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AN INVESTIGATION OF BUILT-UP COLUMNS UNDER LOAD

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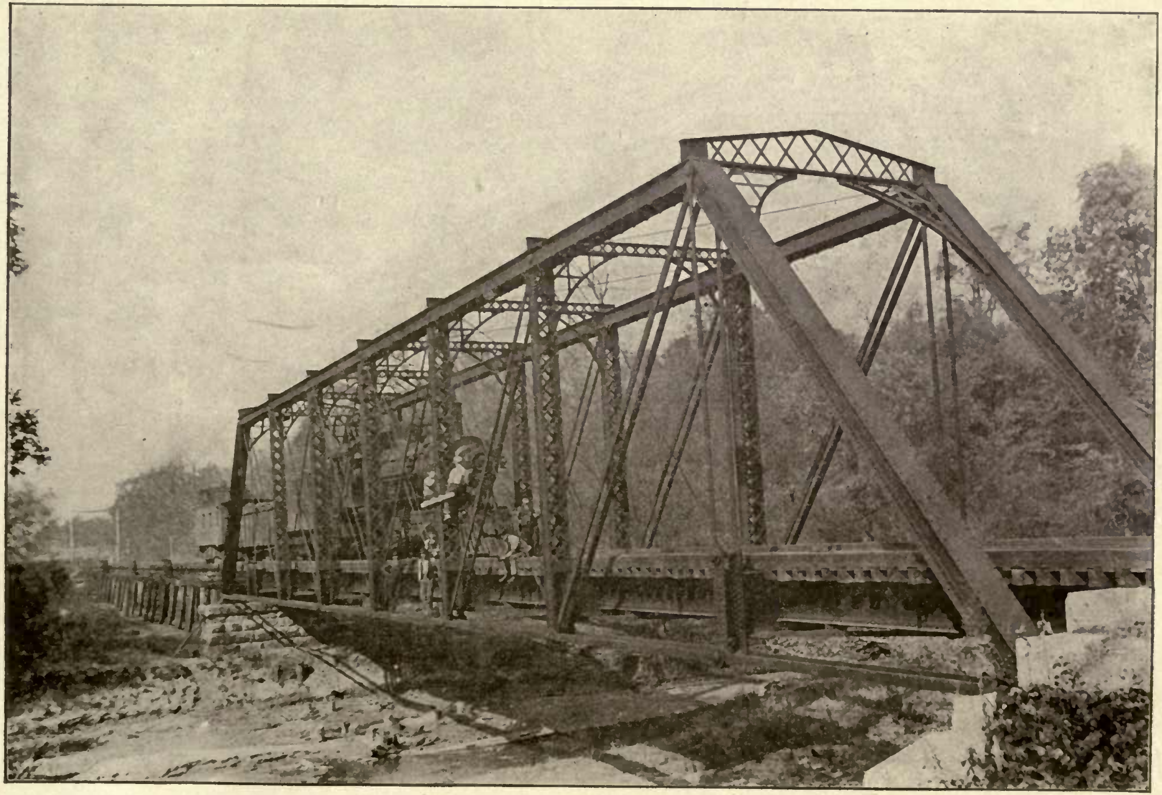

White Heath Bridge and Test Train

UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

BULLETIN No. 44 JUNE, 1910

AN INVESTIGATION OF BUILT-UP COLUMNS UNDER LOAD

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AN INVESTIGATION OF BUILT-UP COLUMNS UNDER LOAD

I. Introduction.

1. *Scope of Bulletin.*—The investigation described in this bulletin was taken up with a view of determining experimentally: (1) something of the way in which the compressive stresses in built-up columns vary over the cross-section of the channels or other component parts and throughout their length; (2) something of the amount and distribution of stress in the lattice bars of columns, and also the action of similar bars under separate tests with similar conditions of fastening and eccentricity; and (3) the general relation which exists between the component parts and the column as a whole. The investigation may be said to differ from the usual tests of columns, where the main purpose is to determine the ultimate strength of the column and the effect of length, in that emphasis is placed on measuring the distribution and range of stress over the various parts of the column. The making of tests to determine the distribution of stress in such compression pieces has commonly been held to be impracticable. In several respects these tests may be said to be pioneer tests along the line of the determination of the distribution of stress under load, whether that load be applied by a testing machine or by a locomotive and train in service.

The principal tests were made on the following compression pieces: (a) a steel column (called Column No. 1) built up of angles, plates, and lattice bars, all the parts being light with respect to the size of the column; (b) four wrought-iron bridge posts which had seen long service in a bridge truss; and (c) three posts and a top chord in a railroad bridge under service. The tests of (a) and (b) were made in a testing machine; for (c) a locomotive and cars formed the load. The auxiliary tests which were made on lattice bars and other parts have an important bearing in connection with the design of columns.

It is well known that built-up compression pieces (whether long or short) are not perfect, the natural imperfections of the component parts being increased in the process of fabrication. To non-homogeneity of structure and lack of straightness in the component angle or channel are added such further imperfections as kinks and eccentric connection of parts, which go to increase the opportunities for local flexure in the component parts and for flexural stresses in the column as a whole. An attempt has been made in these tests to measure the deformations in the presence of such conditions, and to find the general distribution of stress. In view of the many limitations surrounding such tests, the results are to be taken as suggestive and qualitative, and not as exact determinations.

The methods of testing and the results of the tests are given under the heads: II. Laboratory Tests of Columns, III. Field Tests of Columns, and IV. Tests of Lattice Bars, Small Columns, and Column Material. Under V. Discussion, is given a general discussion of the tests and a short discussion of the bearing of the results upon methods of design, together with a summary of the conclusions.

2. *Acknowledgment.*—The steel test column was furnished by the American Bridge Co., Mr. August Ziesing, President. The wrought-iron columns were bridge posts taken from an old bridge of the Chicago, Burlington and Quincy Railroad, and were furnished through the courtesy of Mr. L. J. Hotchkiss, Assistant Bridge Engineer. The arrangement for the test of the railroad bridge was made through Mr. R. E. Gaut, Bridge Engineer of the Illinois Central Railroad, and to him and to Mr. C. R. Westcott, Division Superintendent of the Illinois Central Railroad, special acknowledgment is made for the use of the engine, train, and crew for eight days.

The investigation was the work of the Engineering Experiment Station of the University of Illinois. The observations both in the laboratory tests and the field tests were made by skilled observers, and care was taken to make the tests trustworthy in all respects. Much of the experimental work has been described in Vol. LXV of the Transactions of the American Society of Civil Engineers.

3. *Basis of Column Formulas.*—For the purposes of this discussion a column may be considered to be a prismatic piece, having a length several times its breadth, and subject to nominal axial compression. It is, then, a compression piece in which there is chance for failure at one side of the column by reason of the added stresses of lateral flexure. The column may be a

single solid piece throughout, as in the case of a rolled section, or it may be built up of rolled angles or channels by riveting the members together or by connecting them by plates or lattice bars, as is the usual practice in bridges and other structural steel work.

The analysis ordinarily used in deriving column formulas assumes the existence of flexure in the column as a whole. The deflection in the axis of the column may result from initial eccentricity at the point of application of the load, lack of homogeneity in the material (which will allow bending to begin), a general bend in the column as a whole, or a combination of two or more of these conditions. Except for the initial eccentricity, the amount of the bending moment producing flexural stress is usually assumed to vary as the square of $\frac{l}{r}$ (ratio of length to least radius of gyration). The constants for these semi-rational formulas have usually been obtained by fitting the formulas to the experimental results, and the results of tests have also been used as a basis of purely empirical formulas. Unfortunately, the range of experiments for any given form or type of column has not been large, and especially has information been lacking on the properties of short compression pieces of the character used in the larger columns. In the light of recent tests it seems probable that too much weight has been given to the bending action of the column as a whole and also that, for short and medium lengths, the strength of the column at its elastic limit is not as great, relatively, as it has been considered to be.

Column analysis further assumes the integrity of cross-section of the column; that is, it assumes that the component angles or channels will act as a unit to resist bending so that a plane section before loading will remain plane after loading. It may well be questioned whether the ordinary riveted column does maintain its integrity to such an extent that the whole section will act as a unit. In the case of lightly built columns and of those having parts inadequately laced together, it would seem that the looseness or lack of integrity may greatly affect the distribution of stresses. At any rate, this is a subject which should be investigated before accepting integrity of section as a feature of column action. It will be seen that if the component members or parts of a section act somewhat independently, the conditions of column action will not agree with the usual assumptions. If, for example, the individual parts of a column are very thin, there may be a tendency for these thin parts to wrinkle under compression, and failure by such wrinkling may occur at loads less than would cause the column to fail by direct compression or by bending as a whole. Professor Lilly, of Trinity College, Dublin, has made an experimental study of this wrinkling effect in small columns of various cross-sections.*

Again, it may be noted that in the process of fabrication of the built-up columns, kinks and bends are formed in the component pieces. This condition produces initial stresses and also gives local bending action under load in these pieces. It will be shown that a very slight bend in a thin channel member may cause very severe stresses to be set up. During the process of fitting and riveting in column fabrication the material may be stressed locally beyond the yield point. It would seem reasonable to suppose that a column may have a much different distribution of stress throughout its members than would be expected in an ideal column which would be perfectly straight and homogeneous and which would have its integrity of cross-section preserved under load.

4. *Secondary Stresses in Columns.*—Such conditions as eccentricity of loading, crookedness of column, either general or local, and lack of homogeneity of parts, which act to produce variations in longitudinal stresses throughout the length in the different members of a column, produce transverse shear in the column. To resist this the column parts are riveted together or connected by plates and lattice bars. These shearing forces are usually small, but in the larger columns they become very important. Various attempts have been made to investigate mathematically the distribution and amount of shear in the different parts of a column, but all such analyses depend upon integrity of cross-section and assume a regular change in bending moment from end to middle of column. The conditions attending fabrication of built-up columns seem to make it impracticable to assert with any degree of certainty how far these assumptions may be right. Besides, it is possible that, by reason of conditions

*The Strength of Columns, Proceedings of the Institution of Mechanical Engineers, June, 1905. The Design of Struts, Engineering (London). January 10, 1908.

resulting from the process of erection, torsional stresses may be set up in the column, and the ordinary column is very poorly adapted to resist such stresses. It seems very desirable that experiments on columns should include a measurement of the stresses in the lattice members.

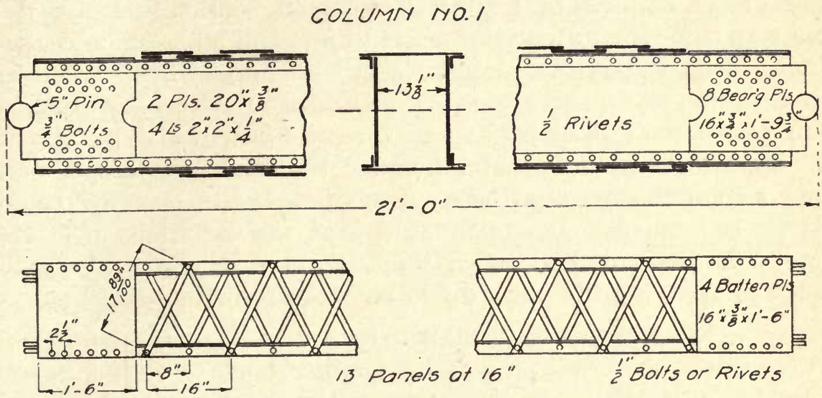

FIG. 1. STEEL TEST COLUMN NO. 1.

5. *Methods of Experimental Study.*—Much of the column testing described in engineering literature has had for its main subject the determination of the ultimate strength of the columns. Observations have been made on the shortening of the column as a whole, and the elastic limit or yield point of the column has been determined. Generally speaking, however, there has been no study of the distribution of deformations throughout the test piece. In outlining the tests described in this bulletin it was believed that a study of the distribution of stress over the cross-section and throughout the length of the column would give results which would be of value. The method adopted was, therefore, to make a measurement of the deformations produced over short spaces at different parts of the column under test and to make these measurements so that the lateral bending of the component pieces of the column could be found. The tests also included the measurement of the deformations in lacing bars and their distribution over the bar. To throw light upon the action of the column, special tests were also made on lacing bars.

II. Laboratory Tests of Columns.

6. *Description of Columns.*—One steel column and four wrought-iron columns were tested. The steel column (designated here as Column No. 1) was specially designed for the purpose of these tests, and was of a much less stocky section than are the built-up columns ordinarily used in bridge and building construction. Fig. 1 shows the details of this column. The section of this column was chosen because it seemed to offer better opportunities than a less flimsy column for the study of distribution of stress, lateral and longitudinal, under the conditions of the test, and also because the stresses developed in the latticing could better be studied. It was thought that the variations of stress due to methods of fabrication, handling in shipment, and conditions of applying the load would be more pronounced than in a

TABLE 1.

Data of Columns.

Column Designation	Column No. 1	Wrought-iron Posts		White Heath Bridge	
		No. 2, 3, 4 and 5	Retest 2a and 4a	Posts U3L3 North U3L3 South	Upper Chord U3U4 South
Area of column section, inches	18.76	17.64	17.64	12.02	48.67
Length, c. to c.	21 ft.	15 ft. 10 in.	14 ft. 7 in.	25 ft.	19 ft. 10 in.
End conditions	Pin parallel to lacing	Pin parallel to lacing	Pin parallel to lacing	Lower end pin, upper end riveted	Riveted
$\frac{l}{r}$ axis parallel to lacing	37.8	43.5	40.1	66.1	40.7
$\frac{l}{r}$ axis perpendicular to lacing	37.2	41.2	38.0	41.0	29.6
$\frac{l}{r}$ of each flange member, axis perpendicular to lacing for full length column	593	400	367	416	——
$\frac{l}{r}$ of each flange member, axis as before, for distance between adjacent lacing rivets	37.7	33.7	33.7	22.2	——
Lattice bars, dimension of section, inches	1x¼-in. and 1¼x$\frac{7}{16}$-in.	2½x⅜-in.	2½x⅜-in.	2¼x⅜-in.	2¾x⅜-in. on bottom
Kind of lacing	Single	Double	Double	Double; riveted at crossing	One cover plate
Angle of lattice bar with axis of column	63° 30′	45°	45°	45°	45° on bottom

stocky column, and hence that the flimsy column would be capable of more accurate study. In this connection it should be noted, however, that the chord members of large bridges are sometimes built up of parts relatively as thin as the parts of this test column. The steel column was built at the Lassig plant of the American Bridge Company. In the earlier tests of this column the lattice bars were fastened in place by turned bolts in reamed holes, and two sizes of lattice bars were used in the different tests, but in the later tests the bars were riveted in place.

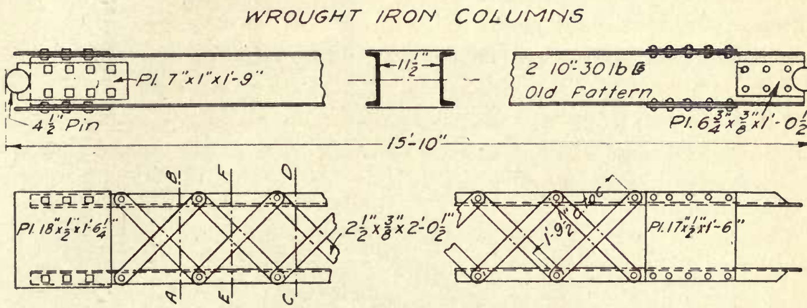

FIG. 2. WROUGHT-IRON TEST COLUMNS NO. 2, 3, 4 AND 5.

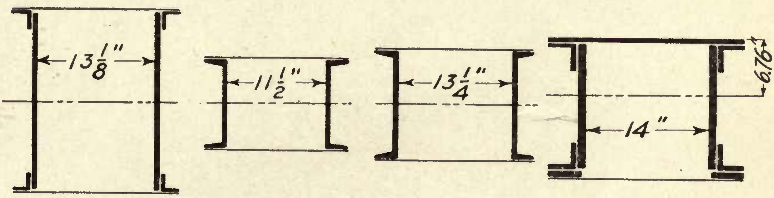

FIG. 3. CROSS-SECTIONS OF TEST COLUMNS.

The wrought-iron columns were from an old bridge of the Chicago, Burlington and Quincy Railroad. For the purpose of the test the posts were cut in two; the old ends were left as used in the bridge, and bearing plates and batten plates were bolted to the other ends. The proportions of these wrought-iron columns represented good practice at the time of the erection of the bridge. The columns became available for testing through the replacement of the bridge by a heavier structure; they were apparently in good condition. Fig. 2 shows the details of one of these columns. Fig. 3 shows to scale the cross-sections of all columns, both in the laboratory and in the field tests. Table 1 gives the general data of all columns tested.

7. *Testing Machine.*—The machine used in testing the columns was the Riehle vertical 600 000-lb. screw-power machine in the Laboratory of Applied Mechanics of the University of Illinois. This machine has a clear space of 36 in. between screws. There is thus room around a column for instrumental work. It will take compression specimens 25 ft. long. It is equipped with a heavy guide frame—not touching any part of the weighing apparatus—which takes any side thrust present in the test. The speed of head in nearly all tests was 0.4 in. per min. The machine has been shown to be accurate and trustworthy.

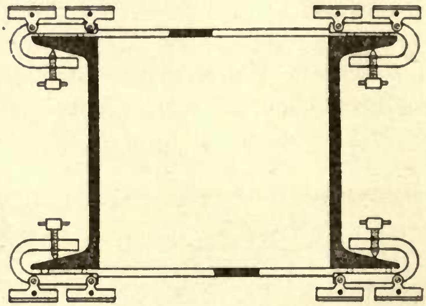

FIG. 4. ATTACHMENT OF EXTENSOMETER TO CHANNEL MEMBERS OF COLUMN.

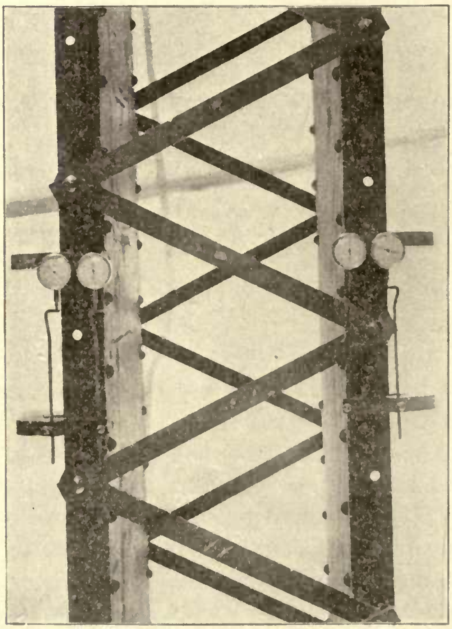

FIG. 5. EXTENSOMETERS IN PLACE ON CHANNEL MEMBERS OF COLUMN.

8. *Extensometers.*—In the earlier testing work various types of extensometers were tried. As a result of the trial the extensometers used in the later tests for measuring deformation in the

channel members consisted of Ames test gauges mounted on suitable frames. Each frame was in the shape of a C-clamp and bore against the channel member through three blunt points and a screw. Fig. 4 shows the shape of these clamps. These instruments magnify change of length by means of clockwork operating a hand rotating over a dial. They read directly to $\frac{1}{1000}$ in. and by estimation to $\frac{1}{10\,000}$ in. For measuring the deformation of the lattice bars of Column No. 1, a Ewing extensometer was used. In this instrument the displacement of a cross hair is viewed through a microscope. The instrument reads directly to $\frac{1}{5000}$ in., and by estimation to $\frac{1}{50\,000}$ in. It is a very accurate

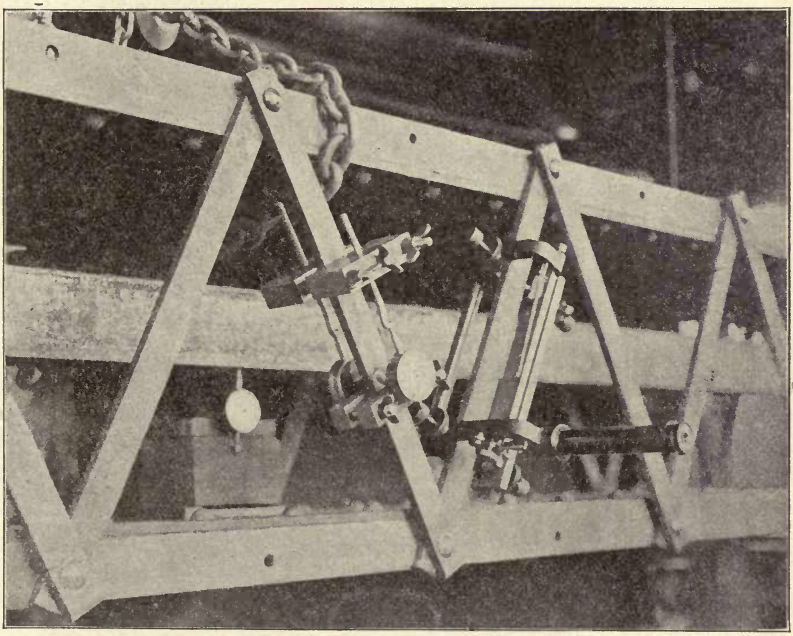

FIG. 6. EXTENSOMETERS IN PLACE ON LATTICE BARS OF COLUMN.

piece of apparatus but is not adapted to a wide range of size of specimens. It could not be used on the lattice bars of the wrought-iron columns, and on these bars the Ames test gauges were used.

Fig. 5 shows the attachment of the Ames instruments to the channel members of a column, and Fig. 6 shows the attachment of both the Ames and the Ewing instruments to lattice bars.

The magnitude of error liable to be present in the determination of stresses from the readings of the extensometers was studied with some care. The accuracy of all the Ames gauges used was tested by comparison with a Brown and Sharpe micrometer acting through a 10 to 1 lever. The average deviation of a reading of the Ames dial was found to be $\frac{9}{100000}$ in. and the maximum observed deviation $\frac{8}{10000}$ in. The tests covered a range of motion of pointer slightly greater than that observed in the column tests. Basing judgment on the maximum deviation observed in calibration, and on the smallest deformation observed in the columns, it seems probable that the error in stress determination for the channel members is in all cases less than $\pm$ 10% and that in general it is much less. This general limit of accuracy is corrobo-

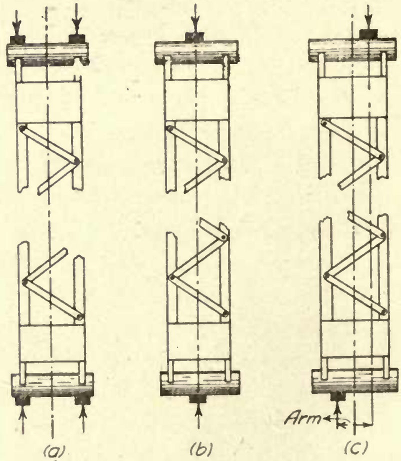

FIG. 7. METHODS OF LOADING: (a) REGULAR CENTRAL LOADING, (b) CENTRAL LOADING, COLUMN NO. 2a AND COLUMN NO. 1 FOR TESTS NO. 11, 12 AND 14, (c) OBLIQUE LOADING, COLUMN NO. 2a, AND COLUMN NO. 1 FOR TESTS NO. 12, 13, AND 15.

rated by a comparison of the average stresses at various cross-sections of the column as determined from the extensometer readings and from the load as indicated by the testing machine. To those accustomed to the apparently greater refinement of many laboratory tests and to the greater precision of calculations frequently employed, the above errors may seem unduly large. However, it may be considered that the instruments gave satisfactory results, especially in view of the large variation of stress distribution over the length of the columns, the general consistency of the results, and the fact that every stress determination is based on more than one reading and also that every conclusion is based on several stress determinations.

As noted above, the Ames test gauges were used to measure deformations in the lattice bars of the wrought-iron columns. On account of the very low stresses in the lattice bars it is felt that the stresses determined in them may be in error by ± 20%. In measuring the deformation of the lattice bars of Column No. 1 with a Ewing extensometer, the accuracy was greater, and the errors in determination of stress in lattice bars of Column No. 1 are probably not greater than ± 10%.

The Ames test gauges were light, durable, easily read, and adapted to a very wide range of conditions. In other tests they had successfuly withstood hard service. Any available instrument of greater precision would have been too bulky or too liable to injury or derangement of parts under the severe conditions of test, and especially under the conditions of field tests of columns.

9. *Procedure of Tests.*—The stress distribution was studied by measuring the compression or shortening over a short distance longitudinally. This measurement was made at several places in the cross-section. The dials were placed slightly outside the column, and the deformation along the extreme fibers of the channel members was later computed on the assumption for each channel member that a section plane before deformation remains plane after deformation. This hypothesis is not dependent upon the integrity of the column as a whole, but only upon that of the individual channel members. The position of the instruments at one location is shown in Fig. 5 (p. 12). As the elastic limit was not exceeded in the tests of stress distribution, in the interpretation of the data the stress in the piece is assumed to be proportional to the deformation. Necessary shifting of instruments and repetition of load made the test proceed very slowly. In studying the stress distribution of Column No. 1 for each method of loading it was necessary to apply the load about three hundred times. This took about three days of actual work after the column was adjusted in place. For each position of the instruments the load was applied and readings taken at least twice, frequently three times, and in cases where especially large readings, or especially small readings, were noted, five to ten readings were taken.

A similar procedure was followed in the tests of lattice bars in the study of stress distribution in them.

TABLE 2.

Stresses in Column No. 1.

Stresses are given in pounds per square inch.

Panel	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber
Test No. 1.												
1	8 100	10 200	10 600	12 200	10 000	9 700	6 500	8 400	8 600	9 600	10 900	11 300
2	9 700	10 100	10 100	8 100	9 600	10 000	11 900	11 500	11 600	8 900	10 500	11 100
3	7 500	10 200	11 000	11 800	10 000	9 700	11 500	9 800	9 800	9 000	10 300	11 000
4	10 500	10 000	10 000	10 500	11 000	11 000	9 000	8 600	8 600	10 800	10 100	10 500
5	9 600	9 100	9 100	10 900	9 700	9 500	10 300	8 500	9 700	8 800	10 100	10 600
6	9 900	11 000	11 100	10 000	9 600	9 700	9 400	8 400	8 100	9 000	9 200	9 100
7	10 000	7 600	6 900	11 000	10 000	9 900	9 000	9 400	9 900	9 200	10 000	10 400
8	8 100	8 900	9 000	10 800	11 300	11 500	12 700	10 500	10 400	12 200	10 100	10 300
9	10 000	11 000	11 100	9 400	11 000	11 000	14 500	10 000	9 000	9 000	9 200	9 200
10	14 000	10 000	9 300	14 300	11 800	11 000	5 400	8 900	9 300	7 200	9 600	10 000
11	8 600	9 500	9 600	10 000	9 800	9 600	9 000	8 800	8 400	9 800	11 000	11 400
12	12 300	9 200	8 700	9 100	9 400	9 800	10 800	9 300	9 000	10 700	9 600	9 500
13	....	....	....	8 700	8 800	9 000	7 800	9 000	9 200	......	......	......
Test No. 2.												
6	11 500	12 100	12 100	11 500	10 000	9 500	9 100	9 000	9 300	5 800	7 500	7 700
7	12 200	12 600	12 700	14 600	13 500	13 300	5 900	7 500	8 000	8 300	9 500	9 500
8	11 300	12 500	12 800	12 400	12 500	10 600	7 800	8 200	8 300	7 500	10 300	10 700
9	14 800	13 300	12 700	9 500	10 500	10 600	11 800	7 300	7 000	10 300	8 400	8 300
10	9 000	10 200	10 400	16 200	10 500	9 600	7 800	10 500	11 400	9 000	8 400	8 500
11	15 200	10 200	9 700	13 700	11 600	11 500	7 600	8 100	8 300	5 000	6 300	8 600
12	10 300	11 900	12 300	12 700	11 600	11 500	9 400	8 200	7 700	7 000	7 500	9 700
13	13 200	11 800	11 700	9 900	10 500	10 800	7 500	8 600	9 100	6 300	6 300	8 400
Test No. 3.												
1	....	....	....	13 800	10 400	9 500	7 500	8 800	8 900	......	......	....
2	8 900	10 600	10 900	8 800	10 400	10 600	8 300	9 300	9 400	10 000	9 700	9 700
3	10 500	9 800	9 800	8 700	9 000	9 200	10 400	8 700	8 700	11 900	10 500	10 300
4	9 800	10 800	11 000	11 100	10 700	10 700	8 300	8 300	8 500	13 300	12 600	12 200
5	10 500	10 000	9 800	11 200	10 700	10 700	9 000	10 000	10 100	9 300	9 500	9 400
6	11 300	12 300	12 500	10 900	10 500	10 700	10 400	9 700	9 500	7 500	9 400	9 600
7	10 400	10 700	10 700	11 900	11 000	10 800	9 200	10 000	10 000	8 400	8 800	8 800
8	12 100	10 200	10 200	12 100	13 000	13 100	8 500	9 300	9 400	7 300	9 000	9 400
9	7 600	9 900	10 200	10 500	10 900	10 800	14 400	9 300	8 400	10 900	10 600	10 700
10	10 500	12 000	12 100	14 200	11 000	10 500	7 200	10 400	10 900	7 800	8 700	8 700
11	14 200	10 500	9 700	7 500	8 500	8 600	8 300	9 400	9 300	7 000	9 400	9 800
12	10 400	12 000	12 200	6 500	7 500	7 800	11 000	9 300	9 000	7 900	9 100	9 400
13	12 600	9 800	9 500	8 300	8 400	8 200	7 200	9 800	10 200	11 000	10 800	10 900

TABLE 2—(*Continued*).

STRESSES IN COLUMN NO. 1.

TEST NO. 4.

Panel	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber
1	......	......	......	......	......	......	......	......	......	......	......	......
2	11 300	10 8000	10 700	10 500	11 100	11 200	6 100	3 000	2 500	8 900	10 200	10 600
3	10 000	11 6000	11 600	10 100	11 100	11 100	8 400	8 500	8 500	12 000	11 300	11 100
4	11 200	12 3000	12 500	9 900	10 900	11 000	7 600	9 400	9 700	7 900	9 300	9 500
5	......	......	......	12 800	11 100	10 700	......	......	......	10 700	10 200	10 100
6	10 100	5 400	4 500	12 800	10 700	10 300	6 500	8 900	9 500	......	......	......
7	8 000	10 600	11 000	10 200	9 700	9 700	......	......	......	8 700	8 500	8 600
8	9 800	12 000	12 400	9 700	10 000	10 100	9 000	9 400	9 400	10 300	11 300	11 400
9	11 500	10 600	10 400	......	......	......	8 500	10 700	10 900	7 900	11 100	11 500
10	8 600	8 300	8 100	6 800	8 700	9 100	10 800	7 500	7 000	9 300	10 700	11 000
11	......	......	......	......	......	......	9 500	9 000	8 900	13 000	12 700	12 600
12	10 900	10 400	10 300	10 700	9 900	9 600	......	......	......	......	......	......
13	11 000	10 700	10 700	11 100	10 600	10 500	7 300	9 800	9 600	12 700	11 900	11 800

TEST NO. 5.

Panel	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber
½	12 100	10 400	9 300	11 000	9 500	8 700	9 700	10 500	10 900	10 400	10 800	10 900
1	9 800	11 100	11 800	10 000	10 200	10 500	12 200	10 400	9 300	13 500	11 300	10 400
1½	7 900	8 500	8 600	5 800	8 200	8 900	7 300	8 400	8 900	7 400	7 900	8 300
2	11 800	11 400	11 100	10 900	10 600	10 600	8 500	8 600	8 700	9 800	10 200	10 400
2½	10 700	11 000	11 100	10 800	10 200	10 200	11 800	11 900	11 800	9 700	9 700	9 600
3	5 800	7 500	8 500	7 800	8 000	8 500	8 300	8 500	8 600	7 700	9 100	9 700
3½	8 000	8 100	8 500	7 400	8 100	8 500	7 800	8 600	9 000	7 500	7 600	7 700
4	15 400	13 000	12 400	13 500	13 000	12 800	11 600	11 800	11 800	9 900	9 900	9 900
4½	12 200	10 700	10 300	12 200	10 200	9 200	9 900	9 900	9 800	9 800	9 800	9 700
5	9 400	9 500	9 600	11 000	10 100	9 600	9 700	9 500	9 500	8 900	9 800	9 900
5½	9 900	10 700	11 100	10 500	10 900	11 000	10 900	9 100	8 300	8 700	8 800	8 900
6	7 400	7 300	7 200	8 100	9 200	9 700	13 800	11 900	11 100	11 100	10 600	10 200
6½	10 900	10 200	9 700	10 200	10 300	10 500	8 600	8 900	8 900	8 500	8 600	8 600
7	12 100	11 700	11 400	11 400	10 900	10 400	8 300	8 800	8 900	5 900	7 900	8 700
7½	9 900	9 200	8 900	9 600	9 400	9 500	6 500	7 600	8 300	4 600	6 400	6 900
8	9 800	8 600	8 300	8 100	9 700	10 400	11 400	9 500	8 500	12 700	10 800	10 100
8½	9 200	9 700	10 000	9 700	9 500	9 300	20 200	15 400	13 100	15 400	14 000	13 200
9	6 500	7 600	8 300	9 700	9 700	9 900	12 900	13 200	13 100	9 400	9 900	10 200
9½	10 000	10 400	10 600	12 900	9 500	8 100	4 600	7 800	8 900	4 600	7 300	8 500
10	16 400	12 900	11 200	17 000	14 400	12 900	8 000	8 600	8 700	6 800	8 000	8 600
10½	11 300	10 700	10 400	9 900	10 000	9 700	10 500	11 100	11 300	7 400	8 900	9 600
11	7 000	8 400	8 900	8 100	9 100	9 600	12 600	11 100	10 800	9 800	10 800	11 000
11½	9 700	8 900	8 600	11 900	10 500	9 700	8 200	11 100	11 700	8 900	9 700	9 800
12	14 000	12 400	11 600	11 900	11 000	10 400	6 900	7 500	7 800	8 100	8 800	9 400
12½	9 700	8 900	8 600	10 500	12 100	12 700	10 800	11 800	12 400	11 900	11 000	10 800

TABLE 3.

Stresses in Wrought-iron Bridge Posts.

Panel	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber
Column No. 2, Test No. 6.												
1	4 600	6 500	7 300	6 400	7 900	8 500	4 300	8 300	9 700	5 000	6 800	7 400
2	7 300	7 300	7 300	8 300	8 400	8 400	9 800	9 000	8 800	9 000	8 000	7 600
3	5 500	5 900	6 000	7 100	7 800	8 000	7 700	8 800	9 200	6 700	7 600	7 900
4	6 200	6 100	6 200	6 100	6 900	7 100	8 400	8 800	9 000	8 200	8 000	8 000
5	7 200	6 500	6 100	8 800	8 600	8 500	7 500	8 800	9 300	8 700	7 800	7 500
6	6 800	6 900	6 900	8 400	8 400	8 400	7 100	7 600	7 800	6 900	7 200	7 400
7	7 500	7 900	8 000	7 100	7 400	7 500	7 500	8 200	8 500	7 500	7 400	7 300
8	6 400	7 400	7 800	6 200	7 200	7 600	7 700	7 900	8 000	6 600	6 400	6 100
Column No. 3, Test No. 7.												
1	9 300	9 000	9 000	8 400	9 500	10 000	10 000	9 000	10 100	8 200	8 000	7 900
2	10 200	10 000	9 900	10 200	11 500	11 900	8 400	9 100	10 000	9 100	8 700	8 500
3	8 800	8 700	8 600	9 800	10 000	10 000	10 000	9 700	9 600	9 400	9 600	9 600
4	10 500	9 100	8 600	8 500	9 500	9 900	8 600	9 800	10 200	9 300	7 500	6 900
5	10 200	10 400	10 500	8 800	9 600	9 900	10 100	9 800	9 700	10 400	10 200	10 200
6	10 600	10 000	9 800	10 300	10 000	10 000	10 400	10 400	10 400	10 000	9 900	9 900
7	10 300	9 900	9 700	9 600	9 900	10 000	8 700	9 500	9 700	10 900	10 000	9 700
8	9 800	9 600	9 500	10 200	10 900	11 000	7 600	9 600	10 300	7 800	9 900	10 600
Column No. 4, Test No. 8.												
1	9 000	9 800	10 000	10 000	10 700	11 000	8 200	10 000	10 800	9 000	9 200	9 300
2	8 700	9 300	9 500	10 100	10 800	11 100	10 100	11 600	12 100	8 600	9 400	9 700
3	9 300	9 800	9 900	10 800	11 200	11 400	10 000	10 200	10 200	7 400	8 400	8 700
4	9 500	9 700	9 800	11 100	10 200	10 000	9 700	11 100	11 600	7 100	8 200	8 500
5	10 200	10 500	10 600	11 900	12 400	12 600	8 400	9 400	9 800	7 700	8 800	9 200
6	11 600	12 000	12 100	12 400	13 600	13 900	9 500	10 800	11 100	7 400	9 500	10 200
7	12 800	12 500	12 400	13 400	14 200	14 600	7 100	8 300	8 700	6 400	6 700	6 700
8	8 600	11 600	12 700	13 000	11 300	10 700	6 600	8 500	9 200	6 200	7 800	8 400
Column No. 5, Test No. 9.												
½	14 000	11 700	10 800	12 600	11 700	11 400	7 400	8 100	8 400	7 400	7 900	8 000
1	11 300	12 000	12 300	11 100	10 000	9 600	6 700	8 500	9 200	5 100	6 800	7 500
1½	11 300	11 100	11 100	11 100	10 800	10 700	5 700	6 200	6 300	6 600	6 900	7 100
2	13 100	12 600	12 400	12 900	12 400	12 200	8 000	8 000	8 000	7 800	8 200	8 300
2½	12 700	12 000	11 700	13 100	11 500	11 100	9 300	9 200	9 200	8 700	8 400	8 300
3	11 800	12 000	12 200	11 400	12 200	12 400	8 200	8 400	8 400	6 300	8 000	8 600
3½	13 500	10 500	9 300	11 600	10 700	10 400	8 200	8 200	8 200	8 400	8 300	8 200
4	14 300	13 100	12 700	14 300	12 300	11 500	7 300	7 400	7 500	7 400	8 700	9 200
4½	11 600	11 600	11 500	12 300	12 300	12 300	9 000	9 000	9 000	8 700	8 700	7 800
5	12 600	12 400	12 300	11 700	10 400	10 000	8 300	8 400	8 400	7 400	8 500	8 100
5½	13 700	12 100	11 500	13 500	12 100	11 400	8 300	8 300	8 300	8 400	8 400	7 600
6	12 600	12 700	12 700	13 500	12 500	12 200	7 600	7 600	7 600	7 200	7 000	7 100
6½	13 100	11 900	11 500	11 400	11 400	11 500	8 400	8 400	8 400	10 500	9 000	8 500
7	11 100	11 300	11 500	11 200	12 300	12 600	7 900	8 000	8 000	6 500	8 100	8 600
7½	10 600	10 100	9 900	11 000	10 300	9 900	8 000	8 200	8 200	5 900	7 900	8 700
8	10 300	12 300	12 800	11 300	12 300	12 400	8 500	8 600	8 600	10 800	11 800	12 200

TABLE 3—(*Continued*).

Panel	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber

Column No. 4a, Test No. 10.

½	8 100	9 500	10 200	8 400	8 100	8 000	11 700	10 800	10 500	9 900	11 200	11 800
1	8 300	10 200	10 800	6 500	7 200	7 500	12 100	10 900	10 400	11 100	12 500	13 100
1½	8 400	9 400	9 700	7 400	8 200	8 400	12 100	11 400	11 100	11 000	11 400	11 600
2	8 700	8 800	9 000	6 200	7 200	7 500	10 800	10 500	9 700	9 300	11 700	12 600
2½	8 300	8 800	9 000	7 300	6 900	6 800	11 800	11 500	12 100	10 900	10 400	10 300
3	8 700	9 700	9 900	6 700	6 500	6 400	13 300	11 100	10 400	10 400	11 100	11 400
3½	8 300	8 000	8 000	7 300	7 300	7 400	13 600	11 600	11 100	13 000	12 400	12 400
4	8 200	9 700	10 100	6 800	6 900	6 800	14 300	13 500	12 700	13 100	11 100	10 500
4½	7 500	8 600	9 100	6 300	7 200	7 500	12 300	11 100	10 800	10 300	15 200	9 400
5	7 500	8 400	8 800	7 000	8 100	8 500	13 000	13 300	13 500	9 000	11 400	12 300
5½	6 300	7 200	7 500	5 900	6 100	6 200	12 700	12 300	12 100	12 300	10 700	10 100
6	6 800	7 800	8 200	5 300	6 800	7 200	14 200	14 200	14 200	11 000	11 200	11 300
6½	6 600	7 800	8 200	7 100	7 100	7 100	15 400	13 700	12 900	13 500	11 500	11 100
7	6 700	7 100	7 200	4 900	6 100	6 700	11 900	13 600	14 600	11 100	12 100	12 400
7½	6 100	7 600	8 300	4 800	8 100	9 400	12 300	12 400	12 400	9 300	16 200	11 100
8	6 000	8 200	9 000	8 400	6 800	6 200	10 700	12 200	12 800	10 000	16 300	11 100

Column No. 2a, Test No. 11.

½	11 600	10 900	10 400	11 400	10 500	10 000	....	....	....	....	....	....
1	7 300	9 300	10 000	7 300	9 700	10 400	8 400	8 800	9 000	10 200	10 200	10 200
1½	11 600	9 900	9 600	11 600	10 900	10 000	8 500	10 000	10 600	11 400	12 000	12 200
2	8 900	8 700	8 600	10 500	10 300	10 000	10 200	9 000	8 600	11 500	11 200	11 000
2½	11 200	9 700	9 200	10 900	10 300	10 000	9 900	10 000	10 200	11 700	10 300	9 900
3	9 500	10 500	10 700	12 200	10 800	10 000	11 800	10 200	10 800	11 400	10 600	10 300
3½	10 500	11 000	11 000	12 100	10 600	10 000	10 400	8 600	8 100	9 700	9 600	9 400
4	9 800	9 500	9 400	10 600	9 900	9 600	9 800	9 400	9 200	10 400	10 600	10 500
4½	10 300	9 600	9 300	11 200	10 200	10 000	8 700	9 600	10 400	10 400	10 000	10 000
5	9 600	9 800	9 900	10 400	10 800	11 000	10 300	9 200	8 800	10 400	10 200	10 200
5½	12 200	11 600	11 200	12 000	11 100	10 700	12 500	10 400	10 000	12 200	11 900	11 700
6	10 100	9 700	9 700	9 900	9 700	9 700	9 100	10 400	10 900	10 300	10 700	10 700
6½	11 300	11 000	10 800	10 600	9 900	9 700	10 800	9 900	9 700	11 000	11 000	10 000
7	9 300	9 600	9 700	9 100	9 300	9 200	9 800	9 800	9 800	9 800	10 800	10 000
7½	10 200	10 900	11 400	9 100	9 400	9 500	9 300	9 600	9 800	9 900	10 700	10 800
8	9 900	9 800	9 700	11 100	10 000	9 600	8 800	10 300	10 900	10 800	10 700	10 800

Column No. 2a, Test No. 12.

½	10 600	10 000	9 800	10 400	10 600	10 600	11 500	10 600	10 000	13 200	12 600	12 300
1	8 400	8 400	8 500	9 700	10 800	11 200	9 800	10 700	10 900	10 000	11 300	11 700
1½	8 200	8 000	7 900	9 200	10 600	10 900	9 200	9 600	9 700	8 800	10 000	10 200
2	9 200	9 800	10 000	9 900	10 900	11 300	11 400	10 000	9 400	12 200	11 400	11 000
2½	10 800	9 900	9 600	10 800	9 600	9 100	10 900	10 900	10 900	11 400	11 600	11 600
3	9 100	10 800	11 200	11 000	10 600	10 400	10 600	11 200	11 300	10 400	10 200	10 100
3½	10 700	10 900	10 800	12 000	10 600	10 000	10 600	10 900	10 900	10 200	10 600	10 600
4	10 000	8 800	8 400	11 400	9 400	9 400	8 900	10 000	10 300	8 800	10 300	10 900
4½	10 600	10 000	9 900	10 200	10 000	9 900	10 000	10 100	10 200	8 600	9 500	9 800
5	10 100	10 300	10 400	11 300	11 000	11 000	11 800	10 200	9 600	10 200	9 700	9 700
5½	10 900	11 300	11 300	11 400	10 600	10 300	13 000	11 000	10 400	11 000	9 500	9 000
6	11 500	10 800	10 500	9 500	9 000	9 000	9 500	9 900	10 000	9 700	9 500	9 500
6½	11 700	9 800	9 200	12 700	10 300	9 600	10 000	10 000	10 000	9 600	10 300	10 600
7	9 900	11 300	11 600	9 900	11 100	10 600	10 500	9 800	9 300	9 600	10 000	10 000
7½	8 000	9 700	10 300	15 400	15 400	15 400	7 800	10 000	10 600	6 800	8 500	9 000
8	10 000	9 800	9 900	11 700	10 100	9 900	8 800	10 400	11 100	9 400	9 400	9 400

TABLE 3—(*Continued*).

Panel	North Channel						South Channel					
	West Side			East Side			West Side			East Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber
	Column No. 2a, Test No. 13.											
1	4 500	7 000	8 600	7 000	8 600	9 100	9 800	10 800	11 100	10 300	12 300	13 000
2	7 600	9 000	9 500	10 300	10 400	10 600	10 900	10 100	9 700	12 800	12 600	12 600
3	7 000	8 600	9 100	9 400	10 100	10 300	11 300	11 300	11 300	11 600	12 600	13 000
4	8 100	9 100	9 500	10 400	10 800	10 900	9 300	10 400	10 700	9 600	11 600	12 500
5	8 200	9 500	10 000	10 000	10 200	10 200	10 900	8 900	8 100	10 600	10 700	10 800
6	10 000	10 400	10 600	10 500	10 000	10 000	8 500	9 600	10 000	9 600	10 400	10 800
7	9 800	9 800	9 900	10 200	10 300	10 300	9 500	8 800	8 700	9 900	10 000	10 100
8	8 000	7 800	7 700	12 300	10 300	9 200	7 700	8 700	9 100	8 700	10 000	10 500

10. *Method of Loading.*—Two methods of loading were used, central and oblique. In all cases the load was applied to the end of the column through the pins, and in a plane passing through the nominal axis of the column and paralled to the lacing. In central loading the pin was adjusted to an even bearing on the machine. In oblique loading the pin was supported on a narrow block as shown in Fig. 7 in such a way as to secure a given eccentricity. The center of the block was taken as the point of application of the load. This assumption is approximately correct, the error being probably not greater than ¼ in. In two tests of Column No. 1, the point of application of load is somewhat uncertain, as by an oversight the two bearing blocks were not placed symmetrically with respect to the axis of the column. Probably the loading was nearly central.

11. *Routine of Tests for Stress Distribution.*—In a test for stress distribution, the column was placed in the machine and a light initial load was applied. The extensometers were then attached in position to measure the deformation at some point of the column, and an initial reading was taken. A known load was applied by the testing machine and the instruments were read again. The load was then released to its initial value and another application of the load was made. If the second readings did not exactly check the first, further applications of the load were

made. In cases where the observed deformations were large or seemingly abnormal, the test was repeated at another time, and in some cases as many as ten observations were made on the same gauged length. In some of these cases the instruments were reset, their places being exchanged. The instruments were next attached in a new location, and the process was repeated. Thus the stress distribution in various parts of the column was finally determined.

The above method of changing instruments from position to position is practically necessary, as the expense of providing a sufficient number of extensometers to measure the deformation in every panel of the column would be very great.

The load generally used in the laboratory tests was 10 000 lb. per sq. in. of section of the column in excess of the initial load.

12. *Results of Tests for Stress Distribution.*—Tables 2 and 3 give results of the tests to determine stress distribution and variation in the flange members found in thirteen of the column tests. The stresses given are calculated from the observed deformation, using for the modulus of elasticity 28 000 000 lb. per sq. in. for steel and 26 000 000 lb. per sq. in. for wrought-iron, these values checking closely with the total shortening of the columns and with the average deformations observed throughout their length. As heretofore described, the stress noted is the average over a space of 4 in. or 4½ in. on either side of the point indicated. Any lack of agreement between the average stress on the center of gravity of the flange members and the average stress for the load applied is probably due principally to instrumental errors.

Fig. 8 to 15 show graphically the stress distribution and variation. The full line gives the stress at the east side (back) and the dotted line at the west side (front).

Table 4 gives a number of the most marked deviations from average stress. The excess of the maximum fiber stress is given as a percentage of the average stress.

In most cases the maximum stress was in the outer fiber of the channel; sometimes very high stresses were found in the inner fiber. Generally, the stress in the opposite channel was correspondingly less.

13. *Stress in Lattice Bars.*—Table 5 gives the results of tests to determine the average stress in the various lattice bars of the columns. Tests 14 and 15 were tests on the lattice bars only.

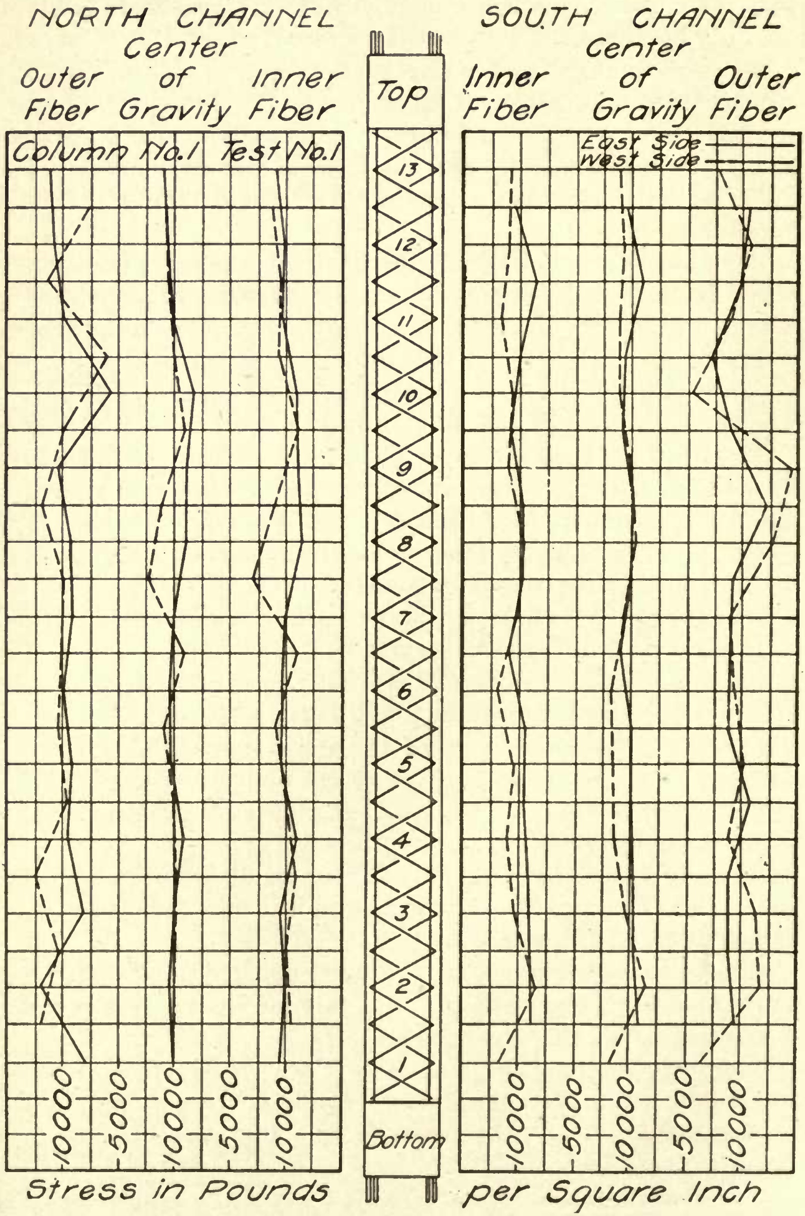

FIG. 8. STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1, TEST NO. 1.

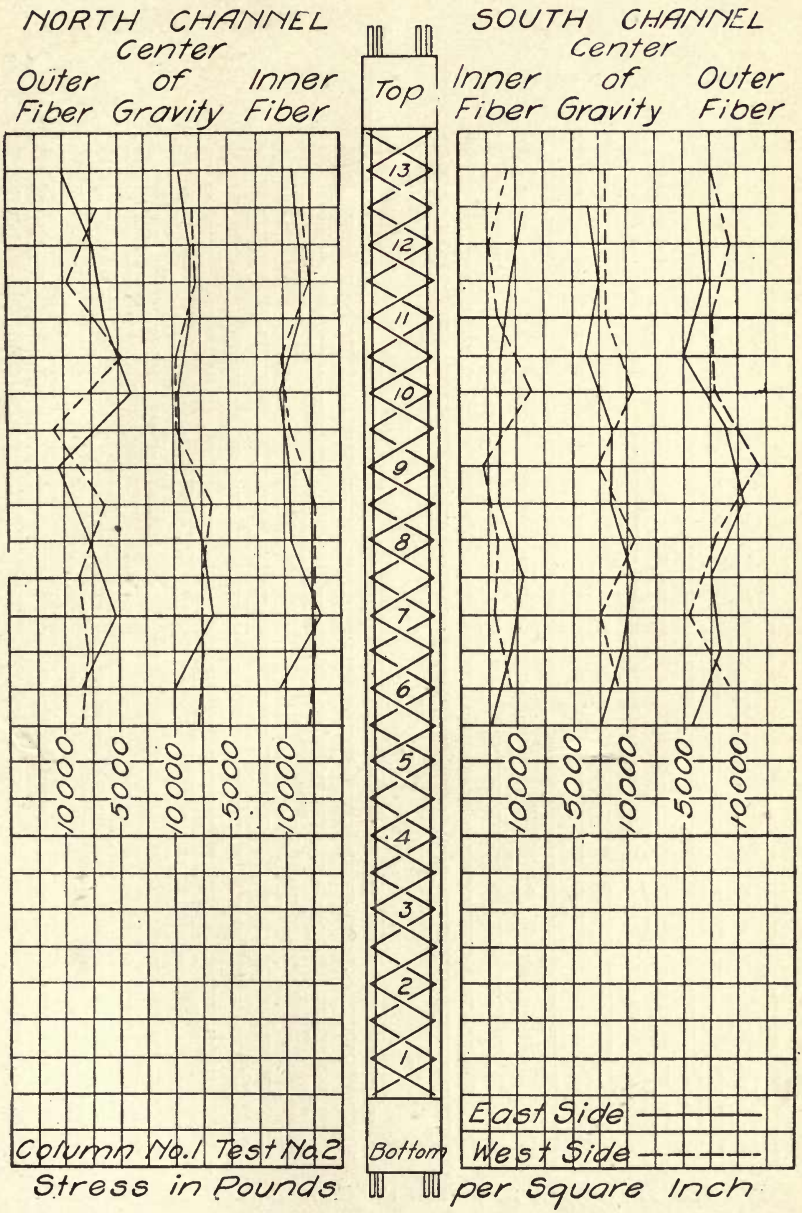

FIG. 9. STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1, TEST NO. 2.

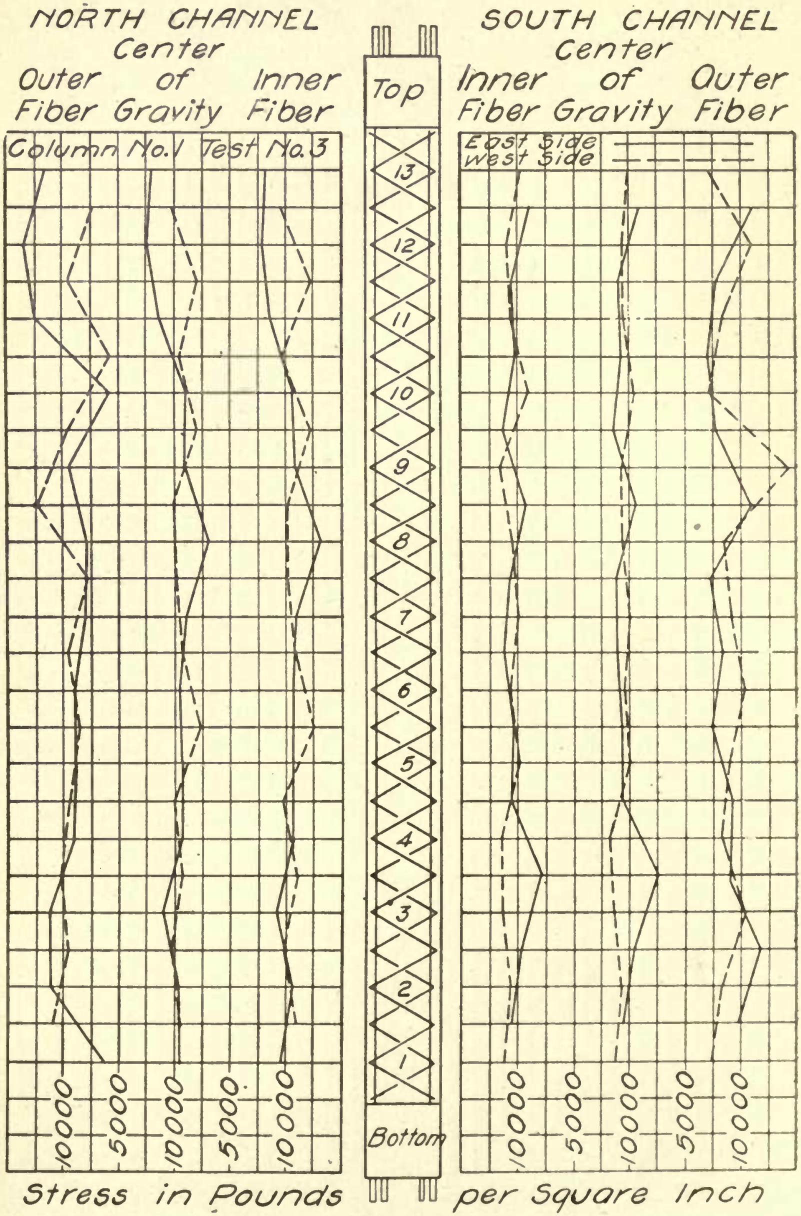

FIG. 10. STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1, TEST NO. 3.

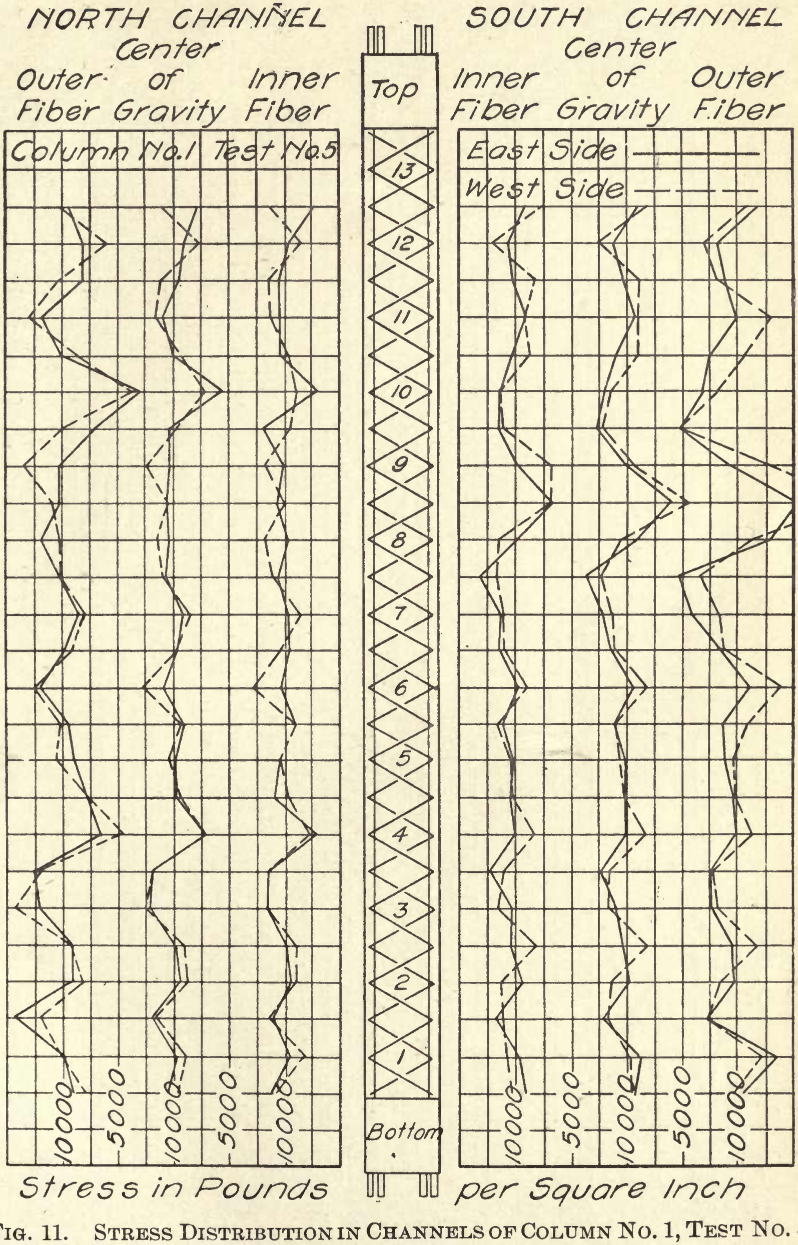

FIG. 11. STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 1, TEST NO. 5.

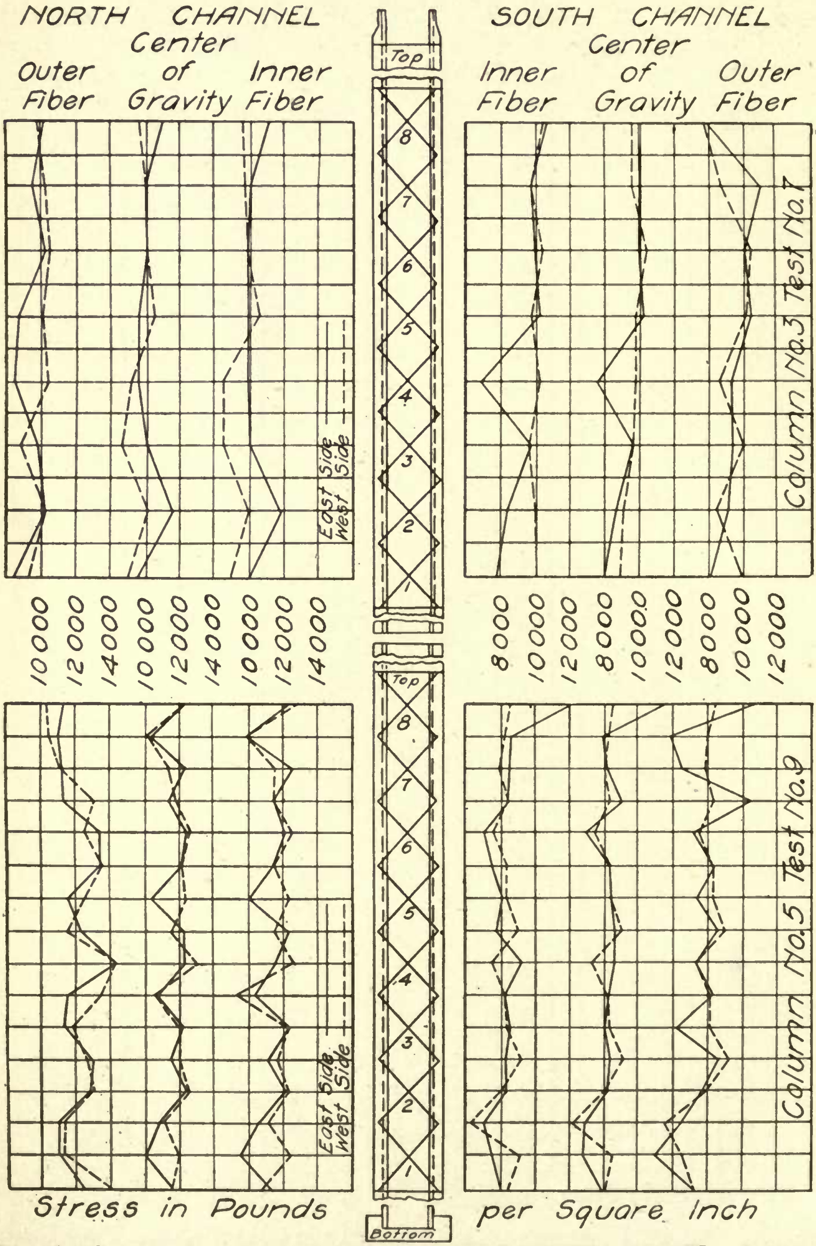

FIG. 12. STRESS DISTRIBUTION IN CHANNELS OF COLUMNS NO. 3 AND 5.

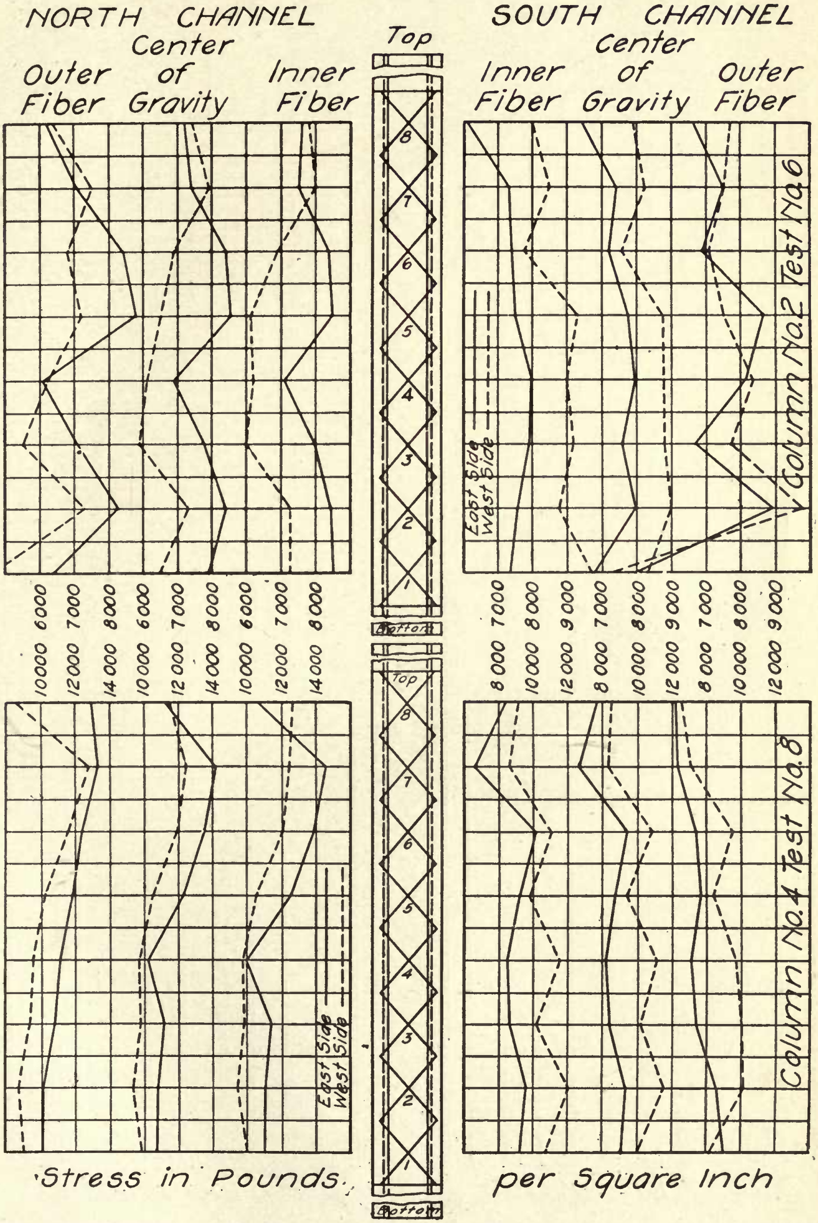

FIG. 13. STRESS DISTRIBUTION IN CHANNELS OF COLUMNS NO. 2 AND 4.

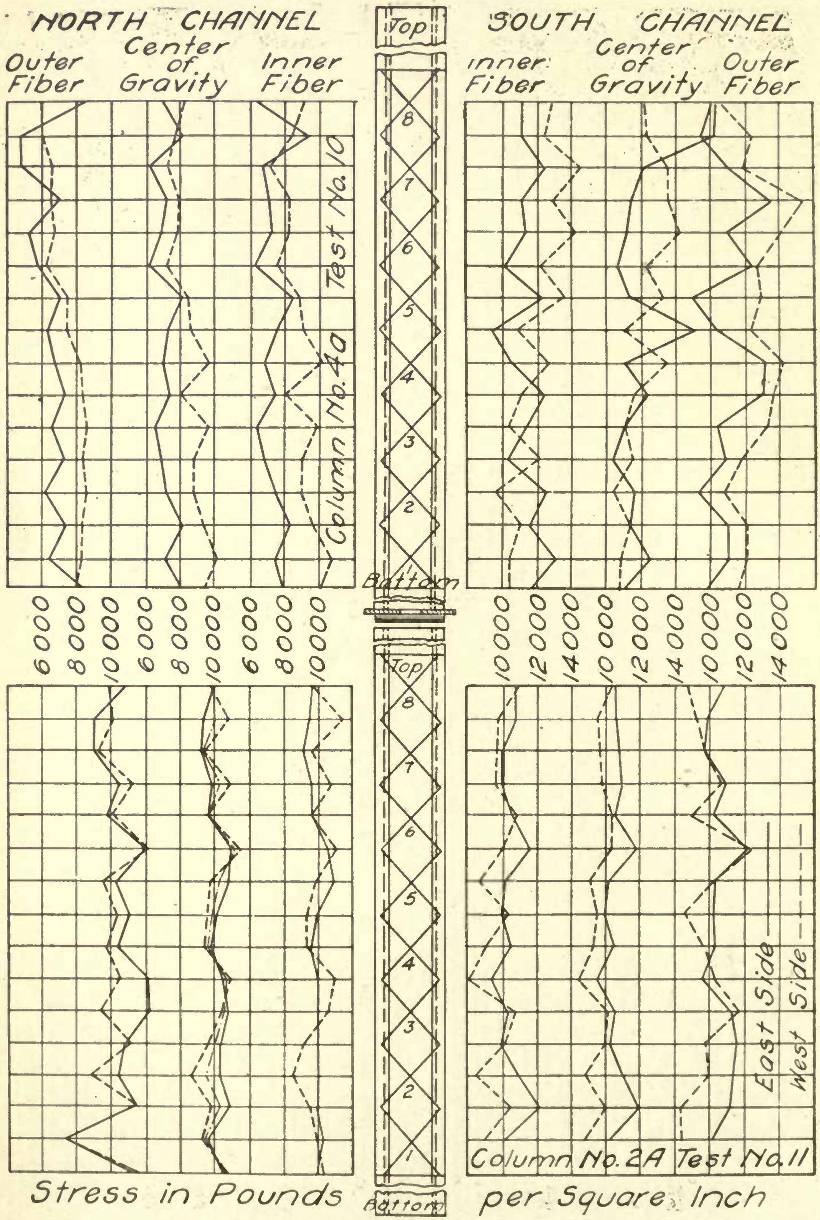

FIG. 14. STRESS DISTRIBUTION IN CHANNELS OF COLUMNS NO. 2a AND 4a.

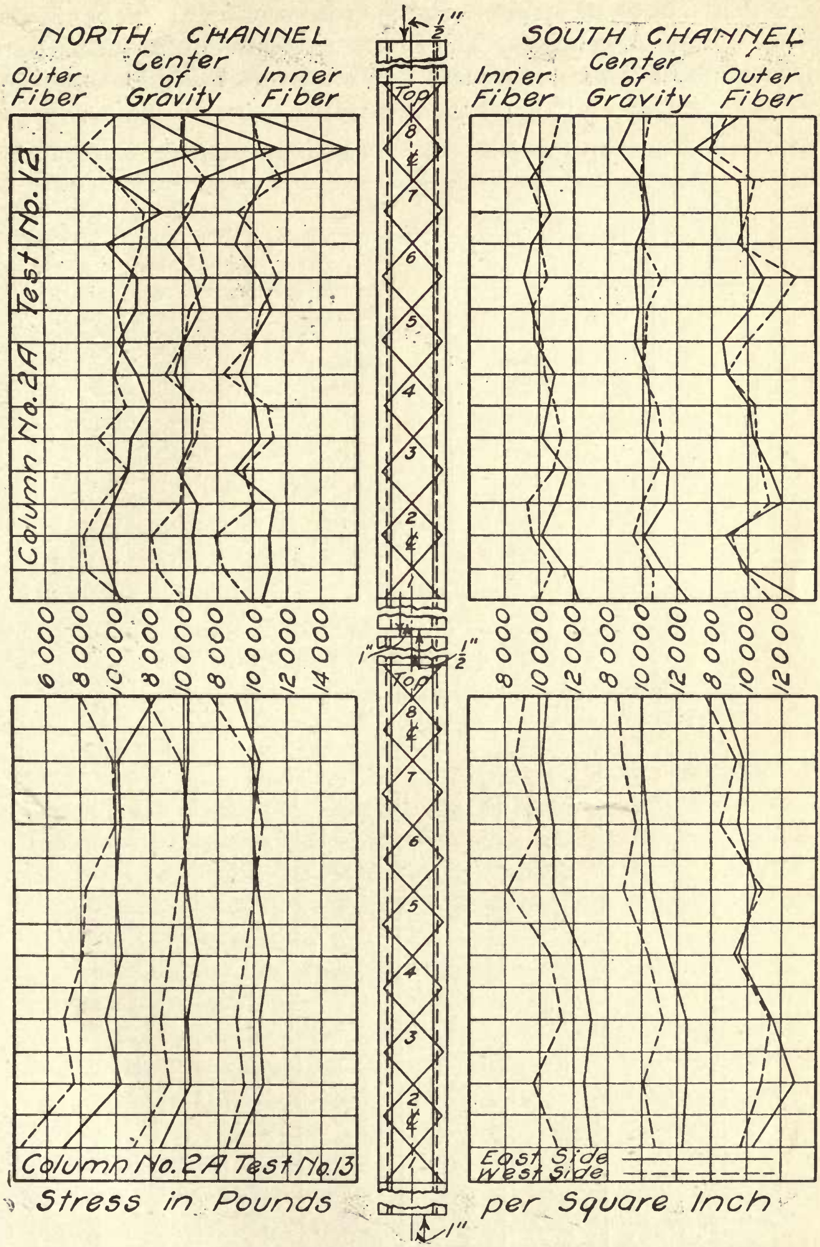

FIG. 15. STRESS DISTRIBUTION IN CHANNELS OF COLUMN NO. 2a.

The distribution of stress over the cross-section of the bar is discussed in another place. The average stresses in the lattice bars are computed from the observed deformation, using a modulus of

TABLE 4.

MAXIMUM OBSERVED FIBER STRESSES IN FLANGE MEMBERS OF COLUMNS.

Col. No.	Test No.	Feature of Lacing	Method of Loading	Excess of Maximum Fiber Stress over Average Stress per cent Highest five values
1	1	$1\frac{1}{4} \times \frac{7}{16}$ in. bolted	Central	42, 41, 39, 31, 23
1	2	"	Slightly eccentric	68, 64, 50, 49, 35
1	3	"	"	50, 41, 35, 32, 19
1	4	$1 \times \frac{1}{4}$ in. bolted	Central	35, 29, 28, 27, 26
1	5	$1 \times \frac{1}{4}$ in.	"	67, 55, 49, 29, 27
2	6		"	31, 23, 23, 21, 17
3	7		"	20, 17, 12, 11, 9
4	8		"	41, 29, 24, 22, 19
5	9		"	43, 43, 38, 35, 35
4a	10		"	53, 49, 42, 40, 37
2a	11		"	21, 16, 13, 12, 11
2a	12		Oblique (Fig. 6) Arm 1 in.	42, 42, 24, 23, 20
2a	13		Oblique (Fig. 6) Arm 2 in.	35, 34, 22, 21, 21

TABLE 5.

TOTAL STRESS IN POUNDS ON LATTICE BARS UNDER LOAD ON COLUMNS OF 10 000 LB. PER SQ. IN.

c denotes compressive stress; t, tensile stress.

Lattice Bar	Test 5		Test 14		Test 15	
	Front	Back	Front	Back	Front	Back
1	400c	400c	300t	400t	—	—
2	100t	600t	1600t	400c	—	—
3	0	600c	800c	400t	—	—
4	100t	400t	300t	100c	—	—
5	100c	100c	—	—	—	—
6	200t	300t	—	—	—	—
7	100c	200c	—	—	—	—
8	200t	200t	—	—	—	—
9	0	300c	—	—	—	—
10	0	200t	—	—	—	—
11	100c	300c	—	—	—	—
12	0	100t	—	—	—	—
13	100c	200c	800c	0	—	—
14	100t	200t	900t	900t	2500t	1600c
15	0	300c	200c	1000t	2100c	—
16	0	400t	800t	0	1600t	200c
17	200c	200c	500c	300t	1900c	—
18	200t	100t	700t	1000t	700t	1100t
19	500c	200c	500c	200t	1500c	—
20	200t	300t	800c	200t	1400t	700t
21	0	600c	800c	600t	1500c	0
22	100c	2000t	0	200t	700t	300t
23	200t	900c	300c	800c	1300c	—
24	200c	800t	200t	800t	600t	400c
25	200t	800c	0	0	2700c	—
26	200c	700t	400t	300c	400t	400c

TABLE 5—(*Continued*).

TOTAL STRESS IN POUNDS ON LATTICE BARS UNDER LOAD ON COLUMNS OF 10 000 LB. PER SQ. IN.

Lattice Bar	Front Side		Back Side	
	Under	Over	Under	Over
	Column 2a,	Test 11.	Column 2a,	Test 11.
1	700c	700c	1300c	100c
2	100c	100t	200t	200t
3	3000t	0	1000t	0
4	100c	100c	800c	200c
5	3000t	0	800c	300c
6	200t	200c	0	300c
7	3700t	0	200t	200c
8	0	800c	800c	300c
	Column 2a,	Test 12.	Column 2a,	Test 12.
1	1500c	2100t	200c	200c
2	200c	400t	2300t	200c
3	800t	200t	2750t	400c
4	700c	500t	2750t	300c
5	1600t	200t	2750t	300c
6	0	0	2800t	800c
7	1400t	800c	4100t	300c
8	100t	800c	300t	800c
	Column 2a,	Test 13.	Column 2a,	Test 13.
1	1000c	700t	800t	500c
2	500c	700t	2650t	400c
3	700c	900t	2750t	300c
4	500c	1000t	2100t	200t
5	800c	600t	3150t	1000c
6	1000c	700t	4100t	700c
7	700c	900t	1500t	400c
8	1600c	200c	2000t	1400c

elasticity of 28 000 000 lb. per sq. in. for the steel column and 26 000 000 lb. per sq. in. for the wrought-iron columns. As might be expected, from the irregular variation of stress along the flange members of the columns, the stress in the lattice bars was found to vary greatly.

Table 6 gives the largest stresses observed and the corresponding transverse shear. The transverse shear given in this table is that which would cause a stress in the lattice bars equal to the maximum stress observed in any lattice bar, and was computed by doubling the transverse component of the maximum load observed on a lattice bar. In the case of obliquely loaded columns, the transverse component of the load was computed on the assumption that the load was applied through the center of the bearing blocks. This transverse component was then subtracted from the amount of shear which has been calculated from the def-

TABLE 6.

MAXIMUM OBSERVED AVERAGE STRESS IN LATTICE BARS OF COLUMNS NO. 1 AND 2a.

Column number	1	1	1	2a	2a	2a
Test number	5	14	15	11	12	13
Method of loading	Central	Central	Oblique (Fig. 6) Arm 4 in.	Central	Oblique (Fig. 6) Arm 1 in.	Oblique (Fig. 6) Arm 2 in.
Total load, pounds	187 600	187 600	187 600	176 400	176 400	176 400
Lacing	Single 63° 30′ with axis, 1×¼ riveted	Single 63° 30′ with axis, 1¼×7/16 riveted	Single 63° 30′ with axis, 1¼×7/16 riveted	Double 45° 2½×⅜ riveted	Double 45° 2½×⅜ riveted	Double 45° 2½×⅜ riveted
Maximum observed average stress in lattice bar, lb. per sq. in	8200	3000	4900	3900	4350	4350
Corresponding total stress in bar, pounds	2000	1600	2700	3700	4100	4100
Corresponding transverse shear in column, pounds	3700	2900	4800	5200	5800	5800
Shear due to known eccentricity of load, pounds	0	0	3000	0	1000	2000
Transverse shear in column due to nominal central load, pounds	3700	2900	1800	5200	4800	3800
Ratio of shear to compression load	.020	.016	.009	.029	.027	.021
Next highest observed values of ratio of shear to compressive load	.009 .008 .008 .007	.010 .010 .009 .009	.009 .007 .006 .006	.024 .023 .010 .008	.019 .018 .018 .018	.016 .014 .014 .011

ormation of the lattice bars as before noted, and the remainder has been tabulated under the heading "Transverse Shear in Column due to Nominal Central Load".

The failure of Column No. 1 by buckling of the lattice bars, as described elsewhere, gives further information along this line. Tests to destruction under compression had previously been made on lattice bars like those used in this column, and the results, in the absence of other data, may be useful in estimating the load carried by the lattice bars at failure. Under conditions of loading similar to the conditions in the column lattice bars, these sample bars failed under an average load of 2100 lb. Assuming that the bar in this column which first failed was carrying 2100 lb. when failure occurred, the transverse shear in the column may be computed. The following tabulated statement gives the conditions of this test, and may be regarded as supplementary to Table 6.

Column No.	Compressive Load in Pounds	Lacing	Manner of Loading	Probable Maximum Load on Lattice Bar in Pounds	Corresponding Shear in Column in Pounds	Ratio of Transverse Shear to Compression Load
1	150 000	Single 63° 30′, 1 x ¼ in. riveted	Very slight obliquity	2100	3760	0.0251

14. *Tests to Failure of Wrought-iron Columns.*—After the wrought-iron bridge posts had been tested for stress distribution under working loads, they were loaded to failure. Deformations were measured in the flange members of that part of the column on which the previous test had given the heaviest stress. Table 7 gives the result of the tests to failure. For all the tests of wrought-iron bridge posts, whether loaded centrally or eccentrically, the failures were very gradual. Final failure occurred near the middle or at the end. In the former case, high stresses in one channel had been shown by the deformation measurements at working loads. In the latter case, a bending in one channel at working loads was noted by the instruments at the panel nearest the end of the column. In two of the three columns in which failure took place in the end of the column, as the instruments did not show over-stress in the laced portion of the column, the injured ends were removed, new end connections were put on, and the columns were retested as No. 2a and 4a.

15. *Tests to Failure with Column No. 1.*—Two tests to failure were obtained with Column No. 1. In the first test, the lattice

bars failed by buckling suddenly and without warning. As tested, the column was fitted with the light lattice bars (1 x $\frac{1}{4}$ in.) riveted in place. The test had in view the trial for stress distribution under a slight obliquity, which was not carefully determined. No measuring instruments were in place. A preliminary load was being applied. When the load reached 150 000 lb. (8060 lb. per sq. in. of cross-section), the alternate lattice bars in

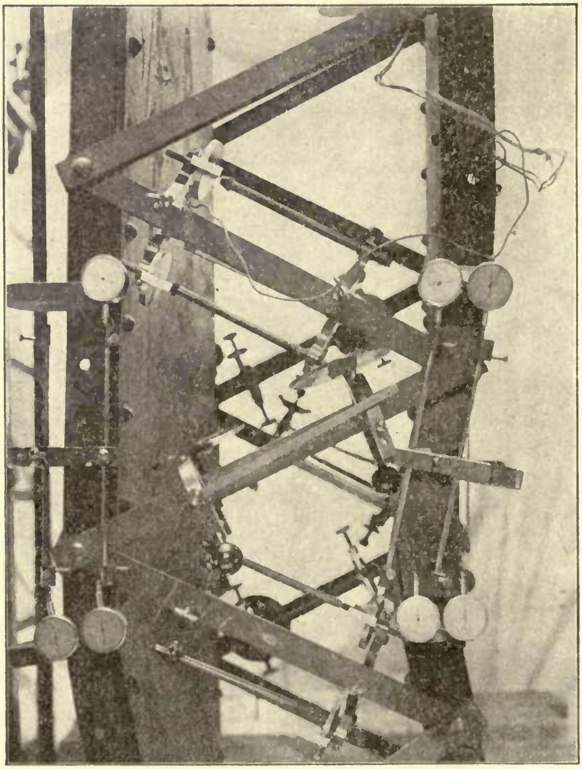

FIG. 16. COLUMN NO. 1 AFTER FAILURE.

the upper half of the column buckled. A failure of this kind was quite unexpected at such a low load. Although an observer was watching the column, the failure was so sudden that he was unable to follow the movement of the parts. In this respect it was quite in contrast to the failure of the other columns. The machine was at once stopped. Little damage was done to the column, except to the lacing bars. The webs were easily straightened, new lacing bars put on, and the column was used in another test.

TABLE 7.

RESULTS OF TESTS TO FAILURE.

Column No.	1	1	2	3	4	5	2a	4a
Test No.		17	6	7	8	9	13	10
Material	Steel	Steel	W. I.	W. I.	W. I.	W. I.	W. I.	W. I.
Method of loading	Very slight obliquity	Central	Central	Central	Central	Central	2-in. oblique (Fig. 6)	Central
Load at failure, pounds	150 000	440 000	466 000	480 000	452 000	492 000	475 000	475 000
Average stress at failure, lb. per sq. in.	8 060	23 450	26 400	27 200	25 700	27 900	26 900	26 900
Average stress at failure in tests of short pieces of flange members, lb. per sq. in.	—	—	36 800	36 800	36 800	36 800	36 800	36 800
Percentage of excess of ultimate strength of short pieces over column strength	—	—	39.5	35.5	43.5	32.0	37.0	37.0
Load at first sign of yielding, pounds	150 000	225 100	317 000	264 000	256 000	264 000	—	—
Average stress at first sign of yielding, lb. per sq. in.	8 060	12 000	18 000	15 000	14 500	15 000	—	—
Method of failure and remarks	Very sudden buckling of lacing bars in upper half	Sudden buckling of flange between adjacent lacing bars	End failed	End failed	End failed	Bowed at middle in plane perpendicular to lacing	Buckled near bottom in plane parallel to lacing	Bowed at middle in plane perpendicular to lacing

The column having been riveted up with heavier lattice bars ($1\frac{1}{4}$ x $\frac{7}{16}$ in.), it was next subjected to several tests for stress distribution and was finally loaded to failure with a central load. Measuring instruments were attached to flanges and lattice bars near that part of the column in which, from the results of previous tests, the greatest stress was expected. Fig. 16 shows the attachment of instruments to the columns. Failure occurred un-

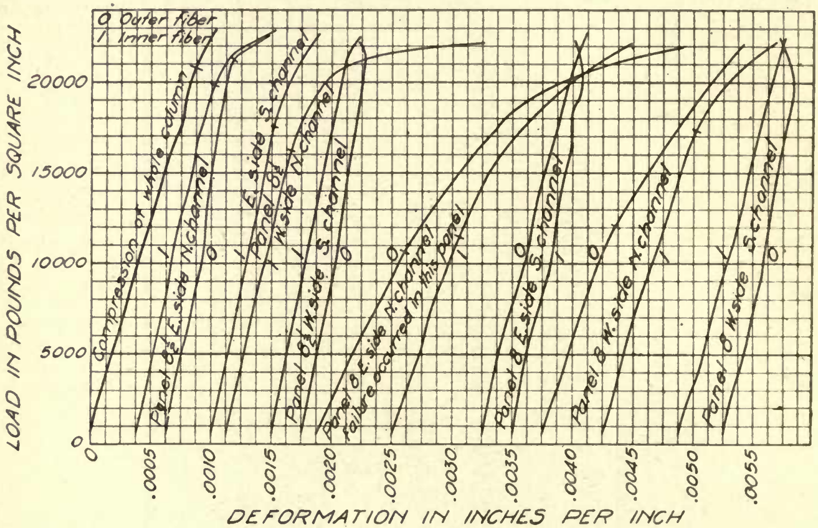

FIG. 17. LOAD-DEFORMATION CURVES OF TESTS TO FAILURE OF COLUMN No. 1.

der a load of 440 000 lb. (23 450 lb. per sq. in. of section); it was caused by the local buckling or "wrinkling" of the north flange in panel 8, the panel in which the greatest stress had been found in Test No. 5. The failure of the column was slower than that of the preceding test in which the lattice bars buckled, but it was much more sudden than were the failures of the wrought-iron columns. One lattice bar on each side was buckled by the crippling of the channel member. The measuring instruments attached to the web at panel 8 showed from the first of the test that there was a very large stress at that point.

Fig. 17 shows the stress-deformation curve as taken at various points. The uneven distribution of stress is clearly shown, and the first sign of approaching failure is seen at about 12 000 lb. per sq. in. Fig. 16 shows the column after failure.

The low average stress at failure in Column No. 1 should be noted, and also the manner of failure. There is a sharp contrast between the gradual bowing of the stocky columns tested and the sudden wrinkling collapse of the flimsy steel column.

16. *Cross-bending Test of Columns.*—Cross-bending tests were made on one of the wrought-iron columns and on Column No. 1. The tests were made in an Olsen 200 000-lb. testing machine fitted for testing beams 20 ft. long. The columns were supported at

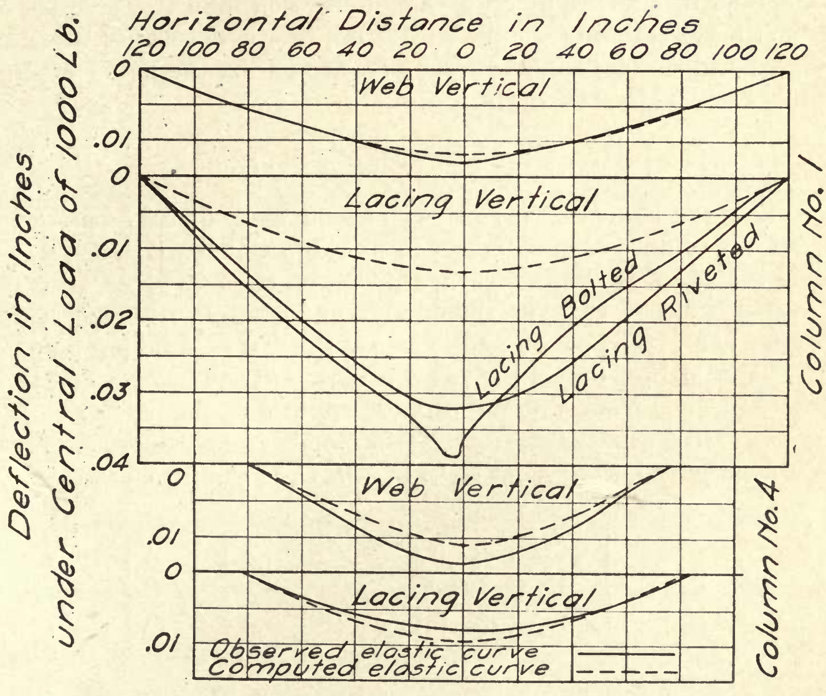

FIG. 18. DEFLECTION OF COLUMNS UNDER CROSS BENDING.

the ends and loaded at the center with a light transverse load. The column was placed first with the plane of the lacing perpendicular to the load, and then with the plane of the lacing parallel to the load. The lattice bars used in the tests of Column No. 1 were $1\frac{1}{4}$ x $\frac{7}{16}$ in. in cross-section; in one test they were bolted in place and in another they were riveted. The deflection at various points along the beam was measured with Ames test gauges, and the actual curve assumed by the column under transverse load was thus determined. The theoretical elastic curve was computed from the common theory of flexure, not counting the lattice bars in the calculation of the moment of inertia. Fig. 6 (p. 13) shows the deflectometers and extensometers on Column No. 1 under the cross-bending test. Fig. 18 shows the deflection curves given by the columns under transverse load and also the computed elastic curves.

It will be noted that when tested with the lacing vertical, Column No. 1 shows much greater deflection than that computed from the usual beam formula, while the stiffer wrought-iron column shows a much closer agreement with the curve, the heavy lacing apparently adding stiffness.

III. Field Tests of Columns.

17. *Description of Bridge.*—The field tests of columns were made on compression members in a bridge which spans the Sangamon river near White Heath, Illinois, on the line of the Illinois Central Railroad between Champaign and Clinton, Illinois. This bridge is an eight-panel, single-track, Pratt truss, having a span of 158 ft. 6 in. Fig. 19 gives a diagram of the bridge, and the frontispiece is from a photograph of the bridge under test.

18. *Members Investigated.*—The members in which stresses were measured were Post U_2L_2 South, U_3L_3 South, U_3L_3 North,

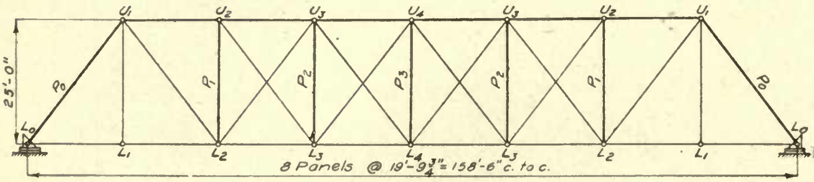

FIG 19. DIAGRAM OF WHITE HEATH BRIDGE.

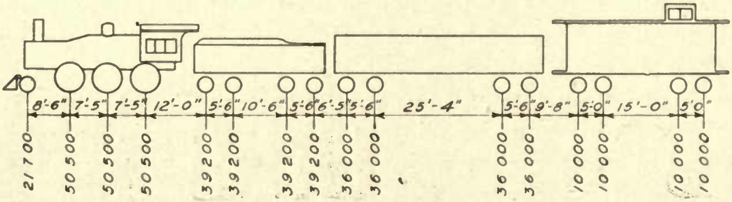

FIG. 20 DIAGRAM OF TEST TRAIN.

and Upper Chord U_3U_4 South. The location of these members is shown in the bridge diagram, Fig. 19. The upper chord was made up of two built-up channels with a cover plate on top and double lacing across the bottom. The end of each upper chord was riveted to a connection plate to which was riveted the adjacent end of the next upper chord and also the post under the junction of the chords. The posts were made up of two steel channels

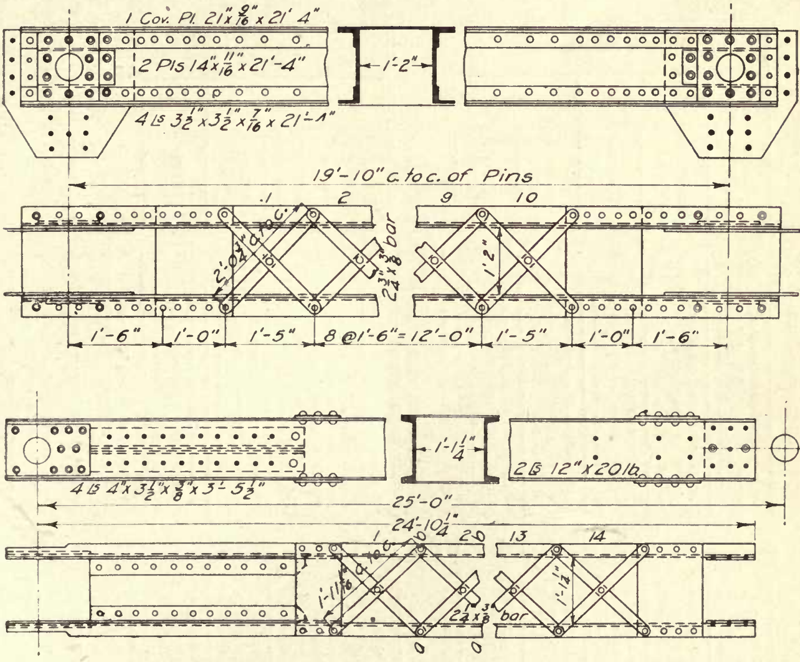

FIG. 21. UPPER CHORD AND POST OF WHITE HEATH BRIDGE.

double laced, and the floor beams were directly riveted to the posts. The upper ends were riveted to a connection plate as noted above and the lower ends carried pins to which the lower chord members (eyebars) were attached. Fig. 21 shows in detail a post and upper chord.

19. *Application of Load.*—The test load applied to the bridge consisted of a mogul locomotive and tender, I. C. R. R. No. 555, followed by a loaded coal car and a caboose. Fig. 20 shows the test train, with dimensions and weights. This train was furnished through the courtesy of the railroad company.

20. *Measurement of Deformation.*—Ames test gauges were used as extensometers, and the method of attachment was the same as in the laboratory tests of columns. The method of reduction of instrument readings to stresses at the extreme fibers of members was also the same.

TABLE 8.

STRESSES IN POSTS OF WHITE HEATH BRIDGE.

Panel	North Channel						South Channel					
	East Side			West Side			East Side			West Side		
	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber	Outer Fiber	Center of Gravity	Inner Fiber
Post U_3L_3 South.												
1*	5 000	5 300	5 400	2 600	2 700	2 700	4 900	5 000	5 000	2 300	2 500	2 600
2	3 400	3 500	3 500	1 900	1 600	1 500	3 300	3 400	3 400	1 600	1 800	1 800
3	3 500	3 800	4 000	1 800	1 900	2 000	2 600	2 900	3 100	2 600	2 200	2 100
4	3 700	3 400	3 300	2 800	3 000	3 000	1 600	3 100	3 600	2 400	2 800	2 900
5	3 500	3 900	4 000	2 900	2 900	2 900	2 800	3 000	3 100	2 200	2 600	2 800
6	3 600	3 600	3 600	3 400	3 800	3 900	3 400	3 400	3 400	4 100	3 400	3 200
7	2 600	3 200	3 400	3 300	3 100	3 000	2 400	3 000	3 300	2 600	2 700	2 800
8	3 300	3 200	3 200	3 900	3 600	3 500	3 600	3 400	3 300	3 700	3 900	3 900
9	3 200	3 100	3 000	3 600	3 300	3 200	1 700	2 500	2 800	2 900	3 300	3 400
10	2 800	2 700	2 700	3 100	3 300	3 300	2 700	2 400	2 300	3 300	3 700	3 900
11	3 300	2 900	2 800	4 500	3 400	3 100	2 900	2 000	1 700	3 500	3 900	4 100
12	1 400	2 000	2 300	3 100	3 200	3 200	1 800	1 800	1 800	4 800	4 700	4 600
13	1 300	1 500	1 500	2 800	2 700	2 600	2 600	2 900	3 100	5 200	4 900	4 800
Post U_3L_3 North.												
1	4 900	4 900	4 900	2 600	2 500	2 500	4 800	4 900	4 900	2 100	2 700	2 800
1½	4 800	4 700	4 700	2 900	2 800	2 700	3 100	4 900	5 700	900	1 500	1 700
2	4 800	4 400	4 300	2 600	2 500	2 500	4 900	4 700	4 600	2 700	1 700	1 300
2½	4 000	3 900	3 900	2 700	2 800	2 900	4 300	3 700	3 500	2 300	1 800	1 600
3	3 900	4 200	4 300	2 800	2 900	2 900	......	......	......	......	......	......
3½	3 900	3 900	4 000	2 400	2 500	2 500	3 200	3 800	4 100	2 400	2 500	2 500
4	3 400	4 600	5 000	2 700	2 700	2 700	3 900	3 700	3 700	3 500	3 200	3 100
4½	3 800	3 400	3 200	2 500	2 300	2 200	3 500	3 500	3 400	3 100	3 000	3 000
5	3 900	3 700	3 600	3 700	3 700	3 700	3 100	3 600	3 800	1 400	2 100	2 400
5½	4 100	3 600	3 400	4 000	3 400	3 200	3 500	3 900	4 100	3 500	2 900	2 800
6	3 600	3 200	3 100	3 400	3 300	3 200	3 700	3 500	3 400	3 900	3 600	3 500
6½	3 300	3 300	3 300	3 600	3 500	3 500	4 500	4 700	4 800	2 000	2 400	2 600

*No explanation for the high values in Panel 1 has been found. Five determinations of stress were made, including the removal and re-attachment of instruments.

21. *Routine of Tests.*—As in the laboratory tests, the stress distribution in the channels and the lattice bars was studied. The method of procedure was as follows. Instruments were placed on some portion of the column to measure the deformation over a short gauge length, and a reading was taken. The test train was then run upon the bridge to a given position (one approxi-

mating the maximum load on the member under test), and the instruments were read again. The train was then run off the bridge, and the instruments were again read. This procedure was repeated several times, at least three applications of the load being made and frequently several more. The instruments were then moved to another part of the column, and that part was tested. Observations were made on both flange members and lattice bars. The tests covered a period of eight days. The weather was ideal with the exception of one day.

TABLE 9.

STRESSES IN UPPER CHORD OF WHITE HEATH BRIDGE.

Stresses are given in lb. per sq. in.

Panel	Lower Side (Laced).				Upper Side (Cover-plate)				
	North Channel		South Channel		Distance from End inches	North Edge	Over North Web Plate	Over South Web Plate	South Edge
	Outer Fiber	Inner Fiber	Outer Fiber	Inner Fiber					
1	4 300	3 900	5 000	5 500	33	5 500	6 000	5 200	5 700
2	4 900	4 300	4 500	4 800	57	....	....	5 200	5 500
3	2 800	5 500	3 400	4 500	81	5 200	5 400	5 900	5 300
4	2 800	5 100	4 200	5 300	105	5 000	6 100	5 700	6 000
5	5 700	5 400	3 900	4 800	129	4 800	5 900	5 900	5 900
6	5 500	4 800	5 100	4 200	153	5 400	5 700	5 500	5 600
7	5 900	4 700	3 400	3 600	177	5 000	5 000	5 700	5 600
8	6 200	5 300	3 300	4 400	....	....	....	....	....
9	6 000	5 300	4 700	3 800	....	....	....	....	....
10	5 900	5 100	4 400	4 900	....	....	....	....	....

22. *Results of Tests for Stress Distribution in Channels.*—Table 8 gives the results of the tests to determine the stress distribution and variation in the channels of the bridge posts, and Table 9 gives those for the top chord. The stresses given were calculated from the observed deformations, using a modulus of elasticity of 30 000 000 lb. per sq. in. The conditions of measurement of deformation were much the same as in the laboratory tests. The stress noted is the average stress over a space of 4½ in. on either side of the point indicated.

Fig. 22 and 23 show graphically the stress distribution and variations. In these figures the full lines give the stresses at the west side (front), and the dotted lines the stresses at the east side (back).

In Table 10 (p. 44) are given a number of the highest observed fiber stresses. The excess of the maximum fiber stress is given

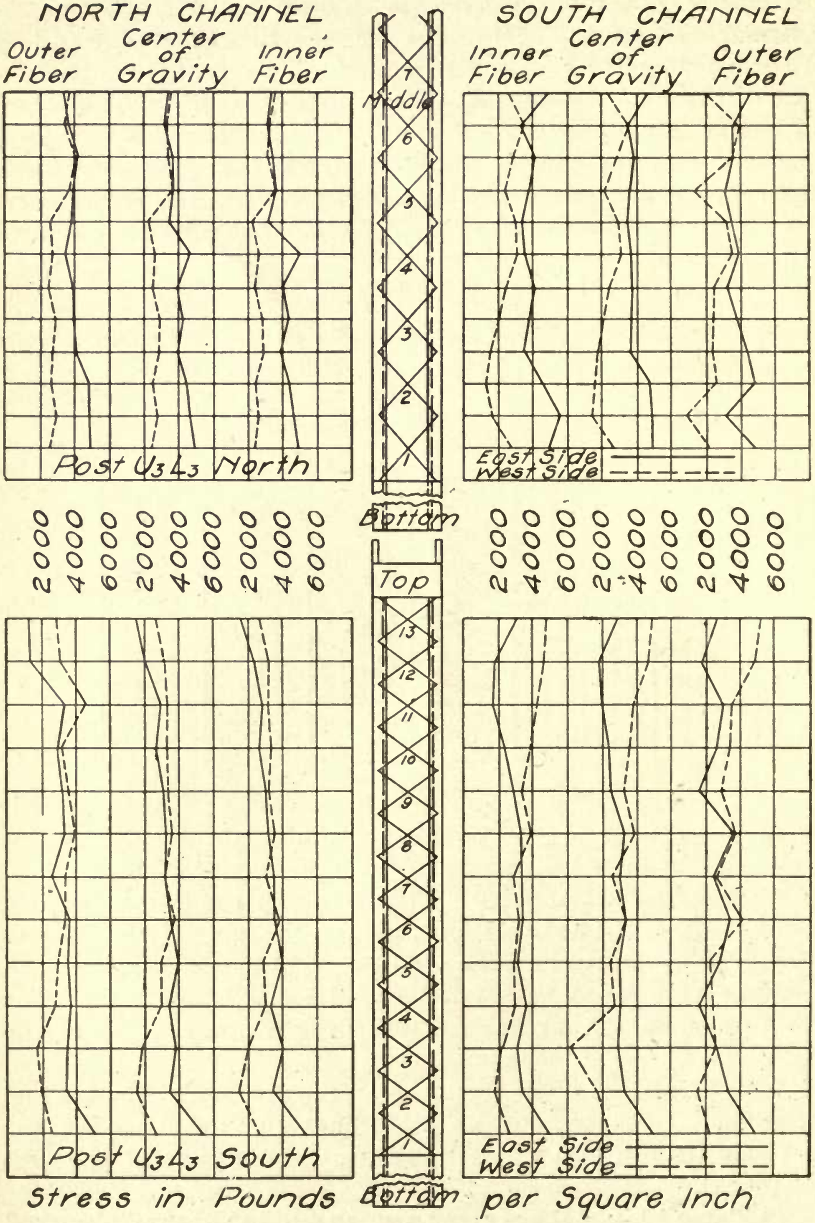

FIG. 22. STRESS DISTRIBUTION IN POST U_3L_3 NORTH AND U_3L_3 SOUTH OF WHITE HEATH BRIDGE.

as a percentage of the average stress. At most sections the maximum stress was in the outer fiber of the channel, but in some cases it was found at the inner fiber.

In the tests of the bridge posts an attempt was made to determine the stresses in a few of the lattice bars. These stresses were very small, and the precision of the extensometer was not sufficient to measure them with any great degree of accuracy.

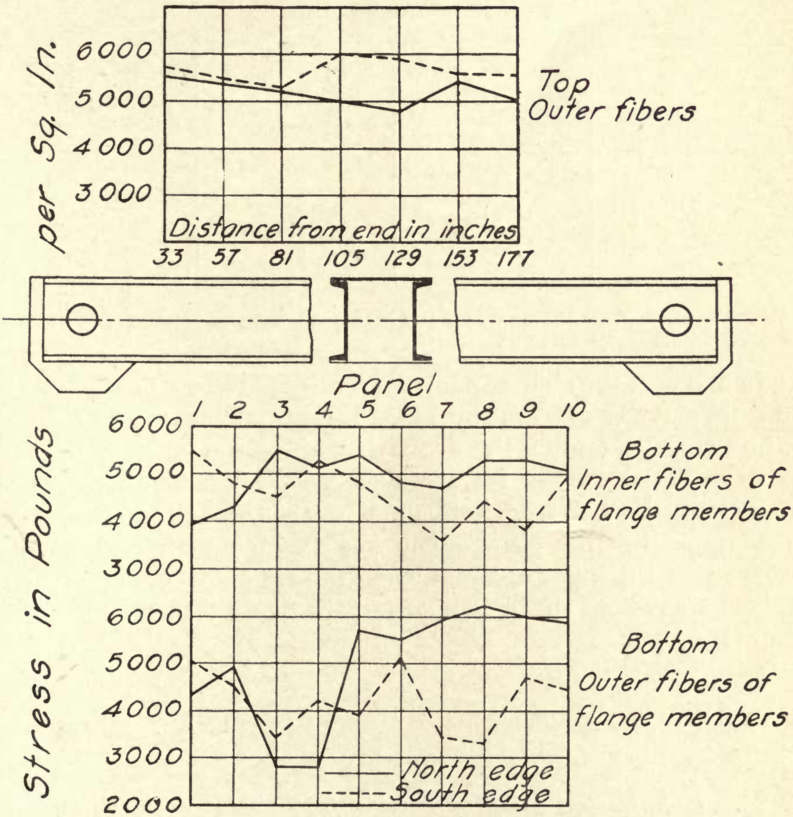

FIG. 23. STRESS DISTRIBUTION IN UPPER CHORD U_3U_4 OF WHITE HEATH BRIDGE.

It should be noted that the lacing of the posts in this bridge was double, and the bars were riveted together at their intersection. In several cases it was found that a lattice bar under load bent in the shape of a very flat S-curve, the point of attachment to another lacing bar, at the middle, being a point of inflection.

23. *Special Tests on Bridge Columns.*—Tests were made on the batten plates at the top of one of the posts, and under load a slight bending of the plates between channels was found. The bending took place in a horizontal plane.

TABLE 10.

MAXIMUM OBSERVED FIBER STRESS IN FLANGE MEMBERS OF COLUMNS IN WHITE HEATH BRIDGE.

Column number...........	$U_3 L_3$ South	$U_3 L_3$ North	$U_3 U_4$ South
Test number......	F 1	F 2	F 3
Lacing......................	Double 45° riveted at crossing	Double 45° riveted at crossing	Cover plate on top. Double 45° on bottom
Maximum observed compressive stress in an extreme fiber, lb. per sq. in.	5200	5700	6200
Excess maximum observed stress over average, per cent. Highest five values......	73*	64	20
	63	48	19
	60	41	17
	57	31	17
	47	31	17

* No values from Panel 1 have been included in this table as no explanation is known of the high stresses indicated in that panel.

In one post the change of stress was observed as the locomotive and train moved across the bridge. Extensometers were placed at aa, bb (Fig. 21), and on the floor beams, and the changes in reading were noted as the train moved across the bridge. In the inner channel of the post, tension was set up as the locomotive came opposite the post. The maximum amount of tension observed in the inner channel when the locomotive was opposite the post was about three-quarters of the compression in the outer channel.

IV. TESTS OF LATTICE BARS, SMALL COLUMNS, AND COLUMN MATERIAL.

24. *Compression Tests of Lattice Bars.*—Many of the lattice bars in a column, as they transmit stress from one flange member of the column to the other, are under compression. To study the action of lattice bars under compression, a series of tests on single lattice bars was made. Fig. 24 shows the arrangement of the apparatus. The lattice bar was tightly bolted to the blocks, B_1 and B_2. The upper block, B_1, was fastened to the cross-head of a testing machine, and the block B_2 was pressed against the

weighing table of the testing machine. A spherical-seated bearing block was used, to insure an even bearing. Ames test gauges, E, mounted on suitable frames, were attached to the lattice bar over a short gauged length. From the readings of these gauges, the deformation of the extreme fiber of the bar was computed.

In this test of lattice bars, the load was applied with an eccentricity approaching that to be expected in a column for the lattice bars which are next to the flange member (here designated "under" bars). The lattice bars outside of these "under" bars are here designated "over" bars. The stress distribution across the section of the "over" bars, which are under compression, is probably more uneven than the stress distribution found in these tests. However, these tests give some idea of the relative behavior of lattice bars of various proportions, and of the large eccentricity of loading of all lattice bars.

Lattice bars of the following cross-sections were tested: Flat bars $1\frac{1}{2}$ x $\frac{1}{4}$ in., 1 x $\frac{3}{8}$ in., $\frac{7}{8}$ x $\frac{7}{16}$ in.; angles $1\frac{1}{2}$ x $1\frac{1}{2}$ x $\frac{1}{8}$ in.; channels $1\frac{1}{2}$ x $\frac{3}{4}$ x $\frac{1}{8}$ in. Several channel and angle lattice bars were tested with ends flattened and ribs turned inward, to minimize the eccentricity of loading. Bars of the following lengths between centers of rivet holes were tested: $8\frac{1}{2}$ in., $13\frac{1}{2}$ in., and 20 in. The rivet holes were $\frac{1}{2}$ in. in diameter. All bars were tested in a Philadelphia Machine Tool Company's 100 000-lb. testing machine, and loads and extensometer readings were taken to failure.

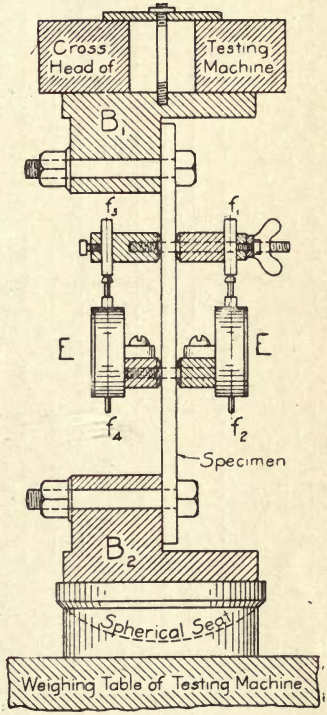

FIG. 24. ARRANGEMENT OF APPARATUS IN COMPRESSION TESTS OF LATTICE BARS.

Observations were also made on the behavior of a lattice bar in a column under load, with a view to determine the distribution of stress over the section. For this purpose Column No. 1 was loaded obliquely. The instruments were placed on an "over" bar which had been found to carry a high compressive stress, and readings were taken to determine the distribution of stress across the section.

When Column No. 1 was under cross-bending test, observations were made to determine the stresses transmitted by lattice bars and their distribution over the section of the bars. Extensometers were placed successively on most bars under compression on one-half of the column, and on some bars which were under tension. In both of these tests the bars were $1\frac{1}{4}$ x $\frac{7}{16}$ in., and were riveted.

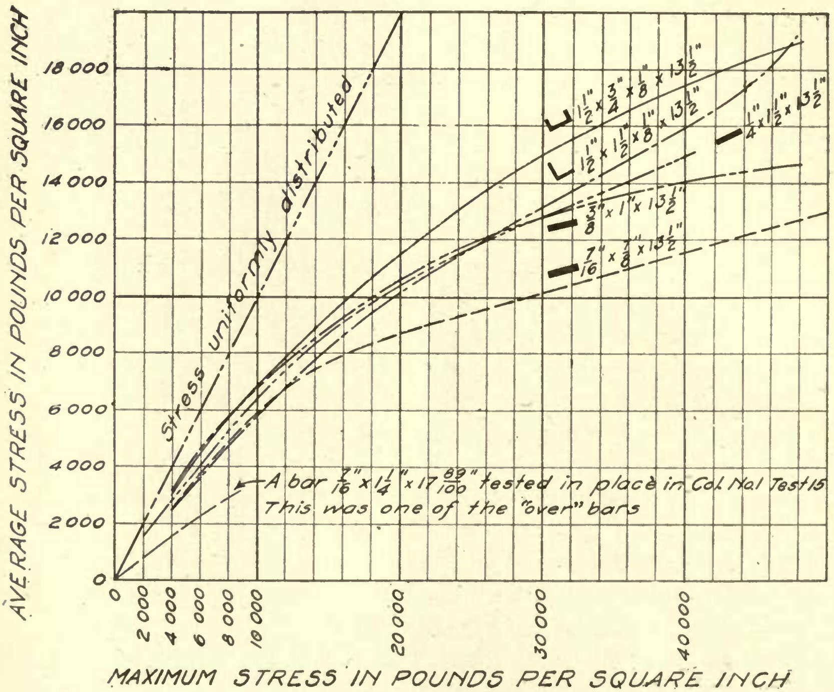

FIG. 25. FIBER STRESS IN COMPRESSION TESTS OF LATTICE BARS, ENDS HELD AS IN FIG. 24.

25. *Results of Tests of Lattice Bars.*—The results of the tests of single lattice bars are given in Fig. 25 and 26, and in Tables 11 and 12. Fig. 25 shows the ratio of maximum to average stress in the bars $13\frac{1}{2}$ in. long between centers of rivet holes. It also gives the result of the test of stress distribution in a lattice bar of Column No. 1. Table 11 gives the stresses at failure of the various bars tested singly. The average stress on the various

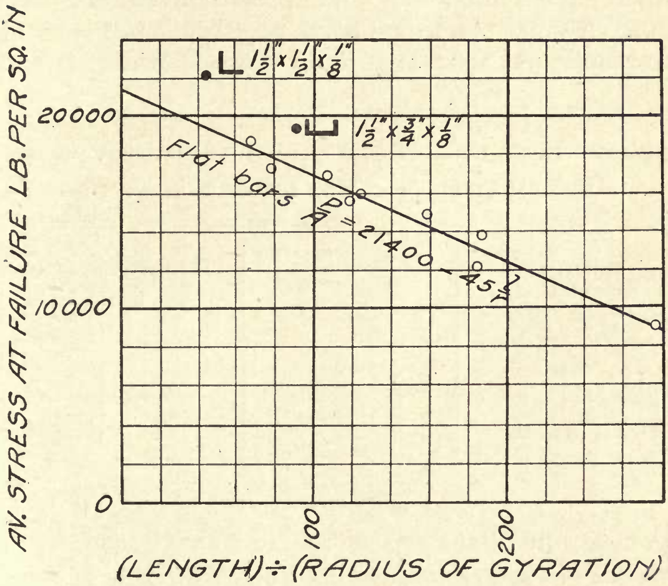

FIG. 26. RESULTS OF COMPRESSION TESTS TO FAILURE OF LATTICE BARS.

TABLE 11.

COMPRESSION TESTS OF LATTICE BARS.

Average of two specimens.

Section of Bar inches	Distance c. to c. of Rivet Holes inches	$\frac{l}{r}$	Stress at Failure lb. per sq. in.	Stress at Failure for Steel of 40 000 lb. per sq. in. Yield Point* lb. per sq. in.
1¼ x ¼ flat	20	277	9 900	8 900
1 x ⅜ flat	20	184	12 900	12 200
⅞ x 7/16 flat	20	158	14 500	15 000
1½ x ¾ x ⅛ channel	20	90	20 800	19 400
1½ x 1½ x ⅛ angle	20	43	22 600	20 100
1½ x ¼ flat	13½	187	15 400	13 800
⅞ x 7/16 flat	13½	107	16 300	16 800
1 x ⅜ flat	13½	124	16 900	15 900
1½ x ¾ x ⅛ channel	13½	61	bolt sheared	......
1½ x 1½ x ⅛ angle	13½	29	bolt sheared	......
1½ x ¼ flat	8½	118	17 300	15 500
⅞ x 7/16 flat	8½	67	18 100	18 700
1 x ⅜ flat	8½	78	18 300	17 200
1½ x ¾ x ⅛ channel	8½	38	bolt sheared	......
1½ x 1½ x ⅛ angle	8½	18	bolt sheared	......

*The values in this column were obtained by multiplying the observed stress at failure by $\left(\frac{40\ 000}{\text{yield point determined from tests}}\right)$.

bars, which gave a maximum fiber stress of 40 000 lb. per sq. in., as taken from these tests, has been noted and is given in Table 12. The results of the tests to failure are shown graphically in

TABLE 12.

AVERAGE STRESS IN LATTICE BARS WHICH CORRESPONDS TO A MAXIMUM FIBER STRESS OF 40 000 LB. PER SQ. IN.

Section of Bar inches	Distance c. to c. of Rivet Holes inches	Corresponding Average Stress lb. per sq. in.
$\frac{7}{8}$ x $\frac{7}{16}$ flat	$13\frac{1}{2}$	11 600
1 x $\frac{3}{8}$ flat	$13\frac{1}{2}$	14 000
$1\frac{1}{2}$ x $\frac{1}{4}$ flat	$13\frac{1}{2}$	14 900
$1\frac{1}{2}$ x $1\frac{1}{2}$ x $\frac{1}{8}$ angle	$13\frac{1}{2}$	15 900
$1\frac{1}{2}$ x $\frac{3}{4}$ x $\frac{1}{8}$ channel	$13\frac{1}{2}$	17 500

TABLE 13.

STRESS IN LATTICE BARS OF COLUMN NO. 1 UNDER CROSS BENDING.

Column tested as a beam centrally loaded over span of 19 ft. 8 in.; lacing bars $1\frac{1}{4}$ x $\frac{7}{16}$ in.; 17.89 inches c. to c. of rivet holes; rivets $\frac{1}{2}$-in. in diameter.

Bar	Maximum Fiber Stress from Ames Dials	Average Stress from Ewing Extensometer	Ratio Maximum to Average
	"Over" bars in compression		
14 E	6000	2600	2.31
16 E	8300	2000	4.15
18 E	4300	1800	2.39
20 E	7700	1400	5.50
22 E	7000	2900	2.42
24 E	8900	2200	4.04
			Av. 3.47
	"Under" bars		
15 W	4300 c*	2500 c	1.72
25 E	3800 t*	3100 t	1.23
17 E	3900 t	3000 t	1.30
25 W	3900 c	2800 c	1.39
19 W	4500 c	2900 c	1.55
			Av. 1.44
	"Over" bars in tension.		
16 W	7000	3000	2.33
18 W	7000	2200	3.19
20 W	4500	2400	1.87
22 W	5700	2200	2.59
24 W	7000	2700	2.59
26 W	3800	2200	1.73
			Av. 2.38

* c, bar in compression; t, bar in tension.

Fig. 26. The angle and channel bars tested with flattened ends failed in the flattened part at loads no greater than similar bars not flattened at the ends.

Table 13 gives the results of the test for stress distribution in the lattice bars of Column No. 1 as it was stressed in cross bending.

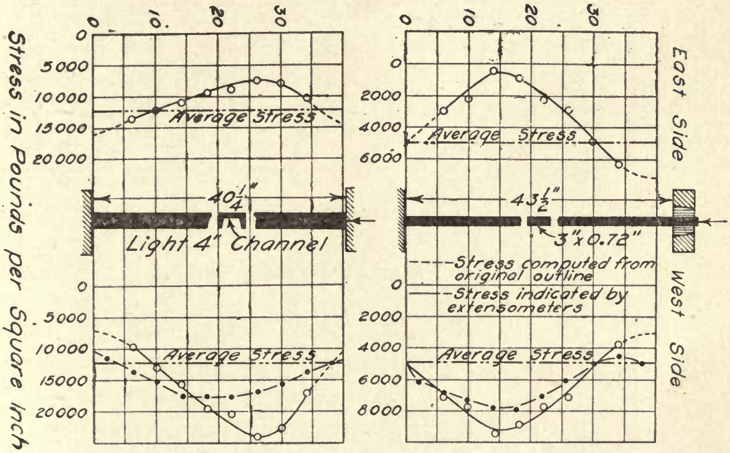

FIG. 27. STRESS DISTRIBUTION IN SMALL COLUMNS.

26. *Tests of Two Small Compression Pieces.*—Tests of two small compression pieces were made in order to study the effect of slight bends and kinks in the column upon the distribution of stress. The deviation from a straight line, in these nominally straight pieces, was measured before the load was applied. The deformations on two opposite faces for a given load were measured. The extensometer was similar to that used on the single lattice bar tests. The instrument was shifted from one position to another along the column. The columns were finally loaded to failure. One of the columns was a flat piece of steel, 3 x 0.72 in. in cross-section, and 46 in. long. It was held at the upper end by wedge grips in the cross-head of the machine and at its lower end rested on a spherical-seated block. The second compression piece was a 4-in. channel 40 in. long. The ends were planed square; the upper end bore on a flat compression plate in the iron head of the machine, and the lower end rested on a spherical-seated block.

TABLE 14.

TENSION TESTS OF MATERIAL FROM COLUMNS.

Test Piece from	Material	Number of Test Pieces	Average Stress at Yield Point, lb. per sq. in.	Average Stress at Ultimate, lb. per sq. in.	Elongation in 2 inches, per cent
Col. No. 1 Angles	Steel	2	43 300	61 600	37
7/16 x 1¼ lacing bar	Steel	2	40 700	58 100	42
1 x ¼ lacing bar	Steel	1	42 400	62 000	38
Channels of wrought-iron columns	Wrought-iron	3	30 700	46 800	17

TABLE 15.

TENSION TESTS OF LATTICE BARS.

Shape	Dimensions of Cross-section of Whole Bar inches	Number of Specimens Tested	Average Stress at Yield Point, lb. per sq. in.	Average Stress at Ultimate, lb. per sq. in.	Elongation in 2 inches per cent
Channel........	1½ x ¾ x ⅛	1	43 000	57 600	36.5
Angle...........	1½ x 1½ x ⅛	2	45 000	59 300	24.5
Flat............	⅝ x 7/16	2	38 700	57 000	45.5
Flat............	1 x ⅜	2	42 400	61 700	45.8
Flat...........	1½ x ¼	2	44 600	60 800	42.8

Fig. 27 gives the results of tests of the small columns. The dotted line represents the maximum fiber stress computed by considering the eccentricity of loading at any cross-section to be equal to the deviation of that section from a straight line connecting the ends of the column. The deflections were slight, and were neglected in the calculations. The solid line represents the stresses on the two sides of the column, as determined from the extensometer readings.

27. *Tests of Column Material.*—Table 14 gives the results of the tensile tests of samples of materials from various parts of the flange members of the columns, and also of tension tests of lattice bars. Table 15 gives tension tests of lattice bars like those used in the compression tests of lattice bars.

V. Discussion.

28. *The Action of Built-up Compression Pieces.*—In analytical discussions of column action, the stress is usually assumed to vary uniformly from a minimum on one side of the cross-section to a maximum on the opposite side, and the whole cross-section of the column is considered to act as a unit. The longitudinal axis of the column is also considered to take a definite elastic curve under load. In the derivation of most column formulas, it is assumed that the amount of deflection of the elastic curve from the original position of the axis is an important element in fixing the maximum stress in the column. Although these assumptions are generally used as the basis of column formulas, it may be well to consider whether conditions may not exist, in columns of ordinary form and dimensions, which will render doubtful the general applicability of some of these assumptions and will dwarf the effect of others. At any rate, it seems worth while to consider the effect of other conditions in a built-up member. It must be borne in mind that the built-up column is subject to imperfections of fabrication, and that some crookedness and eccentricity must exist. The component parts of the column may be relatively slender and flimsy. Whether there is integrity of cross-section under load, is a question. In the tests herein described, the amount of deflection from the original axis, for loads up to a point somewhat below incipient failure, was found to be slight (generally between 0.04 and 0.1 in.), much smaller than necessary to account for the stresses observed in the columns.

The action of short columns at failure may be expected to be different from that of longer columns, although the stresses up to incipient failure may be the same. Granting that the conditions of non-straightness are such that the distribution of stress over the cross-section is the same for the two lengths of columns, and that the deflection of the column is so slight as not to affect materially the stresses developed, the longer column will be in more danger of immediate and sudden collapse after the yield point of the material in any fiber has been reached, and the total load carried before complete failure will, in general, be less. This is because, in a ductile material, after the stress at one side of the column has passed the yield point, the total resistance of the section to compression will increase, while the resistance to cross bending may not. Under the conditions named, the bending moment due to eccentricity will be the same until the yield point in some fiber is reached. After yielding begins, the greater deflection in the longer column rapidly increases its relative eccentricity, and more rapid failure may be expected than with the shorter column.

29. *Indications of Data.*—It will aid in the interpretation of the data of the distribution of stress over the channel members of the columns to point out a few simple indications. Reference may be made to the diagrams in Fig. 8 to 15, and Tables 2, 3, 8, and 9.

1. Any lack of agreement between the average load stress and the average of the stress given for the four centers of gravity of channel flanges may be ascribed to errors of observation.

2. If the stress at the center of gravity of one channel is above the average stress throughout the length of the column, and the corresponding stress for the other channel is similarly below the average stress, there must be an eccentricity in the application of the load at the two ends. If the stresses at the center of gravity of one channel member form in the diagram a straight line which crosses the line of average stress, and that for the other channel crosses in the opposite way, the eccentricity of the load application must be oblique.

3. If the stress at the center of gravity of a channel in near-by points is greater first in one channel and then in the other, the change may be due to crookedness of the column throughout that part of the length.

4. If, in one channel or in one channel flange, the stress at the center of gravity remains constant and that of the extreme fiber varies, the change may be due to local crookedness of this channel and there will be a lateral bending of this member.

5. If the front side of a channel has a higher stress than the back side, there must be bending action through its web, and *vice versa.*

6. Changing stresses in the diagonally opposite corners of a channel may indicate twisting of the channel, and another combination of stresses may indicate a twisting or oblique distortion of the column as a whole.

An inspection of the diagrams shows that all these indications are found in the tests.

7. To be in accord with the principles of column formulas of the Rankine type, there should be from end of column to middle a regular increase in the stress in one channel or in one flange of a channel and a corresponding decrease in the stress in the other. Verification by an agreement between the distribution of stress and the theory would be important. It will be seen that this verification was not obtained.

30. *Does the Built-up Column Act as a Unit?*—Engineers have often expressed doubt as to whether the parts of a built-up column act as a unit, although column formulas assume this unity of action. The tests throw some light on the question of the integrity of cross-section under load. The individual channel, of course, acts as a unit to resist bending action, though there are indications of twisting. The integrity of the whole section with reference to a plane parallel to the lacing seems probable, except as twisting action exists. With reference to a plane through the axis perpendicular to the plane of the lacing, this unity of action is not so certain. The tests on the distribution of compressive stress and likewise the cross-bending tests of the columns indicate that these built-up columns did not in all cases act as a unit but rather as two members not fully restrained by the lacing. The stresses in two channels as points in the same cross-section do not give the regularity of variation which would exist if the column bent as a unit. The elastic curve assumed by Column No. 1 under cross-bending load, shown in Fig. 18, differs from the computed elastic curve, though that for the wrought-iron column gives little difference. In the case of the posts of the White Heath bridge, however, there is much closer agreement and a seemingly closer approach to unity of action.

31. *Effect of Non-straightness of Built-up Columns Upon Distribution of Compressive Stress.*—The effect of crookedness or other irregularities of a constituent member of a built-up column may be realized if a rough analysis of the case be made. Consider a part of one of the channels forming a column, taking the length between the connections of two adjacent lattice bars. This member is under compression. Owing to non-straightness or to the non-homogeneity of the material, the load on this short piece is not evenly distributed over the section; that is, it is not centrally loaded, but may be considered to have an eccentricity with respect

to the gravity axis. Call this eccentricity e (Fig. 28). Neglect any deflection of the piece under consideration due to the load. Call the compression load coming on this piece P; A its area of cross-section; I its moment of inertia about YY, and r the corresponding radius of gyration; and c the distance from YY to the remotest fiber. Then the bending moment due to the eccentricity is Pe. The maximum stress will be

$$f = \frac{P}{A} + Pe\frac{c}{I} = \frac{P}{A}\left(1 + \frac{ec}{r^2}\right).$$

The excess of the stress in the extreme fiber of the piece over the average stress, produced by the eccentricity e is then $\frac{P}{A}\frac{ec}{r^2}$, and hence the term, $\frac{ec}{r^2}$, gives the proportionate excess of stress in the extreme fiber. This value is applicable to the channel, or to

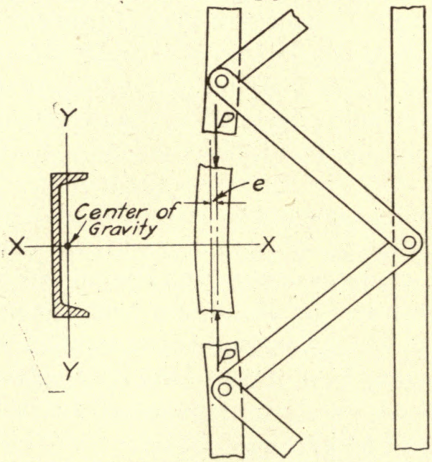

FIG. 28. EFFECT OF ECCENTRICITY IN CHANNEL MEMBERS.

one flange of the channel, or it may be applied to the column as a whole by using the properties of the whole section. In the single channel under consideration, c is relatively large and r is relatively small, and the excess of maximum stress for a given eccentricity e may be expected to be relatively large. It will be seen that for an excess of 50% in the extreme fiber of a channel of Column No. 1, e, by this formula, would be 0.045 in., and, in the wrought-iron columns, 0.057 in. A slight variation from straightness in a channel will account for considerable increase of stress.

32. *Excess of Maximum Fiber Stress over Average Stress in Channel Members.*—The diagrams and data show that the compressive stress is unevenly distributed over the cross-section of the columns tested, and also that there is great variation in this distribution at various sections along the length of the column. It will be noted that in a number of sections the excess of stress was from 40 to 50 per cent. In one test of Column No. 1, an excess of 67 per cent was found, and in the White Heath bridge an excess of 73 per cent. Possibly these values were unusual or the observations were erratic, but the indications of a fiber stress of from 40 to 50 per cent in excess of the average stress were not uncommon.

It may be seen that among the causes to which the high fiber stress may be attributed are (a) non-straightness of the column as a whole, (b) non-straightness of the component channels, or eccentricity in the delivery of stress to them by the lacing, and (c) unknown eccentricity in the application of the load. It would be of interest to know how much of the increase of stress may be due to any one of these conditions. A study of the tests of Column No. 1 shows that generally only a small amount may be said to be due to non-straightness of the column as a whole. In but few cases is it found to be more than, say, 5 per cent; in four places it seems that the excess attributable to this may be estimated to be between 20 and 25 per cent. The effect of non-straightness of the individual channels seems to be greater. At several points the excess of stress attributable to this cause appears to be from 30 to 50 per cent. As already noted, a kink in the channels of 0.045 in. would give, by the analysis made, an eccentricity sufficient for a 50% increase in stress. Not all of this crookedness need be between adjacent rivet points, as the stress may not become normal for some distance on either side. The effect of the third condition, eccentricity of application of the load, will vary with the construction. In Column No. 1 the effect of undetermined eccentricity of application of load appears to be not nearly as great as the effect of non-straightness of the component channels.

In the wrought-iron columns, which are much stockier, the lack of straightness in individual channels has less effect, seemingly less than 15 per cent, and much the larger part of the high fiber stresses appears to be due to general column eccentricity or to eccentricity of loading.

The results for the posts of the White Heath bridge are of interest in this respect. It is evident that the effect of non-straightness of channels was not large, and also that the effect of non-straightness in the column as a whole was relatively small. There is, however, an evident bending in the direction of the web of the channels. For example, in U_3L_3 South, the back side of the channels has the maximum stress at the top and the front side at the bottom. The bending moment producing this may be due to obliquity of end pressures or to a bending by the connecting floor-beam and top cord. A twisting action is also apparent. Posts U_3L_3 North gave quite similar results.

33. *Effect of Cover-plates and End Connections.*—In the tests of the White Heath bridge, the effect of the cover-plate seems striking. The upper chord, U_3U_4, composed of two built-up channels with one cover-plate, gave an excess fiber stress of 20 per cent at the worst section, while the post, composed of two channels laced on both sides, gave a maximum of 73 per cent. The high value in the posts may be due to other causes, but it seems reasonable to expect that the cover-plate will act to reduce the irregularities in fabrication. Engineers have stated that columns having a cover plate are fitted into their places during erection with less labor than is required for columns with lacing on both sides. Tests on the stress distribution of such columns would be valuable as affording a basis of definite comparison.

The connections of the ends of posts evidently exerted a very noticeable effect on the stress distribution. In one of the posts tested, the stress was greatest at one corner of the post at the top and at the diagonally opposite corner at the bottom. It will be remembered that the posts were riveted to the top chord, and were connected with the lower chords by pins. The floor-beams were riveted to the sides of the posts, and this connection affects the stress distribution. Readings of deformations taken on the floor-beams and posts show that the loaded beam was partly restrained at the ends by the post, though this restraint introduced a bending moment at the end of the post only about one-quarter as great as at the center, and that there was an appreciable bending in the post.

34. *Stresses in Column Lacing.*—If the load carried by one channel of a column was the same throughout its length, no stress would be carried by the lattice bars. Such stress is developed

whenever there is a change in the relative amount of loads carried by the two channels. If, at the section AB (Fig. 2, p. 11), there is an equal division of load between the two channels, and also at the section CD, and if at some section EF, the division of load is unequal, it is evident that the lattice bars must be called into action to transmit this stress, and that transverse shear exists in the interval. In general, the conditions producing this will be complex, rendering analysis unsatisfactory, except in so far as the shear may be due to a known eccentricity of loading.

It is evident from the tests that the relative stress in the two channel members varies considerably from end to end and that the stress in the lattice bars also varies. It seems probable that the transverse shear developed may be traced largely to irregularities in outline, or at least that these irregularities may be expected to cover up other causes of stress in the lacing of centrally-loaded columns, if we include in such irregularities all unknown eccentricity. The futility of attempting to determine analytically the stresses in column lacing, using as a basis either a bending moment curve which varies regularly from end to middle or an assumed deflection curve, is apparent from a study of the variation of stress in the columns of the tests and in that of the lattice bars.

The amount of transverse shear necessary to produce the maximum observed lattice-bar stress (given in Table 6) is of interest, though of course it cannot be taken to be conclusive. The measurements were generally made at working loads. So far as observations were made on columns tested to failure, the distribution of stress remained much the same up to incipient failure. The values given in Table 6 indicate maximum average stresses in the bars such as would be caused by a transverse load ranging from 2% to 6% of the central compression load or of a transverse shear of 1% to 3% of the load.

35. *Compressive Strength of Lattice Bars.*—In the discussion of stress developed in column lacing, the stress considered was the average over the bar. As usually attached, there is considerable flexure in the bar, and the ability of the bar to carry this eccentric load should be considered. The bars are most likely to fail in compression, since they act as long columns eccentrically loaded. This compressive strength may be greatly diminished by the bending which they frequently receive in transportation and erection.

The tests of individual lattice bars (Fig. 25 and Table 12) show that the maximum fiber stress may be several times the average stress. It is also seen that even in a short lacing bar the maximum load carried is only about one-half the yield point of the material. The necessity of using very low working loads on lattice bars appears to be important. It will be noted that at low stresses there is similarity of distribution of stress in the slender bars and in the thicker bars, but the slender bars fail at smaller computed fiber stress.

The results of tests to destruction of individual lattice bars (flats) are fairly well represented by the formula:

$$\frac{P}{A} = 21\,400 - 45\frac{l}{r}$$

where $P =$ load at failure in pounds, $A =$ area of cross-section in square inches, l is the distance in inches from center to center of rivet holes, and r is the radius of gyration, in inches, of the cross-section of the lattice bar. The results of the tests were adjusted so that this formula applies to material having a yield point of 40 000 lb. per sq. in. These results may be considered to be applicable to "under" lattice bars. For "over" bars it seems probable that the average stress at failure would be considerably less. If $\frac{l}{r}$ is 0 in the above formula, that is, if we have a very short lattice bar, the average stress over the bar at failure would be 21 400 lb. per sq. in. If the extreme fiber stress in this short bar is 40 000 lb. per sq. in., the yield point of the material, the equivalent eccentricity of loading (e) which would produce this, may be found from the equation

$$40\,000 = 21\,000\,(1 + \frac{e \times \frac{1}{2}t}{r^2}),$$

where t is the thickness of the bar. The resulting e is found to be very nearly $\frac{t}{7}$. We may then regard the lattice bar to have been loaded with an initial eccentricity equivalent to $\frac{1}{7}$ the thickness of the bars.

36. *Effect of Form of Section.*—The large variation in stress over a cross-section of the column and the marked changes in stress from section to section along the column are evidently due to local crookedness, local eccentricity, lack of rigidity of lacing,

and other variations which may be independent of the general flexural curve usually assumed in deriving the usual formulas for column strength. It would seem that the form of section (including in this term the relation of the thickness of the metal to the section as a whole) has a bearing on the strength. The thinner and flimsier component angles and channels are more liable to receive kinks, bends, and distortions before and during punching and riveting in the shop and in the later transportation and erection than are the stockier sections. The value $\frac{ec}{r^2}$ of the formula given on page 54 may be expected to vary with the form of section used. Besides, some sections are better fitted to withstand lateral twisting or diagonal distortions and to preserve the integrity of the cross-section than others. The wrinkling tendency in plates and thin parts under compression, heretofore referred to, is another element affecting the strength of columns. It may be expected, then, that differences in section, in type of component parts, in method of relating and tying the parts together, and for the same type of section differences in relation of thickness of parts to extreme dimensions of sections, will have an important influence upon the compressive strength of columns. It follows, therefore, to give the best results, that the section of the column, and its web construction, should be chosen so that (1) the shop processes shall leave the component parts of the column in the best condition (giving the minimum of bending, buckling, twisting, and interior eccentricity), and (2) the section will be adapted to resist local lateral bending and twisting action. Evidently, different forms of section may be expected to give considerable difference in strength. This difference has been recognized heretofore in formulas which have been proposed and used for certain types of columns.

37. *Effect of* $\frac{l}{r}$.—A study of the tests does not show any relation between the stresses actually observed and the stresses computed by column formulas. The high stresses do not come where the curve of flexure used as the basis of formulas of the Rankine type would place them, and the position and amount of the maximum stresses are very irregular. Although there is little range in the slenderness ratio $(\frac{l}{r})$ of the columns tested, no

effect is noticeable for which the value of $\frac{l}{r}$ would seem to have much influence. This view seems to be in disagreement with theoretical considerations. The lengths for which Euler's formula may be expected to govern column strength are much greater than the length tested, and probably are higher than is generally assumed in engineering literature. Within the critical length at which Euler's formula governs, the general flexure of the column as a whole under load has less influence upon the strength of the column than is ordinarily assigned to it, and therefore the influence of $\frac{l}{r}$ is not as great as is represented in the usual column formula. Of course, the longer the column the more the amount and influence of its defects may be. The recent tests of columns at the Watertown Arsenal indicate that, within the range of lengths tested ($\frac{l}{r}$, 25 to 175), the reduction in strength at elastic limit with increased length is relatively small, perhaps not much more than may be due to increased variation from straightness and homogeneity. In this connection it should be noted that the column formulas in common use give altogether too high strengths for short columns, if the elastic limit is to govern. So far as ultimate strength is concerned, tests show the strength of short columns to be considerably above their elastic limit, but beyond a limit of, say, 35 for $\frac{l}{r}$, there is much less difference between elastic limit strength and ultimate strength.

38. *Column Formulas.*—That the column formulas in common use have limitations, has been well understood, but the effect which the conditions of the component parts of a compression member exert on the distribution of stress over the section has not been appreciated, nor has that of eccentricity of connection of latticing, and of the possible non-integrity of section. It would seem quite probable that, for columns of the same length and containing the same amount of metal, one which is of stocky form and in which the metal is distributed so as to resist local flexural and torsional action will be much stronger and more satisfactory than a column of more flimsy form, which has its metal spread in thinner sections, even though the slenderness ratio, $\frac{l}{r}$, of the for-

mer may be considerably more than that of the latter. It seems reasonable to expect that a form of section which resists lateral bending, torsional and collapsing stresses, will be much more satisfactory than a more flimsy type of column, for the lengths most common in ordinary bridge construction. If these statements are trustworthy they express an important principle. For the longer lengths, the slenderness ratio must exert a stronger influence. For the strength of the component angle, channel, or other structural shape used in a built-up compression piece, many engineers have been satisfied with the provision that the slenderness ratio of the component member shall be less for the length between the points of attachment of lacing than the slenderness ratio for the column as a whole, and have given little attention to the possible non-integrity of the section or to the probable effect of imperfections of manufacture. Fortunately, the large influence of the slenderness ratio in column formulas has given sections with which failures have not occurred. Whether a column formula should include a factor depending on the form of the section and the relative thickness of the metal, or whether the allowable stresses for any form of column should be based on experimental data for the section used, will depend on future developments.

39. *Field for Investigation.*—The tests herein recorded have shown the practicability and also the importance of making tests on the distribution of stress over built-up columns within the elastic limit, both under laboratory conditions and in field service. It is evident that much experimental information is needed on the stresses which are developed in compression members built under ordinary conditions of fabrication and erection before a satisfactory column formula may be established. Tests giving the needed information involve extreme care, and they are expensive, with regard to time and labor, whether done in the field or in the laboratory. A full study of the action of the compressive piece under loads which do not stress the material beyond the elastic limit should be included. The expenditure involved is far beyond that of tests to destruction alone. An investigation should be accompanied by a careful study and analysis of the tests and results. A program of tests need not involve a large number of test pieces; but, to be really useful for the purpose in view, the time devoted to the test and the study of each piece must be ample, and the total

cost of even a fairly comprehensive investigation will be large. It may be expected, however, that the value of the results would repay many times the cost of the work, and the expense would be justified by the added security and, perhaps, by the economy of metal which might result from the investigation.

40. *Summary.*—The main points brought out in the preceding discussion may be recapitulated as follows:

1. The practicability of making tests to determine the actual stresses which are developed under working loads and up to the elastic limit of the material in the members of a column, throughout its length and over its cross-section, has been shown. The results significantly point to the importance of making investigations of this kind. The experimental work involved is tedious and laborious, and of course, the work requires skilled and experienced experimenters. The need of such information has been recognized heretofore, but tests have not been taken up because of the supposed impracticability.

2. An important result of the tests is the evidence that considerable local flexural action exists in the channel members of the columns, such as may be produced by lack of straightness or by any method of applying the load eccentrically. This is especially true in the flimsier column.

3. The condition of flexure varies markedly throughout the length of the channel member, in some cases the maximum compression in one cross-section being at the extreme fiber on one side of the channel, and in a near-by section the other side of the channel showing the excess of stress.

4. There were also indications of sudden changes in the relative amount of stress carried by the two channels at near-by sections, indicating general flexure of the column.

5. The measurements made indicate in a number of cases stresses in the extreme fiber from 40% to 50% in excess of the average stress, and in some cases even higher.

6. The amount of eccentricity necessary to account for the increase of stress found in individual channels, based on lack of straightness and the ordinary theory of flexure, is relatively small.

7. The amount of deformation observed in lattice bars is relatively small, and its variation throughout the length of the column is quite irregular. The measurements indicate a stress in

the lattice bars which would be produced by a transverse shear equal in amount to 1% to 3% of the applied compression load, or to that produced by a concentrated transverse load at the middle of the column length equal to 2% to 6% of the compression load. The stress referred to is the average stress over the section of the lattice bar.

8. It seems futile to attempt to determine the stresses which may be expected in column lacing for central loading by analysis based on theoretical considerations or on data now available.

9. When the column was tested as a beam, the extreme fiber stress in lattice bars in compression was found to be from 1.4 to 5.5 times the average stress over the cross-section of the lattice bars, and the extreme fiber stress in lattice bars in tension was found to be 1.7 to 3.2 times the average stress.

10. Tests of individual lattice bars for load-carrying capacity under conditions which resemble those of service show that the usual form of bar is a very inefficient compression member when loaded eccentrically through a riveted connection. The ultimate strength was in no case as much as one-half of the yield point of the material.

11. The formula, $\frac{P}{A} = 21\,400 - 45\frac{l}{r}$, represents fairly well the ultimate strength of the flat lattice bars tested, based on material having a yield point of 40 000 lb. per sq. in.

12. It seems evident that the component members of a built-up column do not act together in such a way as to give entire integrity of cross-section in resisting bending.

13. The distribution of stress under working loads, and even up to incipient failure, may be different from that which exists after the column becomes crippled. This is due to the yielding of the more strained parts after the yield point is reached at any fiber, and a consequent redistribution of stress.

14. The sudden failure of a test column at a relatively low load by buckling of the lattice bars is accounted for when the amount of transverse shear developed in other test columns and the strength of lacing found in lattice-bar tests are taken into consideration.

15. No relation has been found between the stresses actually observed and the stresses computed by column formulas. The stresses do not increase toward the middle of the length of the

column, as may be expected from the Rankine form of analysis, but are quite irregular in their location and distribution.

16. Much of the excess of extreme fiber stress over average stress is evidently attributable to local crookedness of piece, eccentricity of bearing of lattice bar connection, lack of rigidity of lacing, and other irregularities that are due to the condition of the material and its fabrication, and what may be considered to be inherent variations and defects in the constructed compression piece. Within the elastic limit of the material and for the lengths most commonly used the lateral flexure of the column as a whole is very slight, and slenderness ratio can not be said to be the governing consideration. Undoubtedly, the chances for variations from the ideal column will become greater as the column length becomes greater, and these variations may have a more marked effect upon its strength.

17. It is evident that the form of section is important. Stocky and stiff component members are less liable to receive kinks, bends, and distortions during and after fabrication and will resist the effect of such imperfections with less resulting stress than will flimsy pieces. Some column sections are well calculated to resist bending, buckling, and twisting, and are so tied together as to preserve integrity of section, while others have less resistance to general distortion. Even the wrinkling action in plates and thin parts needs consideration. It seems reasonable that, for columns of the same length and containing the same amount of metal (within the ordinary dimensions), one which is of stocky form and in which the metal is distributed so as to resist local flexural and torsional action will be stronger and more satisfactory than a column of more flimsy form, which has its metal spread in thinner sections, even though the slenderness ratio, $\frac{l}{r}$, of the former may be considerably more than that of the latter. Further, a section which will come through the shop and erection processes with the least imperfections has advantages.

18. This field of investigation is a promising one, and its importance to the engineering profession warrants its being taken up in a thorough and comprehensive manner. Full information on many matters is needed before better and more nearly satisfactory column formulas may be established.

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