COMPRESSIVE STRENGTH OF STEEL COLUMNS INCASED IN BRICK WALLS

By Albert L. Harris, A. H. Stang, and J. W. McBurney

ABSTRACT

Three H-shaped steel columns, 23 feet long, 6 inches in depth, weighing 20 pounds per foot, and having flat ends, were tested in compression. Six similar columns, incased in brick walls 14 inches thick, 6 feet long, and carried up to about 8 inches from the top of the columns, were also tested in compression. The six incased columns were divided into two groups of three each depending on the orientation of the steel column in the brick wall.

Compressometer and deflection measurements were made on all columns.

The incased columns were much stronger than the bare specimens. The increase in strength may arise from two different causes. The load may be partially transferred from the steel to the incasement by the bond between them. Added strength from this cause might be expected in columns of any length. Long columns when laterally unrestrained fail by bending at loads much smaller than the yield strength of the steel. Incasement of long columns would act as a continuous lateral restraint which might be sufficient to prevent the lateral bending and cause the columns to fail in the same manner as short columns at much higher loads.

The compressometer readings over a 150-inch gage length centered at mid-height showed that the steel in the incased columns was stressed, on the average, less than 20 per cent as much as was the steel in the bare columns for the same load. The lateral deflection of the incased columns was very small and the brickwork was not appreciably affected by the tests.

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I. INTRODUCTION

1. PURPOSE

The engineer department, Government of the District of Columbia, Washington, D. C., requested the Bureau of Standards to test 9 steel columns, 3 bare and 6 incased in brick walls, to determine the relative

strengths of the bare and the incased columns. The purpose of the tests was to determine whether for design purposes, the strength of steel columns incased in brick walls should be calculated by considering them as long, bare columns which would fail by bending at low loads or whether the brick walls could be relied upon to restrain the lateral bending and enable them to carry the higher loads which short columns of the same material would carry.

2. ACKNOWLEDGMENTS

The incasement for the steel columns was designed by Albert L. Harris, municipal architect, and H. H. Marsh, chief of the structural division, engineer department, Government of the District of Columbia. The steel columns were donated by the Bethlehem Steel Co. The cost of incasing the columns in brick walls was borne by the Washington division, Common Brick Manufacturers' Association of America, Jordan A. Pugh, district manager.

The tests of the columns were made in the engineering mechanics section, of the Bureau of Standards, under the direction of H. L. Whittemore.

II. THE SPECIMENS

The nine steel columns were 6-inch, 20-pound Bethlehem H-sections. The nominal dimensions and section properties of the steel columns are given in Table 1. The ends of each column were machined with a face mill to give a nominal column length of 23 feet. After the columns were tested, a short length of each steel column was weighed and the length measured. The cross-sectional area of this short length was computed on the assumption that 1 cubic inch of steel weighs 0.283 pound. This value was taken as the cross-sectional area of the steel column.

TABLE 1.—*Nominal dimensions and sectional properties of the steel columns*

Weight per foot	pounds	20
Depth	inches	6. 00
Flange width	do	6. 00
Web thickness	inch	. 25
Area	square inches	5. 89
Least moment of inertia, centroidal axis parallel to web	$in.^4$	13. 5
Least radius of gyration, centroidal axis parallel to web	inch	1. 51
Greatest moment of inertia, centroidal axis perpendicular to web	$in.^4$	39. 2
Greatest radius of gyration, centroidal axis perpendicular to web	inch	2. 58
Length	feet	23
Slenderness ratio		183

The steel columns which were tested without the brick incasement were designated as columns 1, 2, and 3.

Six of the steel columns were incased in brick walls as shown in Figures 1 and 2. The nominal dimensions of the walls were as follows: Thickness 14 inches, length 6 feet, and height 22 feet 4 inches.

Previous tests on sturdy columns of this length indicated that these incased columns might be compressed 6 inches before the maximum compressive strength was obtained. The distance between the top of the wall and the upper end of the steel column was about 8 inches to allow the maximum load on the steel column to be determined before the upper platen of the testing machine came into contact with the top of the wall. This unincased length of the steel columns above

the walls was greater than would be likely to occur in an actual structure. The actual unincased lengths and the heights of the walls are given in Table 5. The unincased lengths varied from 7½ to 11 inches. For the three incased columns, designated A1, A2, and A3, the web of the steel column was perpendicular to the face (length) of the wall; and for the three designated as B1, B2, and B3, it was parallel to the face. (Fig. 2.)

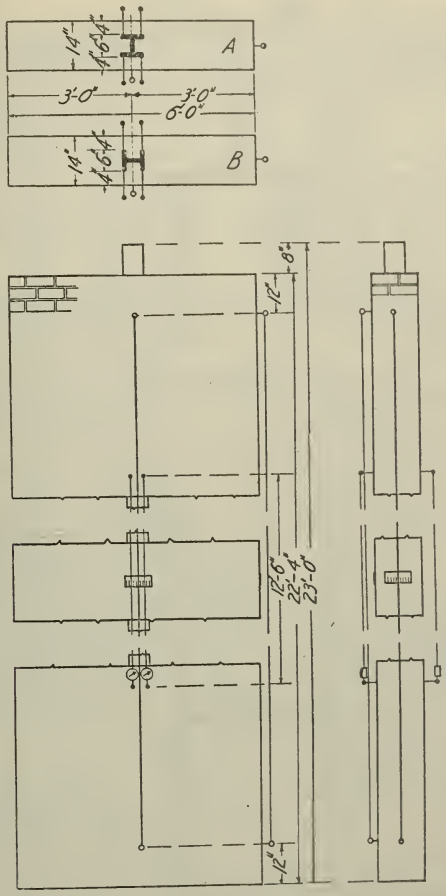
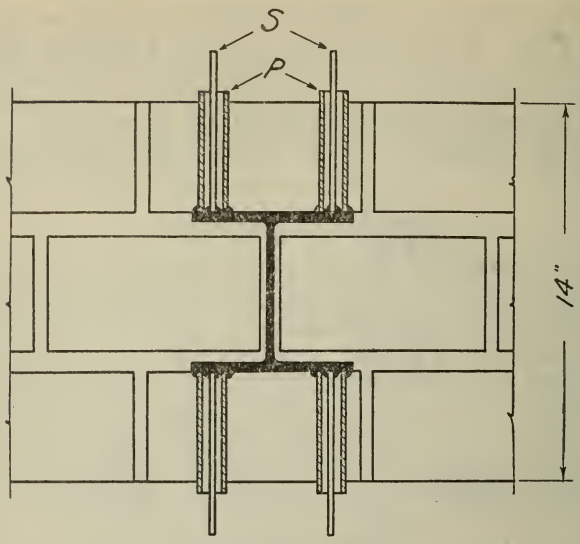


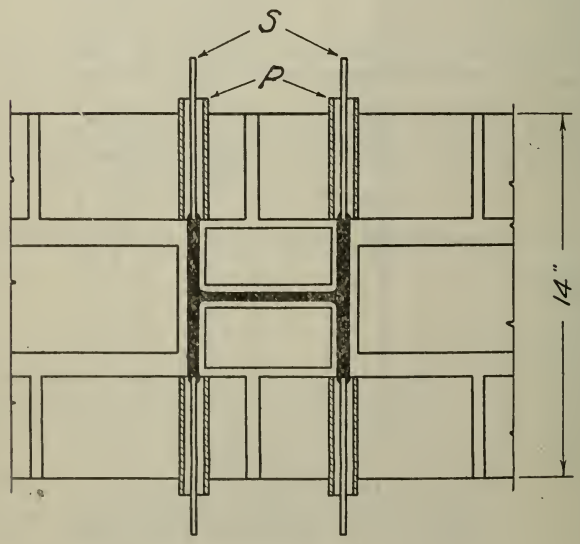
FIGURE 1.—*Incased columns showing nominal dimensions and the apparatus for measuring the compression and lateral deflection*

The bricks were made in Virginia from surface clay by the semidry press process. The mortar was composed of 1 part of Portland cement to 3 parts of damp sand, by volume. Hydrated lime was added equal to 10 per cent of the cement, by weight. The proportions by weight were 1 part cement, 0.1 part lime to 2.34 parts dry sand.

The steel columns were secured in a vertical position on the floor of the laboratory and plumbed before the brick walls were built. The bricks were laid on smooth, horizontal mortar beds and with full-shoved vertical joints. Every fifth course was a header course. The



A Specimens



B Specimens

FIGURE 2.—*Cross sections of the incased columns showing the orientation of the steel columns and the bonding of the brick*

The section is taken through the supports for the compressometers to show the method of attaching them to the columns

FIGURE 3.—*Incased specimens during construction*

A, The lifting plates; *B*, the compressometer supports.

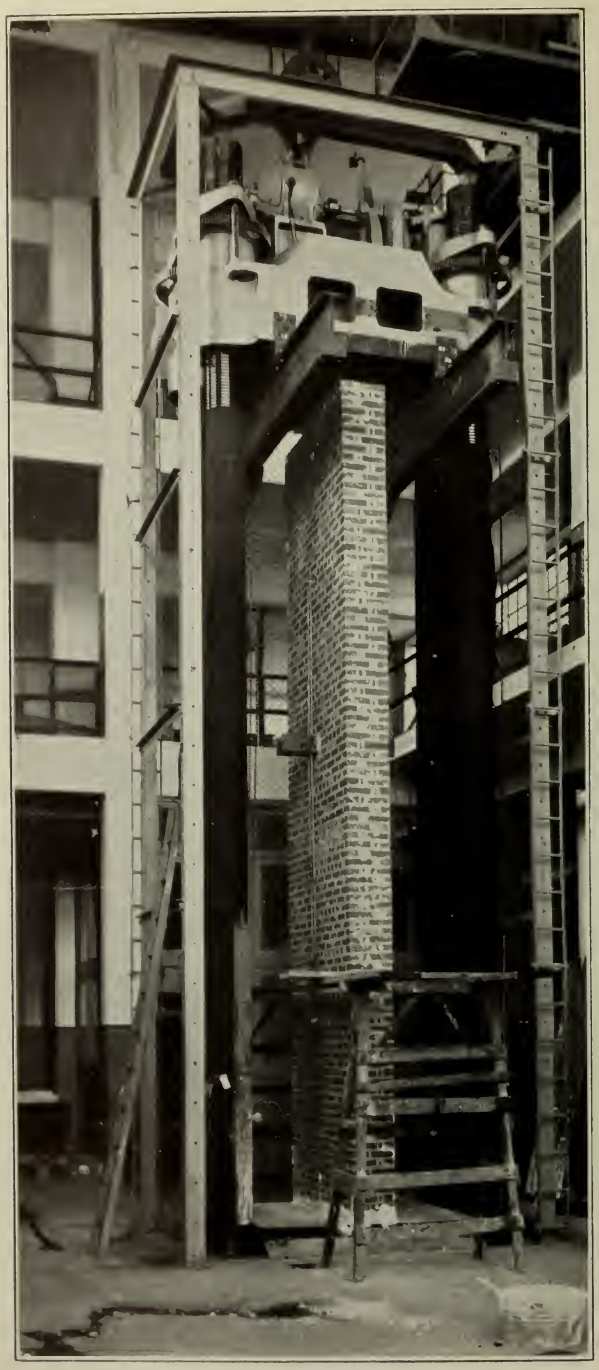


FIGURE 4.—*Incased column B3 in the testing machine before test*

bricks were carefully fitted between the flanges of the steel columns, and the spaces between the bricks and the column were grouted with the mortar mixed with additional water.

The incased columns during construction are shown in Figure 3. The steel plates, shown at *A*, were bolted to the steel column about 10 feet from the top. They projected from the face of the wall as shown in Figure 4 and were used to lift the incased columns from the floor and place them in the testing machine. The incased columns were tested about 30 days after the completion of the walls.

III. TESTING PROCEDURE

1. THE BARE AND THE INCASED COLUMNS

All the columns were tested as flat-end columns in a vertical, hydraulic testing machine having a capacity of 10,000,000 pounds. Steel plates, 8 inches square and one-half inch thick, were placed between each end of the columns and the platens of the testing machine. For the incased columns, therefore, there was a ½-inch space between the bottom of the brick wall and the lower platen. This space was filled with a mixture of plaster of Paris and water, the plaster being allowed to harden before the loads were applied.

The shortening of the columns under load was measured with four compressometers having a gage length of 12 feet 6 inches. The rods of the compressometers actuated dial micrometers graduated to 0.001 inch. Readings were estimated to one-tenth of a division. On the bare columns the middle of the gage length was at the middle of the column. The compressometers were attached by studs screwed into the outside surfaces of the flanges, one near each edge. On the incased columns (fig. 1) they were attached to rod supports welded to the flanges, as shown at *S*, Figure 2, and *B*, Figure 3. Pieces of pipe surrounded the rods and were welded to the flanges, as shown at *P*, Figure 2, so that the compressometer supports were not in contact with the brickwork. After the column had been set in place and the compressometers attached, the lower platen of the testing machine was adjusted in its spherical seat until, at low loads, the compressometer differences for the bare columns were all within 10 per cent of the average compression. For the incased column A1, these differences were 32 per cent. For the other incased columns, they did not exceed 16 per cent.

The lateral deflections of the columns at midheight were measured by the taut-wire mirror-scale method. The scales were graduated to 0.1 inch and readings were estimated to one-tenth division.

On the bare columns three wires were used. They were attached 6 inches from each end of the column by studs screwed into the middle of each flange and the middle of one side of the web. On the incased columns (fig. 1), two wires were used, attached 12 inches from the top and bottom of the wall to steel supports built into the wall.

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2. AUXILIARY TESTS

(a) THE STEEL COUPONS

As large differences were to be expected between the strength of the bare and the incased columns, it was not felt necessary to test as many coupons as would be required in the case of column tests planned to determine small differences in column strength caused by differences in construction.[1] Only two coupons were, therefore, taken from each steel column. Since no extra lengths of the columns were furnished, one coupon was cut from the flange and the other from the web of the column, after test, the portion being used which was considered to have been subjected to the lowest stress during the column tests. The coupons from the bare column were cut from the region of the contraflexure and those from the incased columns from the lower end. They were A.S.T.M. type specimens having a gage length of 8 inches and a reduced width of 1½ inches. They were tested in a screw-power, beam-and-poise testing machine having a capacity of 100,000 pounds. The yield point was obtained from the "drop of the beam" of the testing machine.

(b) THE SHORT LENGTHS OF STEEL COLUMNS

Lengths equal to the unincased length of the columns were cut from each of the incased columns after test. These short lengths were cut from the lower end next to the lengths from which the coupons were taken. The ends were milled perpendicular to the axis of the sections and the maximum compressive strength determined by loading them between hardened steel plates in a screw-power, beam-and-poise testing machine having a capacity of 600,000 pounds. The compressive strengths were computed by dividing the maximum load by the computed cross-sectional area of the steel column from which the length was cut.

(c) THE BRICKS

Twenty-five bricks were selected at random and tested in accordance with Federal specification No.504 for brick, clay, common.

(d) THE MORTAR

Each working day a sample of the mortar was taken and the flow value determined in accordance with paragraphs F–3g(3) *a* and *b* of Federal specification No. SS–C–181 for cement, masonry. Six cylindrical specimens (diameter 2 inches, length 4 inches) were made from each sample. Three of these cylinders were stored in water and the compressive strength determined after 28 days. The other three cylinders were stored in air in the laboratory and the compressive strength determined on the day the incased column which they represented was tested.

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[1] L. B. Tuckerman and A. H. Stang, Tests of Large Columns with H-Shaped Sections, B. S. Tech. Paper No. 328, Oct. 20, 1926.

IV. RESULTS AND DISCUSSION OF THE TESTS

1. AUXILIARY TESTS

The results of the tensile tests of the steel coupons are given in Table 2. The average values for each group of columns do not show any important differences in the properties of the steel in the steel columns. The tensile strength of some of the steel coupons was less than the minimum value specified in A. S. T. M. standard specification for structural steel for buildings, designation A9–29.

TABLE 2.—*Properties of steel coupons*

Specimen		Thickness	Yield point	Tensile strength	Elongation in 8 inches
From column—	Cut from—				
		Inch	*Lbs./in.²*	*Lbs./in.²*	*Per cent*
1	Flange	0.340	38,300	54,600	31.2
1	Web	.278	47,200	56,700	29.4
Average			42,800	55,600	30.3
2	Flange	.350	45,300	54,200	34.5
2	Web	.278	49,700	58,900	30.9
Average			47,500	56,600	32.7
3	Flange	.350	37,000	53,900	30.5
3	Web	.277	47,100	54,400	34.2
Average			42,000	54,200	32.4
A1	Flange	.332	40,200	54,800	33.9
A1	Web	.275	42,000	55,500	30.8
Average			41,100	55,200	32.4
A2	Flange	.348	38,600	55,200	33.1
A2	Web	.280	39,400	55,100	32.0
Average			39,000	55,200	32.6
A3	Flange	.345	39,100	54,900	31.7
A3	Web	.283	41,100	55,000	32.5
Average			40,100	55,000	32.1
B1	Flange	.375	39,600	55,000	29.2
B1	Web	.285	38,500	54,500	30.7
Average			39,000	54,800	30.0
B2	Flange	.393	39,700	54,500	33.0
B2	Web	.286	43,200	57,400	23.0
Average			41,400	56,000	28.0
B3	Flange	.351	39,400	55,600	29.7
B3	Web	.280	41,600	56,600	35.8
Average			40,500	56,100	32.8
Average values:					
1, 2, 3			44,100	55,400	31.8
A1, A2, A3			40,100	55,100	32.3
B1, B2, B3			40,300	55,600	30.2

The results of the compressive tests of the short lengths of the steel columns are given in Table 5.

The results of the tests of the bricks are given in Table 3. Each average value is the mean from tests on 25 bricks. As a measure of the dispersion of the results of individual tests about their corresponding averages, values for the standard deviations are given which were calculated by means of the following formula:

$$\sigma = \sqrt{\frac{\Sigma V^2}{n-2}},$$

where

σ = standard deviation,

n = number of individual values,

ΣV^2 = sum of the squares of the deviations of the individual values from their mean.

The bricks complied with the requirements for the hard grade in Federal specification No. 504, brick, clay, common.

The results of the tests on the mortar are given in Table 4.

TABLE 3.—*Properties of bricks*

	Length	Breadth	Depth	Weight	Transverse breaking load (7-inch span)	Modulus of rupture	Compressive strength, flat	Water absorption		
								5-hour cold	48-hour cold	5-hour boil
	Inches	*Inches*	*Inches*	*Pounds*	*Pounds*	*Lbs./in.²*	*Lbs./in.²*	*Per cent*	*Per cent*	*Per cent*
Minimum	8.25	3.80	2.20	5.15	970	476	2,960	6.70	7.8	10.4
Average	8.38	3.97	2.27	5.63	1,555	800	5,253	8.94	9.95	11.8
Maximum	8.55	4.10	2.30	5.81	2,540	1,335	7,750	12.0	12.6	14.2
Standard deviation	.091	.074	.037	.215	358	201	1,230	1.21	1.12	1.04

TABLE 4.—*Properties of mortar*

	Mortar flow values	Compressive strength of mortar cylinders (average of 18 specimens) conditioned in—	
		Air, age about 30 days [1]	Water, age 28 days
	Per cent	*Lbs./in.²*	*Lbs./in.²*
Average	136	2,090	2,510
Maximum	142	2,310	2,770
Minimum	128	1,820	2,090

[1] See Table 5, time between completion of wall and test.

The workmanship of the walls of the incased columns was the same as the workmanship of walls 43, 44, and 45 (Mississippi brick) reported by Stang, Parsons, and McBurney.[2]

These three walls were solid brick walls having no steel columns or other reinforcement. They were 12½ inches thick, 6 feet long, and 9feet high. The average compressive strength was 1,640 pounds per square inch. Differences other than workmanship make it difficult

[2] Compressive Strength of Clay Brick Walls, B. S. Jour. Research, vol. 3 (RP108), October, 1929.

to estimate the strength of the brick masonry of the incased columns. These differences are: First, although the modulus of rupture of the Virginia brick used for the incased columns was almost the same as that of the Mississippi brick, the compressive strength (flat) of the Mississippi brick (3,410 pounds per square inch) was much lower than that of the Virginia brick (5,253 pounds per square inch); second, the absorption of the Mississippi brick was about twice that of the Virginia brick; and third, the shape and surfaces of the Mississippi brick were much more uniform than those of the Virginia brick.

2. THE BARE AND THE INCASED COLUMNS

The results of the tests of the bare and of the incased columns are given in Table 5. Stress-strain diagrams were drawn for each column using the average strain computed from the compressometer readings. The stress for both the bare and the incased columns was obtained by dividing the load by the computed cross-sectional area of the steel columns. Due to the fact that a portion of the load was carried by the brick walls of the incased columns, the actual stress in the steel within the compressometer gage length was much lower than the values used in plotting the diagrams. The average stress-strain diagrams for the three bare columns, for the three A incased columns, and for the three B incased columns were also drawn. The diagrams for the columns are shown in Figures 5, 6, and 7. The average stress-strain diagrams are shown in Figure 8.

TABLE 5.—*Results of compressive tests of the columns and of auxiliary short lengths*

Column	Condition	Computed area of steel olumn	Uncased length of steel column and of the short length	Height of wall	Time between completion of walls and test	Yield point of coupon	Compressive strength		Load carried by	
							Column	Short length	Steel column	Brick wall
		Square inches	*Inches*	*Ft. In.*	*Days*	*Lbs./In.²*	*Lbs./in.²*	*Lbs./in.²*	*Per cent*	*Per cent*
1	Bare	5.90				42,800	23,900		100	0
2	do	5.90				47,500	23,000		100	0
3	do	5.85				42,000	23,100		100	0
Average						44,100	23,300		100	0
A1	Incased	5.85	7½	22 4½	28	41,100	40,000	41,200	18	82
A2	do	6.02	7½	22 4½	31	39,000	41,500	43,200	17	83
A3	do	5.98	11	22 1	30	40,100	40,700	38,300	19	81
Average					30	40,100	40,700	40,900	18	82
B1	Incased	6.03	10	22 2	29	39,000	38,800	40,600	16	84
B2	do	6.14	8	22 4	31	41,400	42,100	41,500	19	81
B3	do	6.10	8½	22 3½	31	40,500	41,100	42,200	16	84
Average					30	40,300	40,700	41,400	17	83

All the stress-strain curves are nearly straight. The slope of the straight lines shown on each diagram is the average ratio of stress to strain for that series of observed values. These average ratios are given in Table 6.

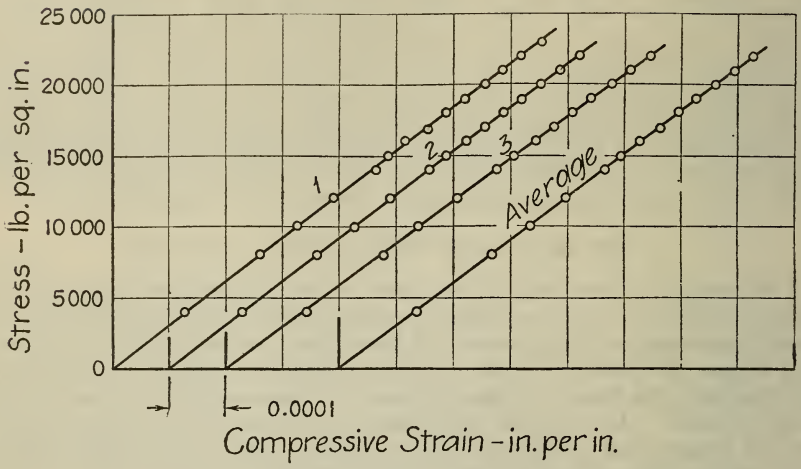

FIGURE 5.—*Stress-strain diagrams of the bare columns*

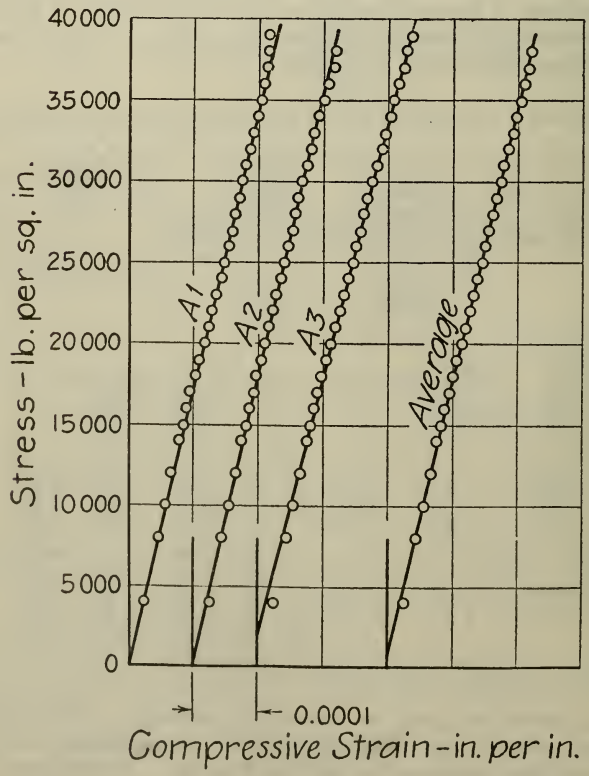

FIGURE 6.—*Stress-strain diagrams of the incased columns, group A, in which the web of the column was perpendicular to the length of the wall*

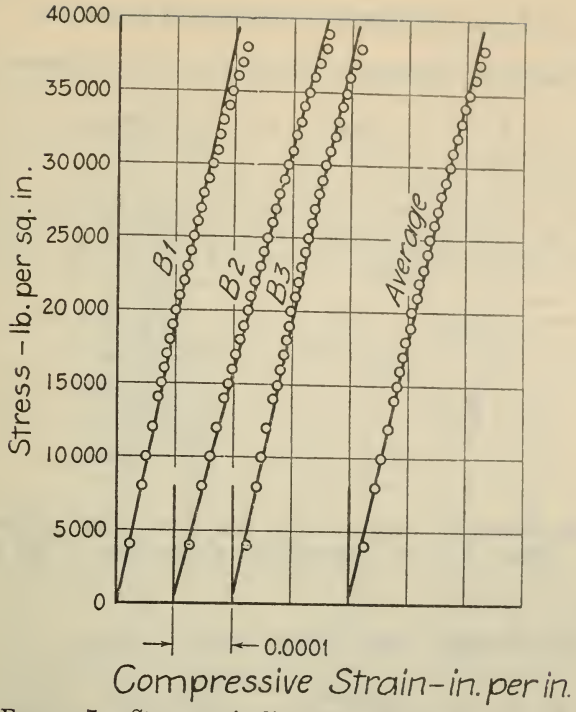

FIGURE 7.—*Stress-strain diagrams of the incased columns, group B, in which the web of the column was parallel to the length of the wall*

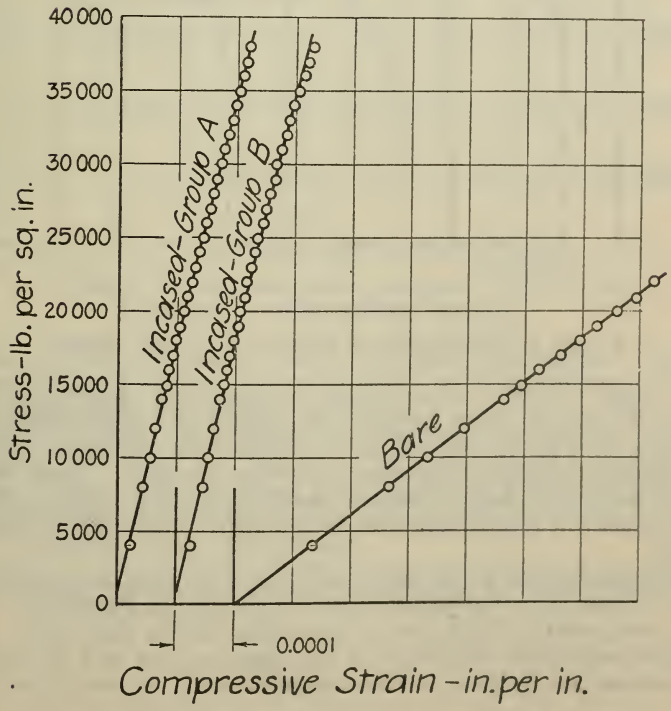

FIGURE 8.—*Average stress-strain diagrams for the columns*

TABLE 6.—*The ratios of stress to strain for the columns*

Column	Average ratio stress to strain
	Lbs./in.[2]
1	30, 800, 000
2	30, 700, 000
3	29, 600, 000
Average	30, 400, 000
A1	170, 800, 000
A2	179, 200, 000
A3	154, 200, 000
Average	168, 100, 000
B1	191, 500, 000
B2	154, 200, 000
B3	182, 400, 000
Average	176, 000, 000

The stress-deflection diagrams for the bare columns are shown in Figure 9. There was no measurable deflection up to a stress of

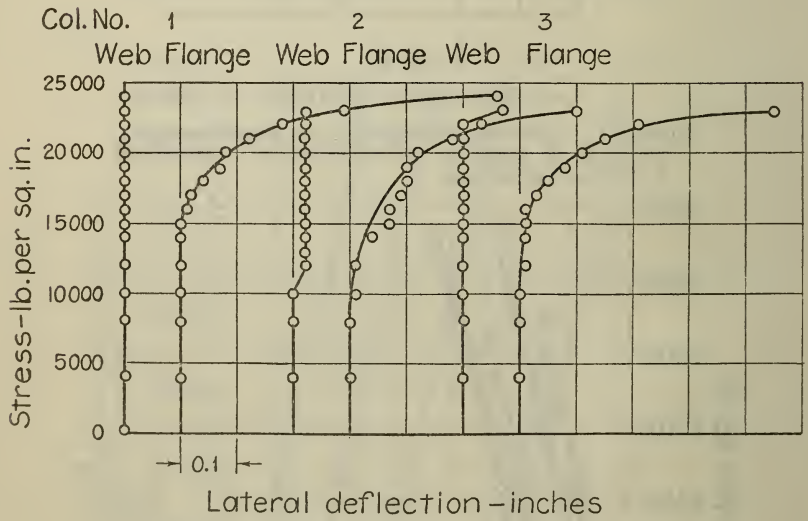

FIGURE 9.—*Stress-deflection diagrams of the bare columns*

about 10,000 lbs./in.[2]. For higher stresses the deflection in the plane of the web of the H section was negligible, but the deflection at midlength in a plane perpendicular to the web increased more and more rapidly until the maximum load was reached. There were no signs of secondary failure such as buckling of the flanges or webs. These curves are typical of those for columns which fail by deflecting laterally.

The stress-deflection curves for the A incased columns are shown in Figure 10 and those for the B incased columns in Figure 11. There was no measurable deflection in the plane of the side (face) of the walls and very little deflection in the plane of the end of the walls. These curves are typical of stress-deflection curves for short, sturdy

columns which deflect little before the maximum compressive strength of the material is reached.

When discussing the strength of steel columns it is convenient to use the column "efficiency." This term is defined by W. C. M. Pettingill [3] as the ratio of the column strength (average maximum compressive stress) to the compressive yield point of the material. When comparing the efficiencies of columns it has been found satisfactory to use the tensile yield point of the material which is obtained

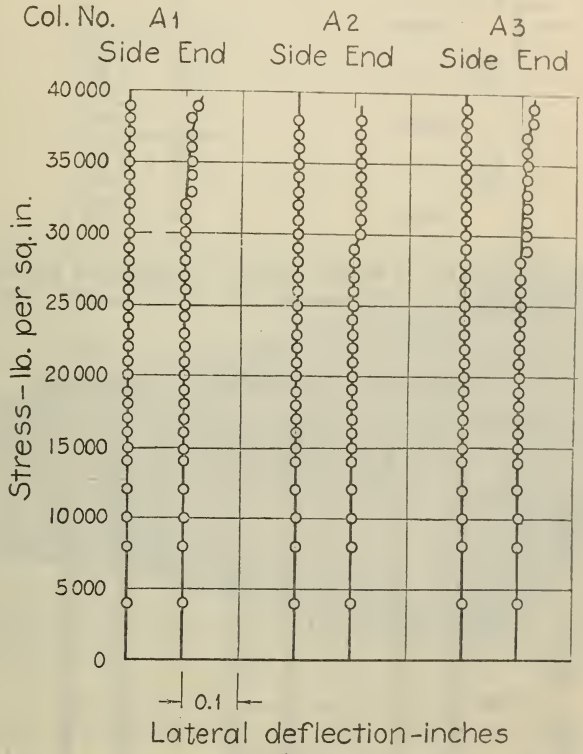

FIGURE 10.—*Stress-deflection diagrams of the incased columns, Group A*

in routine tests instead of the compressive yield point. The same method of determining the yield point should, of course, be used when comparisons of this sort are made.

The efficiencies of the columns and of the short lengths reported in this paper are given in Table 7. The yield point was taken as the average of the yield points of the flange and web coupons without considering the weighting; that is, the proportion of the total cross section represented by each coupon.

[3] R. V. Southwell, The Strength of Struts, Aircraft Engineering, vol. 1, pp. 136–138, 1920. Also see footnote 1, p. 128, and refer to pages 36 and 37 of that reference.

TABLE 7.—*Efficiencies of the columns and of the short lengths*

Column	Column efficiency of bare and incased columns	Column efficiency of short lengths
1	0.559	----------
2	.484	----------
3	.550	----------
Average	.531	----------
A1	.973	1.002
A2	1.064	1.108
A3	1.015	.955
Average	1.017	1.022
B1	.995	1.041
B2	1.017	1.002
B3	1.015	1.042
Average	1.009	1.028

For the short lengths, in which lateral buckling did not appreciably affect the strength, the efficiencies are very nearly 1.00. As the

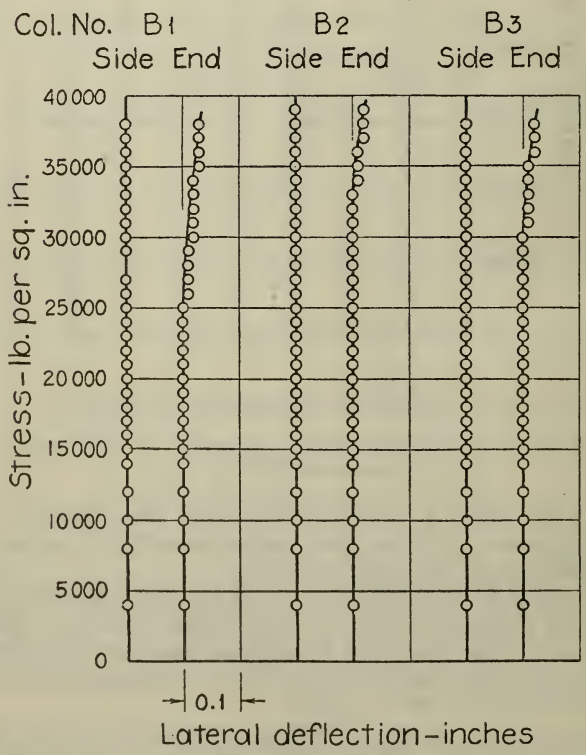

FIGURE 11.—*Stress-deflection diagrams of the incased columns, Group B*

slenderness ratio increases, the effect of lateral buckling gradually lowers the efficiency. For low-carbon structural steel columns having slenderness ratios greater than about 95 for "round end"

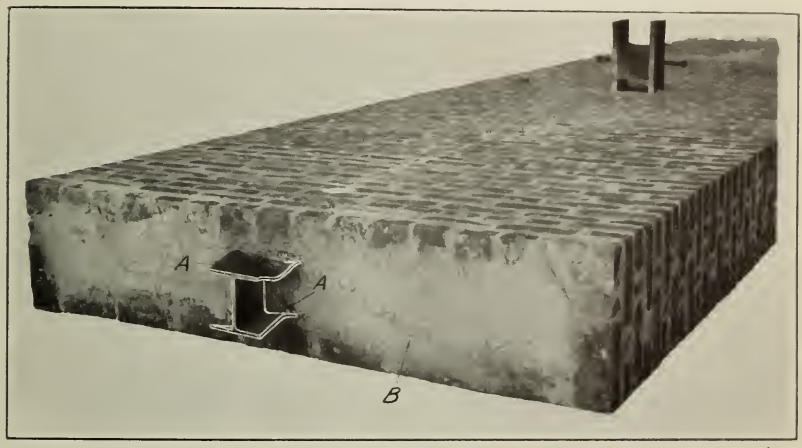

FIGURE 12.—*The top of column A1 after test*

Note the buckling of the unincased length of the column at *A* and the crack between the brick wythes at *B*.

FIGURE 13.—*An incased column being removed from the testing machine after test*

No bricks fell from the wall when it was laid horizontally on the floor.

columns or greater than about 190 for "fixed end" columns, the failure is almost entirely flexural (Euler failure) and the strength depends almost entirely upon Young's modulus of elasticity of the material, the end conditions, and the slenderness ratio. All the efficiencies of the incased columns and of the short lengths cut from the steel columns are of the order of magnitude (approximately 1.00) to be expected for "round end" steel columns having slenderness ratios less than 30 or for "fixed end" steel columns having slenderness ratios less than 60. All the efficiencies of the bare columns are of the order of magnitude (approximately 0.50) to be expected for "round end" steel columns having slenderness ratios of about 110 or for "fixed end" steel columns having slenderness ratios of about 220.[4]

The many tests which have been made on columns indicate that a "flat end" column and a "round end" column having about 0.6 the length may be expected to support the same maximum load.[5] This fraction of the length (0.6) may vary somewhat, depending upon the distribution of the load over the end of the columns and other conditions which affect the degree of end restraint.

The bare columns having a slenderness ratio of 183 and "flat ends" probably would have about the same efficiency as a "round end" column having a slenderness ratio of about 110.

All of the incased columns failed by yielding, followed by local buckling of the unincased portion of the steel columns. The lateral deflection of the brick walls under the maximum load did not exceed 0.03 inch for any of the incased columns. The brick walls, therefore, restrained the steel columns and prevented failure by lateral buckling. The only failure in the brick walls was at the top where small cracks appeared between the brick wythes, apparently caused by the local buckling of the unincased portion of the steel column. The buckling of the unincased portion of the steel columns is shown in Figure 12 at *A*, and the cracks in the vertical mortar joints at *B*. One of the incased columns after test (weight 19,000 pounds) is shown in Figure 13. The fact that all the incased columns could be handled in this way by the overhead crane without any indication of failure in the brick walls is evidence that the brick walls were not appreciably affected by the loads applied during the tests.

For the bare columns the entire load was carried by the steel. The average ratio of stress to strain for these columns, Table 6, is therefore, the average compressive modulus of elasticity of the steel in the bare columns. The values are as close to the usual value (30,000,000 lbs./in.^{2}) for mild steel as was to be expected from tests made with these specimens and apparatus.

For the incased columns in the 150-inch gage length a portion of the load was transferred to the brick walls by the bond between the walls and the steel. On the assumption that the average compressive modulus of the steel in the incased columns was 30,000,000 lbs./in.^{2}, 30,000,000 divided by the computed ratio of stress to strain gives the portion of the applied load which was carried by the steel column. The remainder of the load was carried by the brick wall. These values are given in Table 5. The average load in the steel over the

[4] See footnote 1, p. 128, and refer to Fig. 13, p. 43, of that reference.
[5] Columns, Salmon, pp. 171–174.

150-inch gage length in no case exceeded 20 per cent of the applied load.

The values given in Table 5 show, also, that the orientation of the steel column in the brick wall had no appreciable effect upon the strength of the incased columns.

From the fact that the steel in the incased columns carried only a small proportion of the load, it is evident that a more comprehensive investigation should be made in an attempt to develop a load-carrying structure which would also serve to inclose the building. There is apparently no reason why the load cannot be economically applied directly to the top of a brick wall of suitable material and workmanship. The steel in the incased columns undoubtedly served to reinforce the brick masonry, but the question arises whether a number of vertical steel bars of small diameter placed near both faces of the wall would not have provided much greater restraint against lateral buckling of the brick wall and at the same time reduced the cost. The results of further investigations are needed to give a definite answer to this question. Attention should also be given to developing methods for tying the structure supported by the brick wall into the top of the wall so that they will act as a unit under any forces which are likely to come upon them.

These results show that the compressive strength of long steel columns, incased as these were in brick walls, is much greater than if the columns are not incased, but they do not indicate the final solution of the problem of using brick masonry and steel to build more safely and more economically.

V. CONCLUSIONS

Three H-shaped steel columns, 23 feet long, 6 inches in depth, weighing 20 pounds per foot, and having flat ends, were tested in compression. Six similar steel columns incased in brick walls, 14 inches thick, 6 feet long, and extending up to about 8 inches from the top of the steel columns were also tested in compression.

With steel columns of this size, incased in brick walls of the quality used:

1. The brick walls effectively prevented lateral buckling of the steel column so that the maximum compressive load for the structure was, for all practical purposes, the maximum compressive load for the portion of the steel column above the wall, which in turn was, for all practical purposes, the tensile yield point of the steel multiplied by the cross-sectional area of the steel column.

2. The maximum compressive load carried by the short unincased portion of the steel columns was much greater than the maximum compressive load carried by the long unincased columns (slenderness ratio of 183) which failed by lateral buckling.

3. The lateral deflection of the incased columns was negligible because in no case did it exceed 0.03 inch under the maximum load. The brick masonry showed only small cracks at the top near the steel columns.

4. Over the gage length of 150 inches at midheight of the incased columns, the steel carried, on the average, less than 20 per cent of the applied load.

5. The orientation of the steel column with respect to the face of the brick wall had no effect upon the strength of the incased columns.

WASHINGTON, November 23, 1932.