(*Paper No. 3966.*)

"Composite Columns of Concrete and Steel."

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The effect of a concrete filling on increasing the carrying-capacity of a steel column has never been sufficiently investigated; accordingly the tests here described were made by the Author in order to ascertain the relative strength of plain steel columns compared with composite columns consisting of an outer frame of steel compression members and bracing filled with concrete.

These columns were tested at the testing-laboratory of the civil engineering department of Columbia University, New York, in June, 1908. It was not the intention to secure such refinement of results as might be obtained in a physical laboratory with specimens of small dimensions and of simple character, but rather to make both the plain-steel and the reinforced-concrete columns as nearly as possible similar to those which the engineer uses in practice.

The Two Types of Column.

In investigating the carrying-capacity and other physical characteristics of reinforced-concrete columns, the conditions involved in actual construction should be carefully kept in view. There are at present what may properly be considered as two classes of reinforced-concrete column.

The first class, that generally employed, is designed with the steel used both as a wrapping or banding material, and in comparatively small rods placed parallel to the column axis near to the bands. In some cases these small longitudinal rods may be secured to the wrapping material by steel wire, or they may even be used as a kind of cage, with a comparatively thin layer of concrete or mortar separating them from the enwrapping steel. In columns of this type the reinforcement is not so distributed or held as to constitute a direct load-carrying element of the column; its main purpose is to support the concrete by enwrapping it so as to prevent as far as possible failure due to compression, usually attended by shearing, as it would occur in plain concrete. The longitudinal steel rods may be capable of resisting by themselves some compression, but it is of such small amount as to exert little or no direct material influence upon the carrying-capacity of the member. Indeed the rods usually employed are so easily bulged or crippled laterally by small compressive loads that they would certainly be sources of weakness in the reinforced column if it were not for the spiral or other enwrapping steel. It has been shown by Professor A. N. Talbot and others that with this type of reinforcement the concrete receives little support from the reinforcing steel until the load is nearly equal to the ultimate resistance of plain concrete. In other words, while such reinforcement adds to the carrying-capacity of the concrete, it does not make its influence felt substantially with the progressive application of increasing loads until such limit has been nearly reached. It is obviously not well adapted for effective use in long columns, nor for high buildings where continuous columns frequently reach up through fifteen, thirty, or even forty stories.

The other class of column is distinguished by a type of steel reinforcement which is a direct load-carrying member in itself, while at the same time it effectively enwraps or bands the concrete so as to give it the greatest possible added carrying-capacity which any kind of banding can produce. In such columns the steel reinforcement is self-supporting, and it may be as light as can be made with the smallest shapes rolled, or may be as heavy as the demands of any special piece of work require. Such columns are adapted to buildings of any height whatever, and to any length of long-column design which may be required. The concrete is so closely banded by the steel column, and both parts of the member are so intimately and rigidly bonded, that each column must necessarily act as a firm unit, so that the steel reinforcement gives effective support to the concrete with every progressive addition of loading, from the lowest to the highest. In such columns the reinforcing steel member must have external dimensions of cross section nearly as large as those of the finished column, the outer thickness of mortar or other encasing material being just sufficient to give the requisite protection against fire or other sources of damage. This outer concrete or mortar is simply a protecting shell, only the concrete lying within the steel being considered as load-carrying. This is necessary because in case of fire much of the outside or protecting material may be destroyed or knocked off, so that the carrying member should be considered to consist only of the steel reinforcing column and the concrete lying within it.

This latter type of reinforced-concrete column was used in the design by the Author of what is known as the 39th Street Building, in New York City, a building occupied by the McGraw Publishing Company and other publishers. It has been used partially above the second floor for the operation of large printing-presses. The same type of reinforcement on a large scale will be used, also under the design of the Author, for the great 725-foot span, reinforced-concrete arch of the Henry Hudson Memorial bridge, which it is proposed to build across Spuyten Duyvil Creek in the northern part of the city of New York.

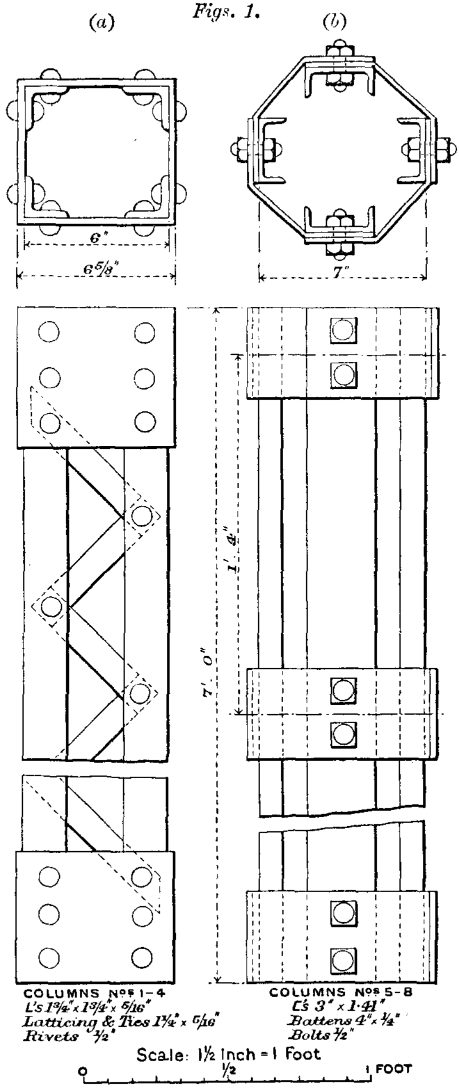
Description of the Columns Tested.

Four of the second class of reinforced-concrete columns were made for testing, and for comparison four similar steel columns without the concrete were also tested. The steelwork of these eight columns followed two designs (*a* and *b*, *Figs. 1*), there being four columns of each design, of which two were tested with and two without concrete. The 3-inch rolled-steel channels in design (*b*) weighed 4 lbs. per lineal foot. The total cross section of the four steel angles was 4 square inches, while the aggregate sectional area of the four 3-inch channels was 4·76 square inches. The steel angles were placed at the corners of a square, each side of which measured 6 inches, and were held in place by $1\frac{1}{4}$ by $\frac{5}{16}$-inch lattice bars with $\frac{1}{4}$-inch batten-plates at each end. The width between the exterior lattice bars filled with concrete was $6\frac{5}{8}$ inches, but the precise dimensions of cross section for purposes of computation cannot be stated exactly. Hence the cross section of the reinforced column as a load-carrying member will be taken as $6\frac{1}{2}$ inches square. The opposite channels of the other four columns were so placed as to be 7 inches from back to back, and were connected with 4-inch by $\frac{1}{4}$-inch batten-plates, so bent as to enwrap the four channels and make the finished column of octagonal form. The exterior dimensions of the steel in these four columns is $7\frac{1}{2}$ inches. The enwrapping batten-plates were 16 inches apart from centre to centre along the length of the column.

The four reinforced-concrete columns were built with the care given in the best constructional practice, as to both the steel and the concrete, but no more. Their total length was 7 feet.

It is evident that these columns are much smaller than those ordinarily used in reinforced-concrete construction, although they may be considered as coming within the minimum sizes of such structural columns. The circumstances of their manufacture and their dimensions, it is believed, give the results of the tests a significance applicable to actual reinforced-concrete columns of this type used in building or other construction.

Figs. 1.



STEELWORK OF COLUMNS TESTED.

The concrete used in all the columns was composed of 1 volume of Alpha Portland cement, 2 of excellent coarse sand, and 4 of trap rock, crushed small enough for the largest pieces to pass through a 1-inch ring, and by far the greater part of it to pass through a $\frac{3}{4}$-inch ring.

The ends of the steel reinforcing columns were carefully milled, to the exact length, at right angles to the axis of each column. The ends of the reinforced-concrete columns were flushed with neat Portland cement immediately before being placed in the testing-machine, so as to make an even bearing as nearly as possible over every part of each end.

Results of Tests.

The tests were made in an Olsen testing-machine of 400,000 lbs. capacity, the heads of which were fitted with hemispherical bearings so as to ensure accurate centre loading, and as nearly as possible perfect distribution of the applied load.

All compressions were measured by a compressometer reading to 1/10,000 inch, and the instrument was so set as to read compressions on an accurately measured distance of 70 inches. The lateral motions due to flexure were measured by screw micrometers, reading from rods held in place by the upper and lower frames of the compressometer; and care was taken to secure measurements in normal directions.

Curves showing compressions and lateral deflections were constructed from a complete set of observations, and are given for all the columns in *Figs. 2–5*. They show the deformations and other physical features of the tests as they progressed from the initial loading up to failure. It may be mentioned that the various set curves represent permanent set of the columns after removal of loads.

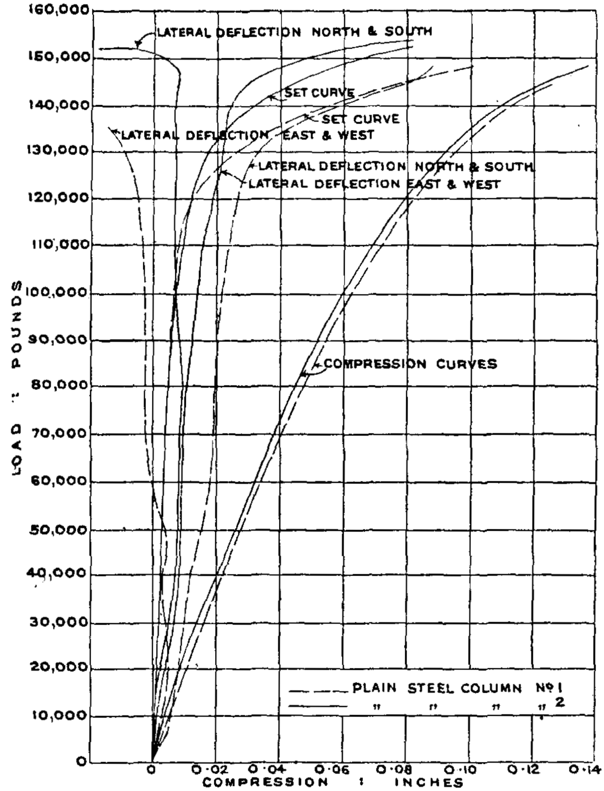
In order that full information regarding all the elements of resistance of these columns might be secured, the four plain steel columns, duplicates of the reinforcing steel in the other columns, were tested simply as steel columns. These columns are numbered 1, 2, 5, and 6, as indicated in the diagrams. The length divided by the radius of gyration was 34 for the angle-bar columns and 31 for the channel-bar columns; it may at once be observed that they were short steel columns and not long members. Curves showing the results of tests on these columns are given in *Figs. 2* and *4*, and the following Table shows the ultimate resistance obtained:—

Plain Steel Columns.

Column.	Type.	Sectional Area.	Ultimate Load.		Length.	Ratio l/r.
No.		Sq. Ins.	Lbs. per Sq. In.	Tons per Sq. In.	Inches.	
1	Angle (*Fig. 1, a*)	4·00	37,050	16·5	84	34
2	,,	4·00	38,000	17·0	84	34
5	Channel (*Fig. 1, b*)	4·76	32,600	14·5	84	31
6	,,	4·76	32,140	14·3	84	31

From this it will be seen that Nos. 1 and 2—the angle-bar columns with the usual lattice bracing—show ultimate resistances at failure distinctly higher than Nos. 5 and 6—the channel-bar

Fig. 2.

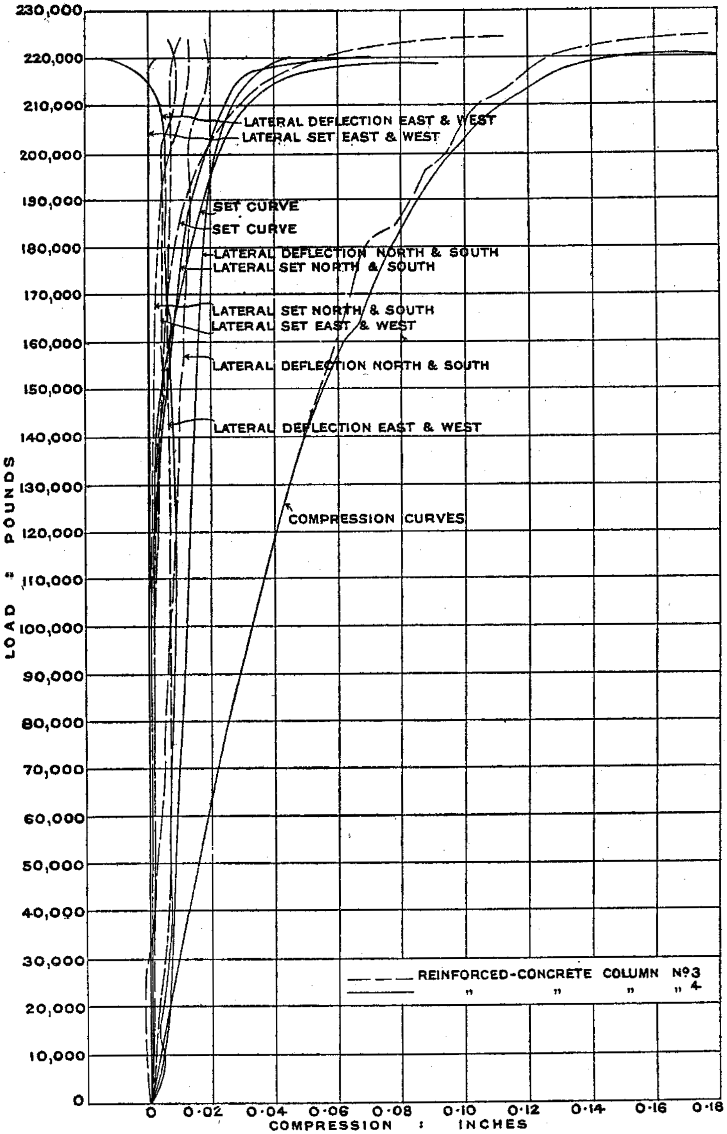


TESTS OF TWO PLAIN STEEL COLUMNS (*Design a, Figs. 1*).

columns without positive bracing between them, but with battens only. The advantage of holding the members of a column under compression by complete effective bracing is thus apparent.

Specimens of the steel taken from the angles and channels of these columns showed an average yield-point of about 40,800 lbs. per square inch, and an ultimate tensile strength of about 61,300 lbs. per square inch. No effort was made to secure any

Fig. 3.

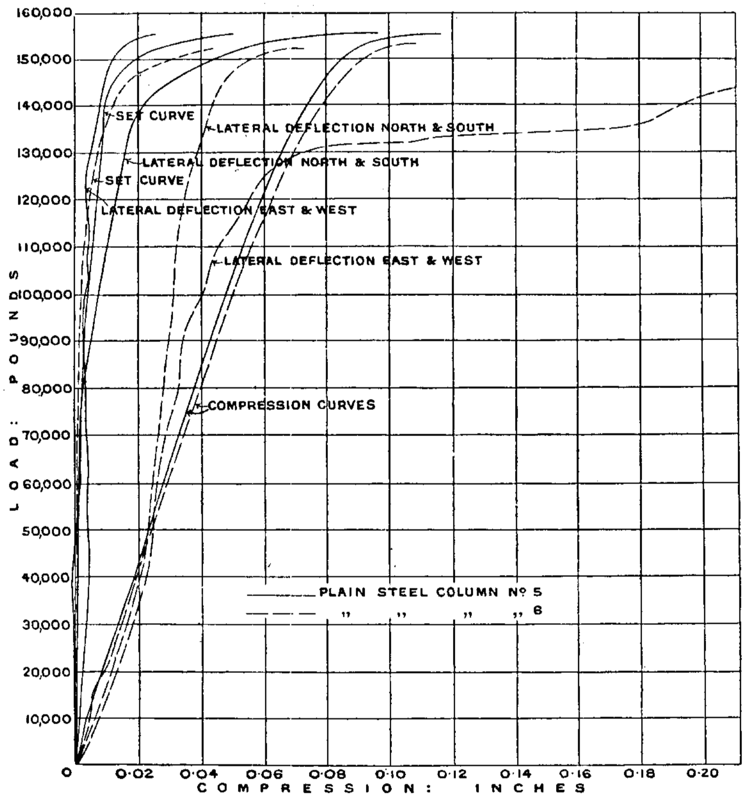


TESTS OF TWO REINFORCED-CONCRETE COLUMNS (*Steelwork of Design a, Figs. 1*).

special quality of structural steel, but the results show that it was a good medium material.

The concrete in the reinforced columns was 3 months old when the columns were tested. It was mixed by hand, reasonable care being taken to secure a thorough mixture, but nothing more, and

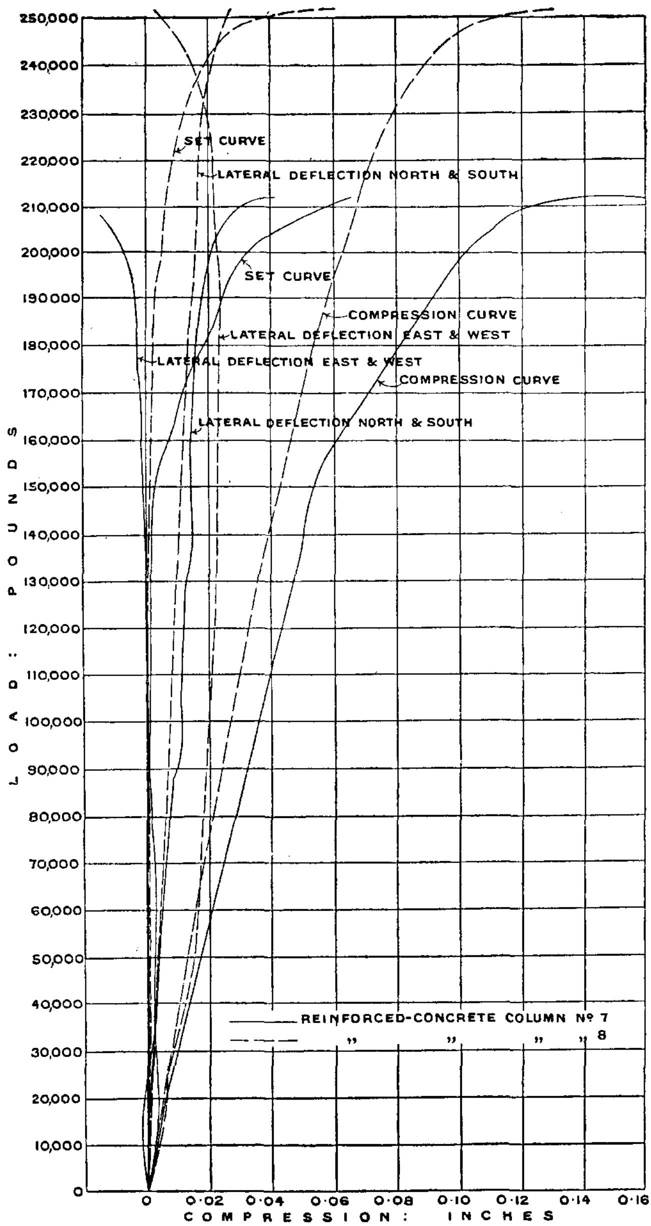
Fig. 4.



Tests of Two Plain Steel Columns (*Design b, Figs. 1*).

the concrete was therefore a fair material corresponding closely with that used in ordinary first-class work. The columns were kept in a vertical position from the time when the moulds were filled to the time of testing. They were frequently wetted, especially in the earlier part of the 3 months.

Fig. 5.



Tests of Two Reinforced-Concrete Columns (*Design b, Figs. 1*).

Calculations.

In discussing analytically the results of the tests of the four reinforced-concrete columns the following notation will be employed:—

E_s denotes the coefficient of elasticity for steel = 29,000,000 lbs. per square inch.
E_c ,, ,, coefficient of elasticity for concrete (to be found).
A_s ,, ,, area of steel section in square inches.
A_c ,, ,, area of concrete section in square inches.
A ,, ,, $A_s + A_c$.
Δl ,, ,, shortening of column or compression for 70 inches length as found in tests.
E ,, ,, coefficient of elasticity for combined section so that $A\,E = A_s E_s + A_c E_c$.
P ,, ,, total load on a reinforced-concrete column, as found by test.

These total loads, P, are obviously taken from the test records shown on the curves.

The coefficient of elasticity for steel is taken at 29,000,000 lbs. per square inch as that is a fair and close value of the quantity for the mild structural steel used in the columns. The coefficient of elasticity for concrete, as is well recognized, is not nearly so well defined, since it depends upon the age of the concrete and the thoroughness of mixing, as well as upon the proportions of the cement and aggregate and upon other conditions attending the making of the concrete. Inasmuch as these reinforced-concrete columns were about 3 months old, the coefficient of elasticity for the concrete is somewhat less than that which it would attain at a greater age, although in most cases its further increase in value would not be great. It is to be remembered, in connection with such tests as these, that the coefficient of elasticity for concrete at an age not greater than about 3 months, and sometimes at much less age, must be depended upon for load-carrying conditions in the progress of construction of almost any reinforced-concrete structure.

In general the following equation may be used for the determination of E_c:—

$$A_s E_s \frac{\Delta l}{70} + A_c E_c \frac{\Delta l}{70} = P \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Since Δl is given by the test curves, all quantities are known in equation (1) except E_c, which can be evaluated at once for any particular loading for a given test.

Results of Tests.

On examination of the curves in *Fig. 3* it will be clear that the reinforced columns Nos. 3 and 4 reached what may be termed, approximately at least, their elastic limit at a load (P) of 180,000 lbs., Δl having the value 0·072 inch. Equation (1) however may be applied to total loads of 160,000 lbs. and 120,000 lbs. respectively, in order to discuss points certainly below the loading at which the columns show deficient elasticity.

Similarly values for columns Nos. 7 and 8 have been calculated from *Fig. 5* by taking total loads at 200,000 lbs., 160,000 lbs. and 120,000 lbs. It is seen, however, that column No. 8 shows a much better diagram than column No. 7, and an independent set of values has been used for that column at a loading of 200,000 lbs. Both these calculations have resulted in the figures given in the following Table. Average values of Δl for each pair of reinforced columns have been taken, except in the case of column No. 8 :—

Reinforced-Concrete Columns.

Total Load.	E_c	Stress in Steel.	Stress in Concrete.	E	Mean Crippling Stress on Total Section.
Lbs.	Lbs. per Sq. In.	Lbs. per Sq. In.	Lbs. per Sq. In.	Lbs. per Sq. In.	Lbs. per Sq. In.
Columns 3 and 4.					
120,000	2,321,000	16,990	1,360	4,850,000	5,210 (Section = 42·25 sq. ins.)
160,000	1,850,000	24,850	1,585	4,420,000	
180,000	1,536,000	29,870	1,582	4,140,000	
Columns 7 and 8.					
120,000	1,843,000	15,750	1,000	4,442,000	4,230 and 5,030 respectively (Section = 49·75 sq. ins.)
160,000	1,718,000	21,550	1,276	4,310,000	
200,000	731,000	33,930	855		
200,000[1]	1,648,000	26,100	1,483	4,270,000	

For columns Nos. 3 and 4, $r_c = 2{\cdot}26$; $\frac{l}{r_c} = 37{\cdot}2$; $\frac{l}{d} = 13$.

For columns Nos. 7 and 8, $r_c = 2{\cdot}4$; $\frac{l}{r_c} = 35$; $\frac{l}{d} = 11{\cdot}2$.

[1] Column No. 8.

The radii of gyration (r) have been computed for the combined concrete-and-steel normal column section. The ratio of column-length to radius of gyration has the values 37·2 and 35, only, showing that the columns, as such, are short rather than long. The ratios of length over exterior dimensions (d) of the column are also given for both pairs.

On the whole the results of the tests are not much different for the two forms of column-section, but what difference there is appears to favour the latticed-angle reinforcement. Somewhat higher values of both the coefficient of elasticity for the concrete and for the intensities of pressure are found for columns Nos. 3 and 4 than for Nos. 7 and 8. Similarly the coefficient of elasticity E for the combined material is rather higher on the whole for the same columns.

The modes of failure of the eight columns were the following:

Column No. 1, plain steel, failed by bulging of angles near one end.

Column No. 2, plain steel, failed partly by general flexure and partly by bulging of angles near one end.

Column No. 3, reinforced concrete, failed by the bulging of angles about 15 inches from one end; there was but little lateral deflection of the column as a whole. At the same time a fine vertical crack developed in the concrete near the top and at the centre of the column.

Column No. 4, reinforced concrete; this column showed initial failure at one corner at the end, although there was considerable general flexure. The angles bulged at one end and produced failure of the adjoining concrete.

Column No. 5, plain steel, failed by general flexure with deformation at centre.

Column No. 6, plain steel, also failed by general flexure.

Column No. 7, reinforced concrete, failed by bulging of channels near one end, causing a rupture of bond between steel and concrete.

Column No. 8, reinforced concrete, failed also by bulging of channels near the end destroying the bond between the steel and concrete.

These tests are too few in number, and the section of the concrete is too small relatively to that of the steel, to furnish a basis for any safe broad generalization, but it is believed that the results are significant as to certain general features of columns of this type. Among other things it would appear that it is advisable to design the steel member by the same general principles which govern the design of the best class of steel columns, as in this manner the steel is given the best form both for effective banding and support of the concrete, and for the carrying of load by itself.

It also appears from these tests that it is abundantly safe and proper to take the coefficient of elasticity for the concrete at least as high as 2,000,000 lbs. per square inch, at the age of 3 months, and for such loads as are ordinarily imposed upon reinforced-concrete structures under proper specifications.

It would be interesting and of much value to continue the testing of similar columns for greater ratios of length to side, or radius of gyration, and at greater ages, in order to determine more completely, and therefore more satisfactorily, the ultimate compressive resistance of the concrete. It is believed that such tests would justify working-stresses as high as 500 to 750 lbs. per square inch in columns of this nature, and possibly higher stresses for structures of unusual magnitude.

The Author is indebted to Assistant-Professor J. H. Woolson and to Mr. J. S. Macgregor for devising details of testing apparatus and conducting the actual tests under his direction.

The Paper is accompanied by three diagrams, from which the Figures in the text have been prepared.