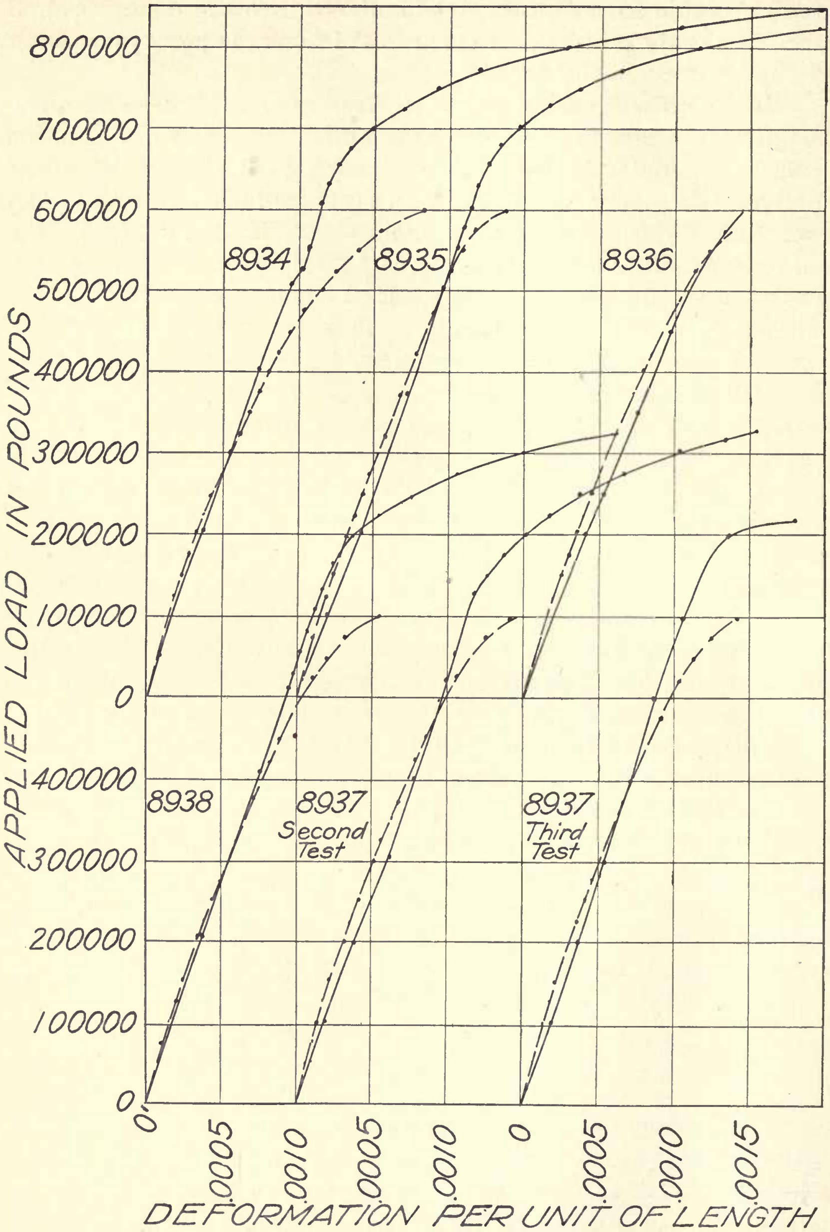

FIG. 20. LOAD-DEFORMATION DIAGRAMS FOR SPIRALED COLUMNS,—SECOND AND THIRD TESTS.

ing over the spiral would be expected to spall. In three of the spiraled columns the maximum load was applied three to five times, the result being increased deformations under maximum load and increased set upon the release of load.

Four of the spiraled columns were subsequently tested a second time at Lehigh University, where a load of 830 000 lb. was applied. Even with the heavier testing machine, the full strength of the columns was not developed, though No. 8934 evidently was loaded nearly to its maximum strength. The load-deformation diagrams are shown in Fig. 20. The deformations at the first tests of the same columns are also given by the broken lines. It must be borne in mind that the columns were about three months older at the second test than when first tested, and the increased strength of the concrete at a given unit-deformation may be accounted for by the increased age. To secure some conception of the amount of the added strength due to the greater age of the concrete, No. 8937 was tested a third time, the spiral first being removed, the outer concrete stripped off, and the column reduced to the section of the standard core type. The column in this condition carried a load of 714 000 lb., as compared with 547 000 lb. carried by the two corresponding columns of the core type at an age of 60 days, (see Fig. 20) and it is evident from this that a considerable portion of the added load carried in the second test was due to the greater age of the concrete. It is well to call attention to the fact that this increase of strength was gained after the concrete had been subjected to high stresses at an age of 60 days.

The large lateral deflection of the four columns tested at Lehigh University is of interest. In No. 8934, at a load of 752 100 lb. the deflection was .09 in. and at this load the concrete began to spall. At a load of 856 000 lb. the deflection was .26 in., a deflection set of .21 in. remaining when the load was released. At the fourth application of a load of about 850 000 lb. the deflection became .54 in., a set of .50 in. remaining with the release of the load. In No. 8935, at a load of 750 000 lb. the deflection was .084 in. At a load of 825 000 lb. the deflection became .31 in., and the concrete at the bottom scaled off between wires. Upon the release of load the set was .31 in. At a fifth application of the load, the deflection became .48 in. and the deflection set shown upon the final release of this load was .43 in. There was no marked bending in No. 8937 until the third application of a load of 830 000 lb., when it became .12 in.

In No. 8938, at a load of 776 000 lb. the deflection was .08 in., becoming .17 in. at a load of 825 000 lb., with a set of .16 in. shown upon release of load. At the fifth application of the load, the deflection became .25 in. with a resulting set of .22 in. This marked bending of the spiraled columns at loads above those which would be carried by an unspiraled column is in keeping with the large deflections found in hooped concrete columns of the usual type.

The phenomena of the second test were not essentially different from those described for the first test. At a load of about 750 000 lb. spalling of the outer concrete began on the columns having ¾ % spiral reinforcement, and this spalling continued during the remainder of the test. It seems evident from the action of the columns and the amount of deformation developed that these columns were very close to their maximum load at the end of the second test. The view in Fig. 5 shows the spalling of the concrete and the buckling of the spacing strip. With the columns having 1 % of spiral reinforcement the spalling was slight even at the highest load carried, and these columns evidently would have carried considerably more load. An examination of the structural steel after the second test showed no crimping of the flanges and no movement of the parts relatively to one another, although, of course, the total shortening of the column was not large. The spiral seems to have acted to hold the steel in alignment and to permit a greater shortening than would otherwise have taken place.

33. *Effect of Spiral.*—The maximum loads placed on the spiraled columns are given in Table 6, page 15. It must be borne in mind that the capacity of the testing machines used did not permit the maximum strength of these columns to be developed.

Within the limits of the first test (600 000 lb. load) the load-deformation diagrams do not show any effect which may be attributed to the variation in the percentage of spiral reinforcement or even to the presence of the spiral. A study of the stresses in these columns of the core type shows that the spiral has little apparent effect upon the action of the column within the load of 600 000 lb.

In the second test the maximum load applied evidently approached very closely to the maximum strength of the columns with ¾ % of spiral reinforcement, but it did not stress the columns with 1 % of spiral reinforcement nearly as severely. The load-deformation curves (Fig. 20, page 38) show that the marked yielding of the column takes place at about the same load for both percentages (650 000 to 700 000 lb.) and that at this yield point the spiral begins to play an important part. During this later part of the test the difference in amount of spiraling becomes apparent and the heavier spirals show greater strength. Since the tests were not carried to destruction, the amount of this added strength can not be ascertained, but its existence seems well established.

For most purposes, then, the significant point on the stress-deformation curves is the point where the curve bends sharply and becomes flat and which may be considered to be the load which would be carried by an unspiraled column. Beyond this point any additional load is carried only by virtue of very greatly increased deformations. For the four columns tested this point lies at or below 750 000 lb., and it will be interesting to compare this with the strength of the column without spiral reinforcement. For the purpose of this comparison the results of the third test of No. 8937, after it was stripped of its spiral and reduced to the standard core type, may be used to estimate the strength of the concrete at the time of the later test. This column carried 714 000 lb. load as against 580 000 lb. load carried by columns of the core type at 60 days of age. If we consider all the concrete within the spiral in the second test to be as effective as the core concrete in No. 8937, the load carried without aid from the spiral would be about 800 000 lb. This figure is probably a little high, as the concrete outside the core, although inside the spiral, would probably take somewhat less load per square inch than the concrete in the core. This tends to confirm the view that the load at the point when the load-deformation curve bends sharply and becomes flat is approximately equal to the strength of the unspiraled column, and that only the part of the strength developed after this yield point is passed should be attributed to the spiral directly. In No. 8934, this added load amounts to about 100 000 lb. which is 715 lb. per sq. in. of inclosed concrete. This is at the rate of 950 lb. per sq. in. of column concrete per 1 % of spiral reinforcement. Applying this figure to the 1 % spiral columns, the computed probable maximum load becomes 900 000 lb., a value which the deformations of the second test seem to indicate as a reasonable expectation for these columns.

34. *Availability of Spiral Strength for Design.*—The tests of this series of columns indicate that up to a unit-deformation of .0015 no appreciable difference in the action of columns with or without spiral reinforcement is found. In building construction the safe unit-deformation may ordinarily be placed at .0007 or less. It would seem that any attempt to use an imaginary spiral strength at working loads could result only in very high actual unit-deformations. The marked tendency of the column to bend laterally after the yield point was passed and the large amount of set found are also evidence of the unavailability of the higher strengths. The spiral does afford protection against sudden failure, and gives a tougher and safer column, and these properties may be considered to warrant the use of higher unit-stresses in spiraled columns. In the columns of the core type tested the need of a spiral is much less than in the ordinary reinforced concrete column, since these columns are found to possess toughness and the flanges of the structural angles restrain the core concrete to some extent. It thus appears that the use of a large percentage of spiral reinforcement in columns of the type here considered is hardly justifiable. A light spiral may serve to tie the shell together securely and protect it from accident, but this spiral should not be directly considered in the computations for strength of column.

F. Summary.

35. *General Comments.*—The columns tested were of a form now frequently used in building construction. The percentage of steel used (area of steel section 10.8 % of the area of the octagon inclosing the structural shapes) is within the range used in building construction. The conclusions given in the discussion relate to the properties of columns which have the forms and sections of the columns tested, and variations in proportions of metal and concrete may give somewhat different results. The tests, however, may be expected to throw light on the properties of columns of the same general type within the limits of ordinary design. The principal conclusions found in the discussion are as follows:

1. The maximum load carried by the plain steel columns is expressed by the straight-line formula, $\frac{P}{A} = 36\,500 - 155\frac{l}{r}$, where $\frac{l}{r}$ is the ratio of length of column to radius of gyration of the section of the steel column.

2. Earlier in the test the effect of length upon load carried at a given unit-deformation was less proportionately than at maximum load, the coefficient of $\frac{l}{r}$ in the equation being only 55 for a unit-deformation of .0008, and 27 for a unit-deformation of .0005, as compared with 155 in the equation for maximum load.

3. The load-deformation diagrams diverge from a straight line at loads well below the maximum.

4. In the concreted columns of the core type, the effect of length upon strength of column was almost identical with that found in the tests of plain steel columns. In other words the stress taken by the concrete may be considered to be nearly independent of the slenderness ratio of the column, within the limits of the lengths tested, and the stress taken by the steel may be considered to be the same as that taken by a plain steel column of the same slenderness ratio.

5. In the tests the concreted columns of the core type showed considerable toughness, though at the maximum load there was no material lateral deflection. The final failure of the concrete generally occurred at or above tie plates. The discussion shows that the concrete of the columns was less strong than the concrete of the cubes and less stiff than the concrete of the cylinders.

6. The stress taken by the concrete within the core or within the spiral is approximately equal to the strength of concrete of the cylinders tested and to two-thirds of the strength found in the 6-in. cubes.

7. The values of the ratio of the modulus of elasticity of the steel column to that of the concrete, n, under the assumptions used in the analysis, are much larger than are commonly used in reinforced concrete design. Values of n for use in designing are suggested in Table 9.

8. A basis for design which seems rational is to determine the strength of the steel column by the use of the column formula for the $\frac{l}{r}$ of the steel column and then to consider the concrete of the core section (without reference to the length of the column

for any ordinary length ratio, say a length of 15 diameters) to have a stress value proportional to the strength of the plain concrete, say two-thirds of the cube strength. A suitable factor of safety would of course be somewhere applied.

9. In the test of the fireproofed type of column (which had a shell of concrete outside the steel) the concrete shell remained intact until a deformation was reached as great as that developed at the maximum load in columns of the core type. This integrity of section at high deformations indicates that the presence of the shell need not impose any restrictions upon the working stresses available for the steel and for the core concrete. Of course, there are good reasons for the use of a metal binder like wire mesh or spiral for holding the shell securely in place.

10. The discussion indicates that the stress carried by the concrete of the shell is only about half of that carried by the core concrete. This lower strength is not objectionable, since the shell is not considered in designing the column.

11. The action of the spiraled columns indicates that the spiral has little effect up to a deformation and load corresponding to the maximum load for an unspiraled column. Beyond this load the column compresses rapidly and the presence of the spiral adds materially to the strength of the column. The tests do not fix the exact amount of this added strength.

12. In view of the large shortening necessary to make the added strength due to spiraling available and the general toughness of columns of the core type, it would seem that for building construction the use of a large percentage of spiral reinforcement in columns made up of structural shapes and concrete is hardly justifiable. A moderate spiral may warrant the use of somewhat higher unit-stresses, since it adds to the toughness of the column and gives a possible higher ultimate strength, and it will also serve to tie the concrete of the shell together securely and protect it from accident, but it does not seem best to consider this spiral directly in the computations for strength of column.

13. The columns tested possess the qualities of a good structural member and seem well adapted to more general use in building construction.

These comments are made on the assumption that the concrete is placed in as workmanlike a manner as is obtainable in the construction of high-grade work in columns reinforced with longitudinal rods or with rods and spirals.

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**Bulletin No. 22.* Tests of Cast-Iron and Reinforced Concrete Culvert Pipe, by Arthur N. Talbot. 1908. *Thirty-five cents.*

**Bulletin No. 23.* Voids, Settlement and Weight of Crushed Stone, by Ira O. Baker. 1908. *Fifteen cents.*

Bulletin No. 24. The Modification of Illinois Coal by Low Temperature Distillation, by S. W. Parr and C. K. Francis. 1908. *Thirty cents.*

Bulletin No. 25. Lighting Country Homes by Private Electric Plants, by T. H. Amrine. 1908. *Twenty cents.*

Bulletin No. 26. High Steam-Pressures in Locomotive Service. A Review of a Report to the Carnegie Institution of Washington, by W. F. M. Goss. 1908. *Twenty-five cents.*

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* Out of print; price attached.

Bulletin No. 27. Tests of Brick Columns and Terra Cotta Block Columns, by Arthur N Talbot and Duff A. Abrams. 1909. *Thirty cents.*

Bulletin No. 28. A Test of Three Large Reinforced Concrete Beams, by Arthur N. Talbot. 1909. *Fifteen cents.*

Bulletin No. 29. Tests of Reinforced Concrete Beams: Resistance to Web Stresses, Series of 1907 and 1908, by Arthur N. Talbot. 1909. *Forty-five cents.*

Bulletin No. 30. On the Rate of Formation of Carbon Monoxide in Gas Producers, by J. K. Clement, L. H. Adams, and C. N. Haskins. 1909. *Twenty-five cents.*

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Bulletin No. 56. Tests of Columns: An Investigation of the Value of Concrete as Reinforcement for Structural Steel Columns, by Arthur N. Talbot and Arthur R. Lord. 1912. *Twenty-five cents.*

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* Out of print; price attached.

ANNOUNCEMENT CONCERNING A MODIFICATION IN THE RULES GOVERNING THE DISTRIBUTION OF BULLETINS

The Board of Trustees of the University of Illinois voted, December 3, 1910, that a price should be affixed to certain University publications, among them being the Bulletin of the Engineering Experiment Station.

This action, so far as it concerns the bulletins of the Engineering Experiment Station, has for its purpose a threefold object:

(1) To provide a greater degree of control in the distribution of bulletins.

(2) To make possible the establishment and maintenance of a trade circulation through the regular publishing houses.

(3) To regulate the distribution of the reserve or "out-of-print" supply.

IT IS NOT INTENDED THAT THIS ACTION SHALL OPERATE IN ANY WAY TO ABRIDGE THE PRIVILEGES OF THOSE WHO HAVE HITHERTO RECEIVED THE BULLETINS OF THE STATION GRATUITOUSLY, OR TO PREVENT REASONABLE EXTENSIONS OF THE EXISTING MAILING LISTS.

Practice under the new procedure will be as follows:

(1) All bulletins, hereafter issued, will have a price printed upon the cover, together with the name of the London sales agent.

(2) EACH BULLETIN, HOWEVER, WILL STILL BE SUBJECT TO A FREE INITIAL DISTRIBUTION, AS HERETOFORE, ON THE BASIS OF EXISTING MAILING LISTS.

It will also be placed on sale with authorized agencies, both in this country and abroad.

(3) THERE WILL BE A LIMITED NUMBER OF COPIES AVAILABLE FOR FREE DISTRIBUTION, UPON REQUEST, AFTER THE INITIAL DISTRIBUTION.

(4) As the supply of each bulletin approaches exhaustion, it will be placed upon a reserve or "out-of-print" list. THE EFFECT OF THIS ACTION WILL BE TO WITHDRAW SUCH BULLETINS FROM FREE DISTRIBUTION. Bulletins withdrawn from free distribution will be available to any applicant upon payment of the assigned price.

(5) A discount from the published price will be allowed to news agents regularly handling the bulletins.

W. F. M. GOSS
Director.