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SOME TESTS OF STEEL COLUMNS INCASED IN CONCRETE

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ABSTRACT

In cooperation with the Bridge Department of the Port of New York Authority, the National Bureau of Standards tested four carbon-steel columns incased in reinforced concrete to determine their strength and stiffness. The steel columns were duplicates of columns TC1 and TC2 reported in Bureau Research Paper RP831.

The temperatures in the columns were measured during the aging of the concrete. The readings of telemeters attached to the steel members indicated that no appreciable stress in the steel members was caused by the aging of the concrete. When the columns were loaded, the telemeter stresses were always less than the quasi-stress obtained by dividing the load by the cross-sectional area of the steel members. The concrete, therefore, carried a portion of the load.

At the column yield strength, the load on the incased columns was 51 percent greater than the load on the unincased columns. At the final maximum load the load on the incased columns was 42 percent greater than the load on the unincased columns.

The greatest stress in the reinforcement rods was 10 kips/in^2.[1]

The lateral deflection was very small until the load on the column approached the maximum. At or about the maximum load, the columns having a length of 24 feet deflected about 6 inches, and large pieces of concrete fell from the columns.

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[1] 1 kip=1,000 lb.

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I. INTRODUCTION

At the request of the Bridge Department of the Port of New York Authority, the National Bureau of Standards, cooperated in an investigation of the strength and behavior, under load, of four steel columns incased in reinforced concrete. The results are directly comparable with those for columns given in National Bureau of Standards Research Paper RP831, Tests of Steel Tower Columns for the George Washington Bridge,[1] because the steel portions of the incased columns were duplicates of the carbon-steel columns TC1 and TC2 in the previous investigation.

The incasement of steel columns in reinforced concrete has been used to increase their strength and stiffness, and to protect them from corrosion. This investigation was undertaken to determine the increase in strength and stiffness.

II. THE SPECIMENS AND THE METHOD OF TESTING

1. THE COLUMNS

(a) DESCRIPTION

The nominal dimensions of the columns are given in table 1.

TABLE 1.—*The nominal dimensions of the columns*

Symbol	Number of specimens	Kind of steel	Cross-sectional area of steel	Length	Remarks
			in.²	ft	
IC1, IC2	2	Carbon	[a] 159	24	Loaded on steel only.
IC3, IC4	2	Carbon	[a] 159	24	Loaded on both steel and concrete.

[a] The concrete incasement was 37.5 in. square, with corners slightly beveled. The cross-sectional area of the concrete was 1,245 in².

[1] J. Research NBS **15**, 317 (1935) RP831.

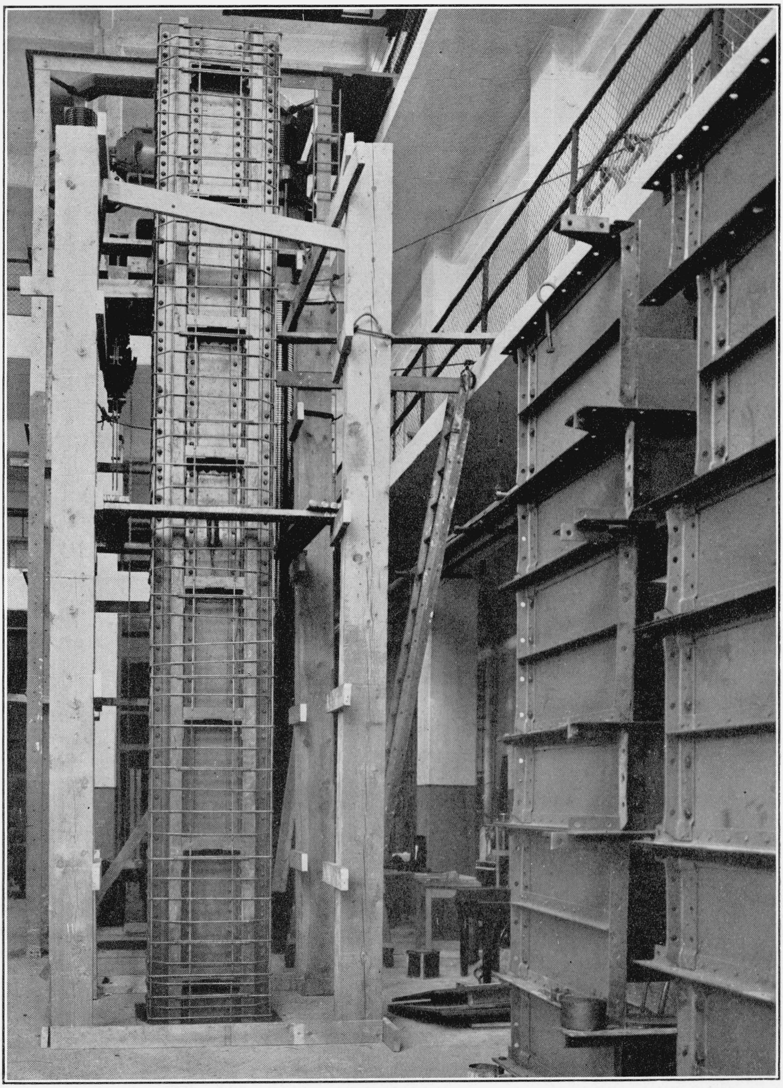

FIGURE 1.—*Column IC3 with the reinforcement in place.*

Note the telemeters covered with rubber pads. The steel forms are shown in the right foreground.

There were 4 columns fabricated by riveting from carbon steel. They were of the same size and shape as the steel columns, TC1 and TC2 of Research Paper RP831. The columns were designated IC, followed by the numerals 1, 2, etc., for the individual columns.

The longitudinal pieces of each column, that is, the longitudinal plates and angles, were cut as shown in the cutting diagram in figure 1 of RP831. Each longitudinal piece of the column was matchmarked to correspond with a coupon cut from the same plate or angle, and its location in the column was recorded.

(b) REINFORCEMENT

The reinforcing rods were plain bars (0.5 in. diameter) of structural-steel grade rolled from new billets. Tensile and bending tests were made in accordance with ASTM Standard Specifications for Billet-Steel Concrete Reinforcement Bars, A15–14.[2]

A column with the reinforcement in place is shown in figure 1, and the layout of the reinforcement is shown in figure 2. The tie rods, A and B, were bent to shape, lapped 1 ft 8 in. at the ends and seized with wire. There was a splice in every fourth rod on each side of the column.

(c) CONCRETE

The materials for the concrete were purchased locally. All of the cement for the columns was delivered before beginning construction. The cement complied with the requirements of ASTM Standard Specification and Tests for Portland Cement, C 9–26.[3]

The aggregate consisted of Potomac River sand and gravel such as is used locally in building construction. The aggregates were delivered as needed, and although they were obtained from the same source, the gradings of the sand and the gravel were not the same in all deliveries.

The proportions of the ingredients of the concrete were controlled to maintain a constant water-cement ratio and approximately a constant slump. On account of the differences in the gradings of the sand and gravel the proportions of cement to aggregates ranged from 1 : 2.1 : 3.6 to 1 : 2.24 : 3.92 by separate volumes of the cement and of the dry, compacted sand and gravel, respectively. The total water content of each batch of concrete, including the moisture in the aggregate, was 7.1 gal per sack of cement, plus an allowance for the absorption of the aggregate equal to 1 percent of the dry weight of the aggregate. The slump of the concrete was usually between 3 and 5 in. when tested in accordance with the ASTM Tentative Method of Test for Consistency of Portland Cement Concrete, D 138–26 T.[4]

Twelve concrete cylinders, 6 in. in diameter and 12 in. long, were made from the concrete for each of the four columns. They were placed in the damp storage rooms in the concrete laboratory when they were 24 hours old and remained there until they were removed for testing.

(d) PLACING THE CONCRETE

After the reinforcement was placed, the steel forms shown at the right in figure 1 were placed around the column and bolted together. The sections were from 13.5 to 24 in. high, and each section was filled before the next section was attached to it.

[2] Stand. Am. Soc. Testing Materials (I) p. 132 (1927).
[3] Stand. Am. Soc. Testing Materials (II) p. 23 (1927).
[4] Proc. Am. Soc. Testing Materials (I) 21, 874 (1926).

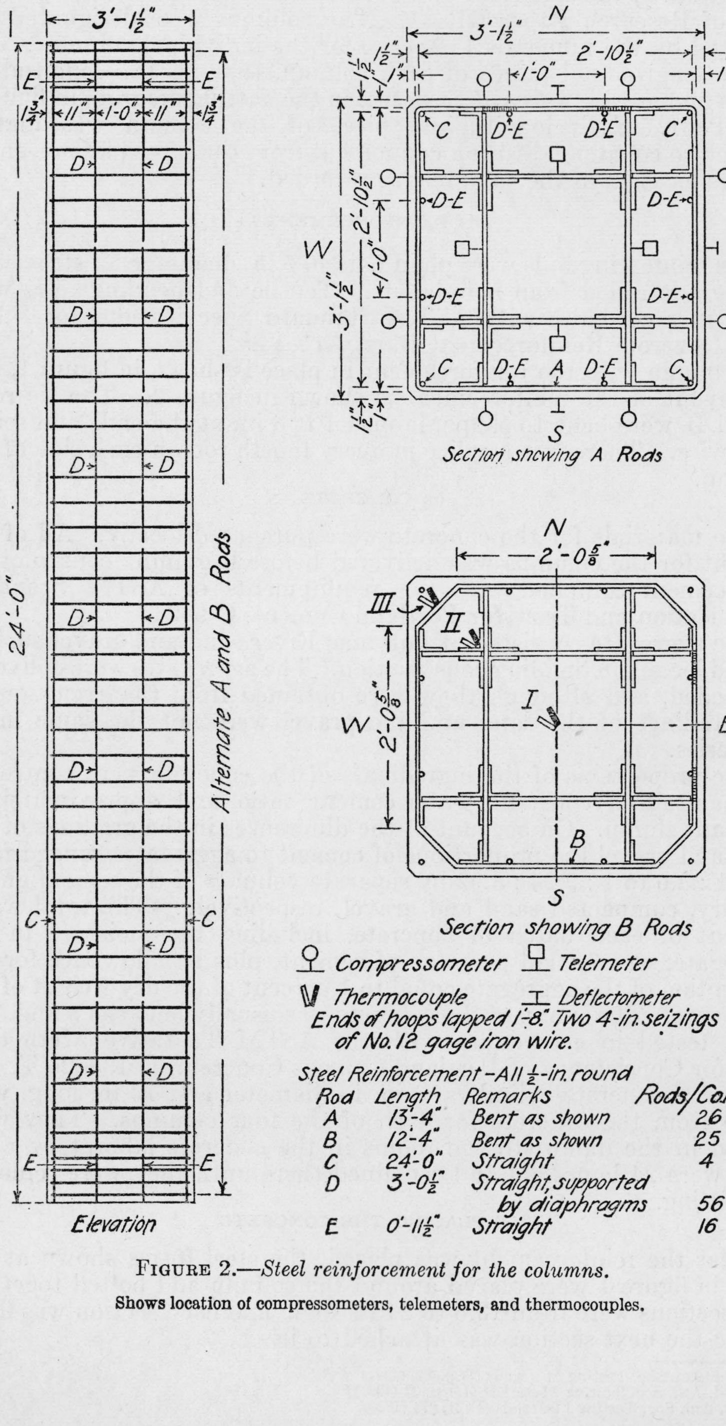

FIGURE 2.—*Steel reinforcement for the columns.*

Shows location of compressometers, telemeters, and thermocouples.

FIGURE 3.—*An incased column and one being incased.*

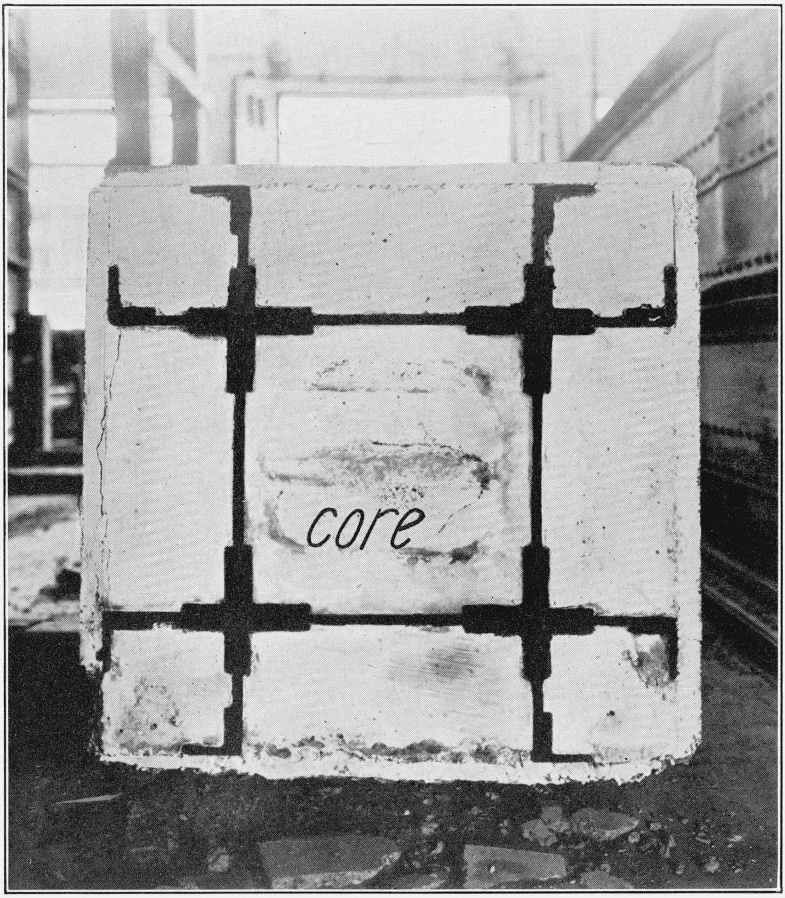

FIGURE 4.—*Lower end of incased column* IC2 *after test.*

The concrete was mixed for 2 minutes in a batch mixer. It was carefully rodded into place to produce a dense concrete free from honeycombing. A light air hammer was also applied to the forms to assist in compacting the concrete. The concrete for the core was deposited through a pipe, the lower end of which was never more than 2 ft above the concrete in the core. The concrete inside and outside the steel column was maintained at as nearly the same level as possible. An incased column and one being incased are shown in figure 3.

For columns IC3 and IC4 the concrete was flush with the ends of the steel column. For the columns IC1 and IC2 the ends of the steel column projected 0.5 in. from the ends of the concrete. Wood fillers were used temporarily at the lower end of the column and the forms filled to 0.5 in. below the upper end of the steel column.

The forms remained on the columns for at least 48 hours after the concrete was placed. The columns were tested at ages from 71 to 74 days. The lower end of column IC2 after test is shown in figure 4.

(e) MEASUREMENTS MADE DURING THE AGING OF THE CONCRETE

Telemeters having a gage length of 8 in. were placed on vertical gage lines at midheight of each column at the four locations shown in figure 2. For column IC1 only, there were four additional telemeters. They were placed in the east and west locations shown in figure 2, two at 2 ft, 6 in. from the lower end and two at the same distance from the upper end. The telemeters were attached to the steel plates before the reinforcement was placed. They were protected by heavy brass castings attached to the column by screws. There was a gasket between the casting and the column. Pads of soft rubber were placed over the castings and secured by wires. A waterproof cable was connected to each telemeter and led out through a hole in the steel form. The readings of the telemeters were recorded during the aging of the concrete.

Nine copper-constantan thermocouples were placed in the concrete at the locations shown in figure 2, three at midheight, and three at 4 ft from each end of the column. It is believed that the error in the temperature determined from the thermocouple readings did not exceed 1° F.

After column IC3 had been incased the room temperature and the readings of the thermocouples and of the telemeters were recorded every half hour for 2 days. Thereafter they were recorded daily (except on holidays) until the column was tested. For the other three columns the readings were recorded daily.

(f) TESTING PROCEDURE

All the columns were tested as flat-end columns by the use of the equipment and methods described in RP831. Columns IC1 and IC2 loaded on the steel only were tested in the same way as columns TC1 and TC2. For the columns IC3 and IC4 loaded on both steel and concrete the method of testing was similar, except that a thin coat of plaster of paris was used between the ends of the column and the bearing plates. An incased column in the testing machine is shown in figure 5.

(1) *Compressometers.*—The compressometers used for the tower columns (gage length 20 ft) were used to measure the shortening of

the concrete under load. The locations of the eight compressometers are shown in figure 2.

(2) *Telemeters.*—The locations of the telemeters are described under section II, 1, (e), page 679.

(3) *Lateral deflection.*—The lateral deflection of the column was measured by the use of the taut wire and mirror-scale deflectometer. The distance between the supports for the wire was 20 ft, and the middle of the wire was at midheight of the column. The locations of the three deflectometers are shown in figure 2. One division on the scale was 0.1 in. and readings were estimated to 0.1 of a division.

(4) *Strain in the reinforcement.*—The strain in the four horizontal reinforcing rods nearest midheight of the column was measured manually by the use of a Berry strain gage having an 8-in. gage length. There was one gage line on each rod, each on a different side of the column.

(5) *Loading.*—The columns were loaded repeatedly as follows:

Loading number	Minimum	Maximum
	kips	kips
1	0	7, 314
2 to 20, incl	5, 565	7, 314
21	0	7, 314
22 to 25, incl	5, 565	7, 314
26	0	Failure

For the twenty-sixth loading the load was increased by increments, and the compressometers and the lateral deflections were read for each increment of load. This procedure was continued until the deflection of the column brought it into contact with some of the compressometers. The compressometers were then removed, and the pump of the testing machine operated at a constant speed. Load readings were taken at intervals of 1 minute until the load had reached a maximum and then decreased. The telemeter reading at midheight of column IC3 showed that the stress in the steel was 18 kips/in.^{2} for a load of 5,565 kips and 30 kips/in.^{2} for a load of 7,314 kips.

2. COUPONS

(a) GENERAL

The coupons for the steel portion of the incased columns were taken and tested in tension as described in RP831. The axis of each coupon was parallel to the direction of rolling (axis) of the plate or angle.

The coupons were standard ASTM tensile specimens for plates, shapes, and flats.[5] These coupons had a gage length of 8 in., a width of 1.5 in., and the thickness was that of the material as rolled.

(b) EXTENSOMETER

The strains in some coupons were measured by the use of a Ewing extensometer having a gage length of 8 in. One division on the scale of this instrument corresponded to a strain of 0.000 025 in the coupon. The readings were estimated to 0.1 division.

[5] Figure 1, Stand. Am. Soc. Testing Materials (I) p. 68 (1933).

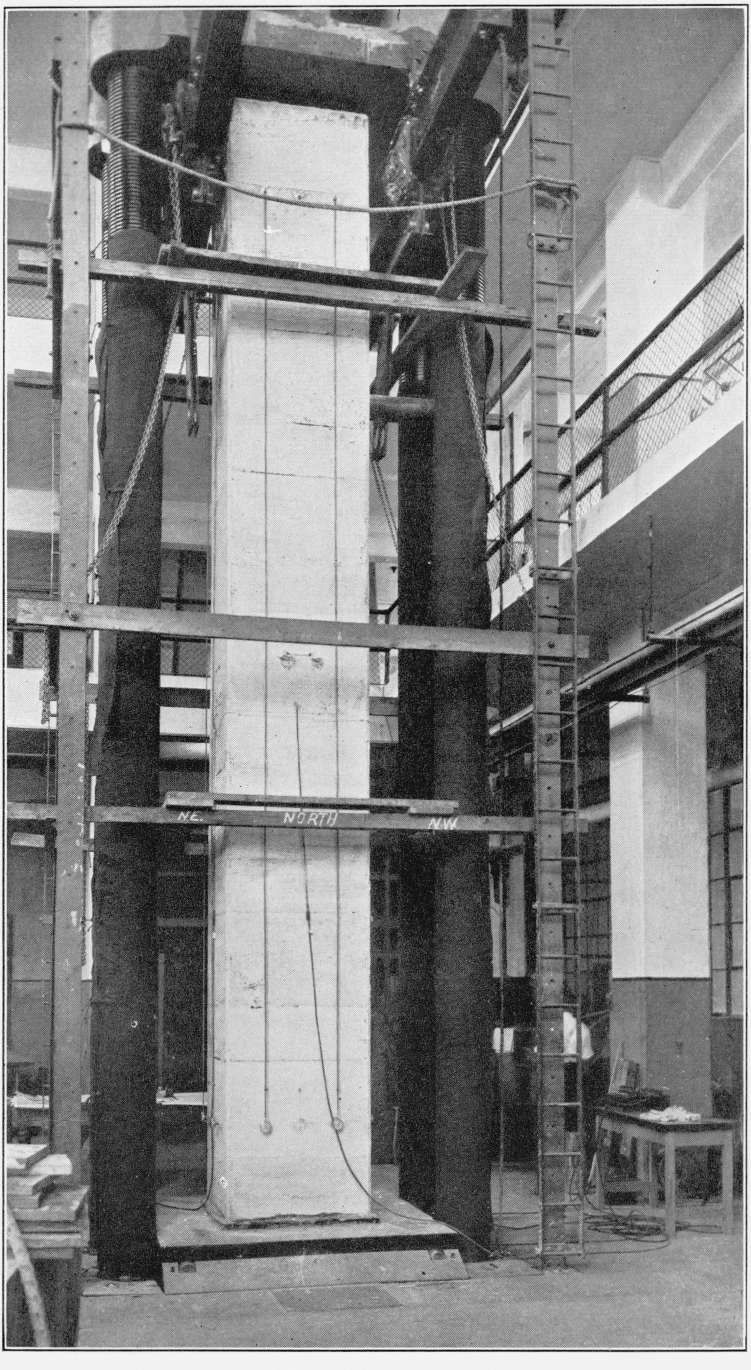

FIGURE 5.—*Column* IC4 *in the testing machine.*

(c) YIELD STRENGTH

For the coupons on which the extensometer was used the yield strength was taken as the stress for which the strain was 0.002 greater than the strain computed from the stress and Young's modulus of elasticity. The values obtained in this way agreed closely with those obtained by the drop of the beam of the testing machine. For the coupons on which the extensometer was not used the yield strength was determined by the drop-of-beam method.

(d) TESTING MACHINE

The coupons were tested in a screw-power, beam-and-poise machine having a capacity of 100 kips.

(e) SPEED OF THE MOVABLE PLATEN

For the coupons on which a Ewing extensometer was used the speed of the movable platen of the testing machine was 0.04 in./min until the stress was about three-quarters of the yield strength. For higher stresses the speed was 0.01 in./min. After the extensometer was removed the speed was 0.4 in./min until the coupon ruptured.

For the coupons on which the extensometer was not used, the speed was 0.04 in./min until the drop of the beam was observed. For higher stresses the speed was 0.4 in./min.

III. RESULTS FOR THE AUXILIARY TESTS

1. COUPONS

(a) TENSILE TESTS

The results for the tensile tests of the coupons are given in table 2. The properties of the material are average values for the longitudinal members of the same size and shape. The values of the yield strength are drop-of-beam values. Each weighted average was obtained by weighting the value for a coupon in the ratio of the cross-sectional area of the main member which it represented to the total cross-sectional area of the column.

TABLE 2.—*Results for the tensile tests of coupons*

COLUMN IC1 CARBON STEEL

Shape	Nominal size	Number of coupons	Yield strength, average	Tensile strength, average	Elongation in 8 inches, average	Reduction of area, average
	in.		kips/in.²	kips/in.²	%	%
2 plates	34⅜ by ⅝	2	32.9	61.0	30.8	53.5
2 plates	17 by ⅝	2	32.1	55.2	30.6	59.6
4 plates	7⅝ by ⅝	4	31.2	55.2	32.9	60.6
4 angles	4 by 4 by 9/16	4	32.4	56.0	32.6	58.3
8 angles	4 by 3 by ½	8	33.6	59.6	28.0	53.3
12 angles	3 by 3 by ½	12	38.6	63.2	28.5	53.7
Weighted average			33.8	59.2	30.3	55.7

TABLE 2.—*Results for the tensile tests of coupons*—Continued

COLUMN IC2 CARBON STEEL

Shape	Nominal size	Number of coupons	Yield strength, average	Tensile strength, average	Elongation in 8 inches, average	Reduction of area, average
	in.		kips/in.²	kips/in.²	%	%
2 plates	34⅜ by ⅝	2	32.9	60.1	30.0	54.1
2 plates	17 by ⅝	2	32.9	54.7	32.8	65.0
4 plates	7⅝ by ⅝	4	31.6	55.2	31.1	60.7
4 angles	4 by 4 by 9/16	4	32.4	55.9	32.9	56.9
8 angles	4 by 3 by ½	8	33.4	58.3	28.8	56.6
12 angles	3 by 3 by ½	12	38.6	63.5	28.2	51.9
Weighted average			34.0	58.8	30.2	56.6

COLUMN IC3 CARBON STEEL

Shape	Nominal size	Number of coupons	Yield strength, average	Tensile strength, average	Elongation in 8 inches, average	Reduction of area, average
2 plates	34⅜ by ⅝	2	33.1	59.6	31.1	54.9
2 plates	17 by ⅝	2	33.2	55.6	30.0	55.3
4 plates	7⅝ by ⅝	4	30.8	55.7	33.2	60.8
4 angles	4 by 4 by 9/16	4	33.4	57.3	30.7	57.1
8 angles	4 by 3 by ½	8	32.9	58.4	28.1	55.3
12 angles	3 by 3 by ½	12	38.8	63.7	29.0	53.6
Weighted average			34.0	59.0	30.2	55.7

COLUMN IC4 CARBON STEEL

Shape	Nominal size	Number of coupons	Yield strength, average	Tensile strength, average	Elongation in 8 inches, average	Reduction of area, average
2 plates	34⅜ by ⅝	2	33.0	60.6	29.0	55.8
2 plates	17 by ⅝	2	34.4	56.6	32.6	59.4
4 plates	7⅝ by ⅝	4	32.2	56.4	33.2	61.0
4 angles	4 by 4 by 9/16	4	32.9	57.2	32.2	56.2
8 angles	4 by 3 by ½	8	33.0	58.3	27.6	54.0
12 angles	3 by 3 by ½	12	39.2	63.4	28.3	52.3
Weighted average			34.4	59.4	30.0	55.9

A typical Ewing stress-strain graph for this carbon steel is shown in Figure 9 of RP831.

The speed of the movable head was much lower than is customarily used when determining the yield strength. If the yield strength is determined by the drop of the beam, the value is dependent on the speed—the higher the speed, the higher the yield strength.[6] For these coupons the rate at which the stress was increased is more nearly the rate for the columns than the rate customarily used for coupons.

(b) CHEMICAL COMPOSITION

Chemical analyses were made by the Chemistry Division of the Bureau of samples from coupons having the highest and the lowest tensile strength for each thickness and each shape. The results are given in table 3.

[6] Proc. Am. Soc. Testing Materials (I) 28, 105 (1928).

TABLE 3.—*Chemical composition of the steels*

Description of samples			Chemical composition				
Thickness	Shape	Tensile strength	Carbon	Manganese	Phosphorus	Sulphur	Silicon
in.		kips/in.²	%	%	%	%	%
½	Angle	55.1	0.18	0.56	0.015	0.033	0.03
½	do	66.9	.24	.55	.018	.037	.03
9/16	do	53.3	.14	.46	.012	.031	.02
9/16	do	61.0	.18	.53	.026	.048	.04
⅝	Plate	53.0	.14	.33	.009	.023	.11
⅝	do	61.6	.19	.46	.019	.022	.11

2. CONCRETE

The cylinders (6 in. in diameter by 12 in. in length) of the concrete for each of the columns were separated into four groups of three cylinders each. One group was tested at each of the following ages: 7 days, 28 days, 2 months, and 6 months. The results are given in table 4. The columns were tested at an age of about 70 days. At that time the strength of the concrete was very nearly the same as the strength of the cylinders at an age of 2 months.

TABLE 4.—*Strength of concrete cylinders*

[Cylinders 6 in. in diameter by 12 in. long]

Column	Number of cylinders tested for each age	Average compressive strength at age of—			
		7 days	28 days	2 months	6 months
		kips/in.²	kips/in.²	kips/in.²	kips/in.²
IC1	3	1.55	2.52	2.89	3.80
IC2	3	1.47	2.04	2.81	2.83
IC3	3	1.77	3.18	3.23	3.47
IC4	3	1.83	2.73	2.91	3.28
Average compressive strength for the four columns		1.66	2.62	2.96	3.35

3. REINFORCEMENT

The carbon and phosphorus content and the tensile properties of the reinforcing bars are given in table 5.

TABLE 5.—*Carbon and phosphorus content and tensile properties of the reinforcing bars*

[Plain bars, 0.5 in. diameter]

Specimen	From reinforcement [1]	Chemical content		Tensile properties			
		Carbon	Phosphorus	Young's modulus of elasticity	Yield strength	Tensile strength	Elongation in 8 inches
		%	%	kips/in.²	kips/in.²	kips/in.²	%
A1	A	0.13	0.108	29,200	51.6	73.4	23.1
A2	A			29,800	50.2	70.6	23.7
B1	B	.11	.110	29,700	45.6	65.3	24.2
B2	B			29,400	47.3	67.9	24.0
C1	C			30,000	47.1	66.0	24.8
C2	C			30,000	45.5	63.0	21.3
C3	C			28,700	40.3	55.7	28.2
C4	C			29,600	44.2	63.0	26.0

[1] The reinforcement is shown in figure 2.

IV. RESULTS FOR THE COLUMNS

1. STRESS IN THE STEEL MEMBERS CAUSED BY AGING OF THE CONCRETE

The telemeters attached to the steel members of the incased columns were read during the aging of the concrete (about 70 days) to determine the stresses in the members. With one exception, these readings did not differ from the initial readings before the columns were incased by more than 1 division of the scale on the milliammeter, indicating that the stress did not exceed 3.75 kips/in ^{2}. The last reading on column IC1 exceeded the initial reading by 1.3 divisions, indicating a stress of about 4.88 kips/in ^{2}. Some of the readings of the telemeters indicated tensile and some compressive stresses, and consequently the readings do not show that aging of the concrete caused appreciable stresses in the steel members.

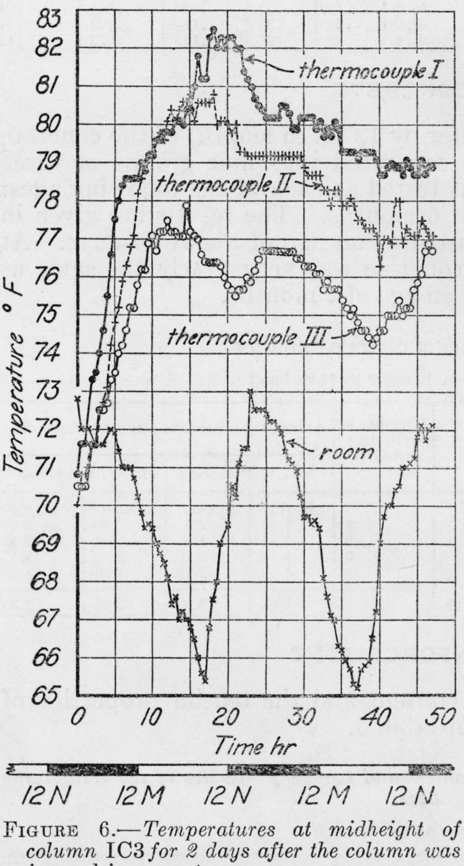

FIGURE 6.—*Temperatures at midheight of column* IC3 *for 2 days after the column was incased in concrete.*

Figure 2 shows the locations of the thermocouples.

2. TEMPERATURE DURING AGING OF THE CONCRETE

The temperatures at midheight of column IC3 for 2 days after the column was incased in concrete are shown in figure 6. During this period the temperatures in the concrete were greater than the room temperature. The temperatures in the concrete were a maximum between 14 and 18 hours after the column was incased. The temperatures were greatest at the axis of the column (thermocouple I). After the temperature at the axis reached a maximum, it decreased continuously with variations which did not exceed 1° F. The temperature at thermocouple III, nearest the sides of the column increased and decreased appreciably with the changes in the room temperature, but there was a time lag of from 2 to 5 hours.

The temperatures at the axis at midheight of colum IC2 from the time the column was incased until it was tested are shown in figure 7.

The temperature-time graphs for the other columns were similar to figure 7. After about 5 days the temperature of the concrete increased and decreased noticeably with changes in the room temperature, and after about 10 days the increase and decrease became marked, but there was a time lag of from 1 to 5 days. The temperature of the concrete became about equal to the room temperature after 45 days. The temperature-time graphs for the concrete near the top and the bottom of the columns were similar to figure 7, except that there was less difference between the temperature of the concrete and the room temperature.

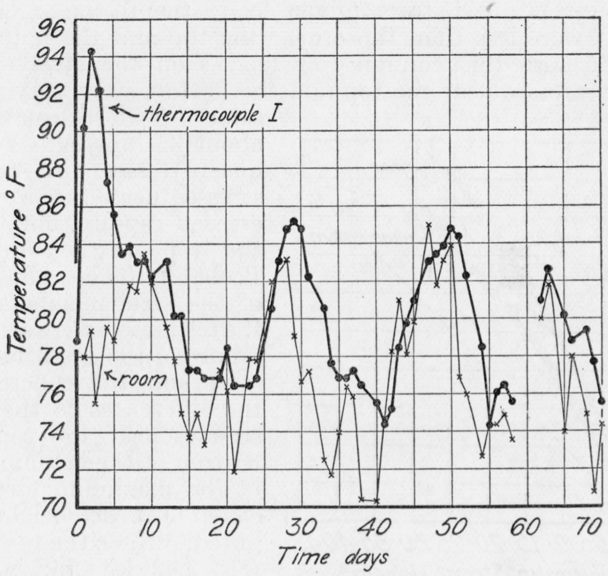

FIGURE 7.—*Temperatures in the axis (thermocouple* I) *at midheight of column* IC2 *from the time the column was incased until it was tested.*

Figure 2 shows the locations of the thermocouples. Usually the temperatures were recorded at 9:30 a. m.

3. QUASI-STRESS

For these columns it is convenient to consider as a quasi-stress in the steel members, the stress obtained by dividing the load on the column by the transverse sectional area of the steel members.

4. TELEMETER STRESS

For column IC1 (loaded on steel only) there were telemeters near the top, the bottom, and at midheight. The strains obtained from the telemeter readings were averaged for the telemeters at each height. The average telemeter stress was computed from the average telemeter strain and the average Young's modulus of elasticity (29,100 kips/in ^{2}) for the coupons. The telemeter stresses for column IC1 are shown in figure 8. The dashed line a–b shows what the relation would be between the telemeter stress and the quasi-stress if the steel members carried all of the load and none of it were carried by the concrete. The telemeter stress was always less than the quasi-stress, indicating that the concrete carried a portion of the load. Let the length of the abscissa for any point on line a–b represent the total load on the column, as, for example, c–d. Then at midheight the portion of the load carried by the steel members is represented by the line c–e and that carried by the concrete by line e–d. As the load increased, the steel members carried a greater proportion of the load.

For a given quasi-stress (given load) the telemeter stresses at midheight were less than those near the top and the bottom of the column because this column was loaded on the steel only. The telemeter stresses near the top and the bottom were very nearly the same up to a quasi-stress of about 35 kips/in^2. For greater quasi-stresses the telemeter stresses near the bottom increased rapidly, but those near the top decreased rapidly. Probably the local deformations of the steel members to which the telemeters were attached and failure of the bond between the steel and the concrete caused the differences in the telemeter stresses near the top and the bottom of the column.

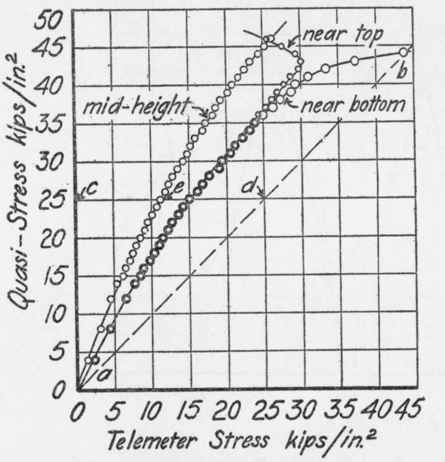

FIGURE 8.—*Average telemeter stresses for incased column* IC1 *loaded on steel only.*

The dashed line a–b shows what the relation would be between the telemeter stress and the quasi-stress, if the steel members carried all of the load.

The maximum stress in the concrete at each height was computed. Near the top this stress was 2.62 kips/in.^{2} when the quasi-stress was 46 kips/in^2. At midheight it was 2.56 kips/in.^{2} when the quasi-stress was 46 kips/in^2. Near the bottom it was 1.40 kips/in.^{2} when the quasi-stress was 36 kips/in^2.

The average telemeter stresses at midheight for each of the columns are shown in figure 9. The dashed line a–b shows what the relation would be between the telemeter stress and the quasi-stress if the steel members carried all of the load and none of it were carried by the concrete.

There was not much difference between the telemeter stresses for the columns loaded on steel only and those loaded on both steel and concrete. No explanation was found for the fact that the telemeter stresses at midheight for the columns loaded on both the steel and concrete were somewhat greater than those for the columns loaded on the steel only. For a quasi-stress of 20 kips/in.^{2}, about the design load, the telemeter stress was between 8 and 10 kips/in.^{2}; therefore at this load about one-half the load was carried by the steel members and one-half by the concrete.

5. STRAIN IN THE CONCRETE

(a) AVERAGE STRAIN

The average strains for the first loading computed from the readings of the compressometers attached to the concrete are shown in figure 10. The values for the unincased tower columns TC1 and TC2 are also shown. The strain in the incased columns was much

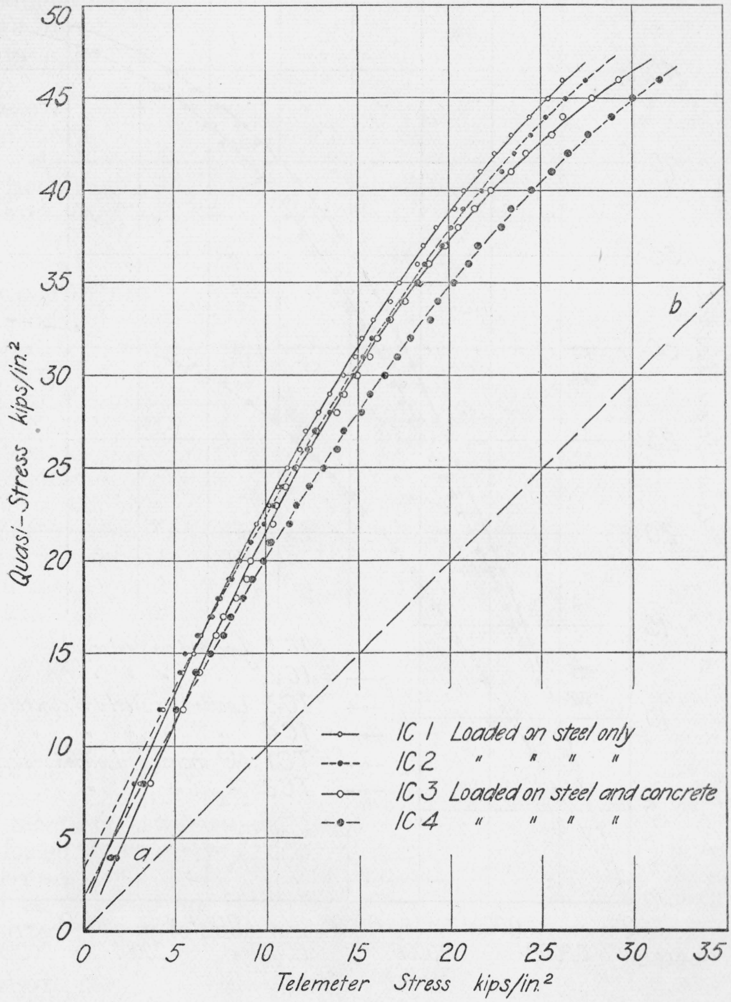

FIGURE 9.—*Average telemeter stresses at midheight for each of the columns.*

The dashed line a–b shows what the relation would be between the telemeter stress and the quasi-stress, if the steel members carried all of the load.

less than the strain in the unincased columns. For a stress of 20 kips/in.² in the unincased columns, about the design load, the strain was 0.0007. At this strain in the incased columns the quasi-stress was about 30 kips/in². Therefore, for the same shortening (strain) the load on the incased columns was about 50 percent greater than the load on the unincased columns.

(b) REPEATED LOADING

The average strains for column IC1 under repeated loading are shown in figure 11. For each application of load the strain increased. The increase was smaller and smaller as the loading number increased. The graphs for the other incased columns were similar to figure 11.

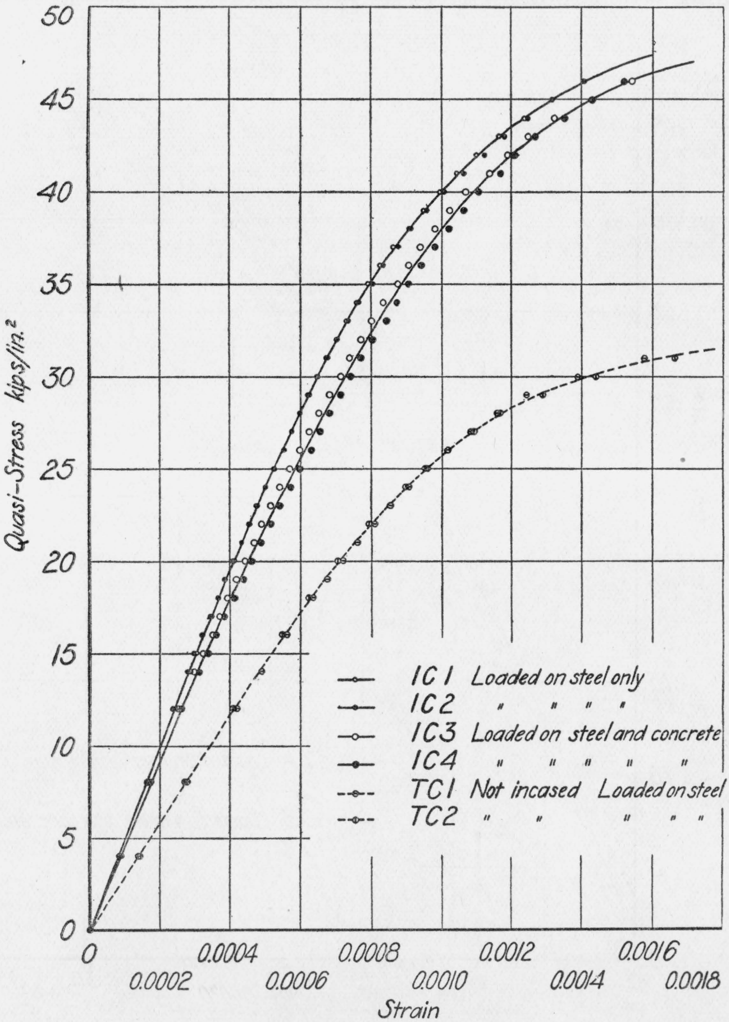

FIGURE 10.—*Average strains for the first loading computed from the readings of the compressometers attached to the concrete.*

The values for the unincased columns are also shown.

The average strain at each application of the loads given in section II, 1, (f), (5) for each of the incased columns is shown in figure 12. The strains in the columns loaded on the steel only were less than those for the columns loaded on both the steel and the concrete. The stress-strain graphs for the first and the twenty-sixth application of load for each of the incased columns are shown in figure 13. The permanent set after the twenty-fifth application of load was less in the columns loaded on the steel only than that for the columns loaded on both the steel and concrete.

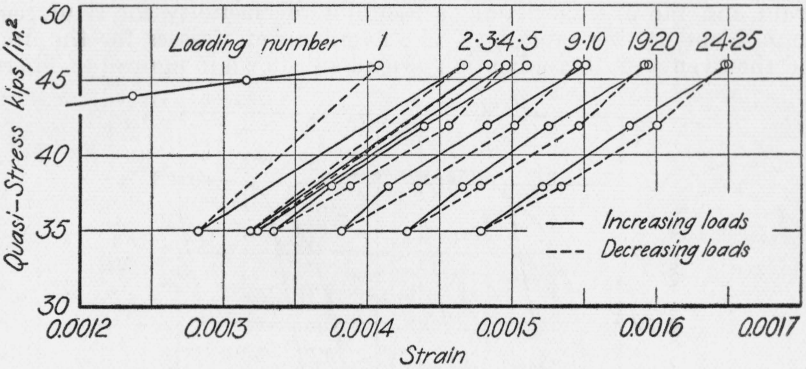

FIGURE 11.—*Average strains for column* IC1 *under repeated loading.*

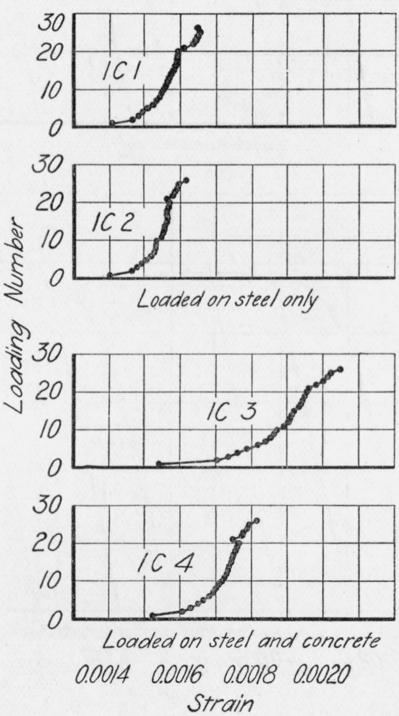

FIGURE 12.—*Average strain for each application of load for each of the columns.*

The results for these steel columns incased in concrete indicate that the steel and the concrete did not act as a unit when loads were applied repeatedly. It appears probable that this was due to the large proportion of steel and the insufficient bond between the steel and the concrete. Whether the behavior of these columns after many years under fluctuating loads can be predicted from the results of these tests is questionable.

6. STRESS IN REINFORCEMENT

The strains in the A and B tie rods near midheight of each column were averaged. The average stress was computed from the average strain and the average Young's modulus of elasticity for reinforcement A and B (see table 5). The average rod stresses for the first and the twenty-sixth application of load are shown in figure 14. There

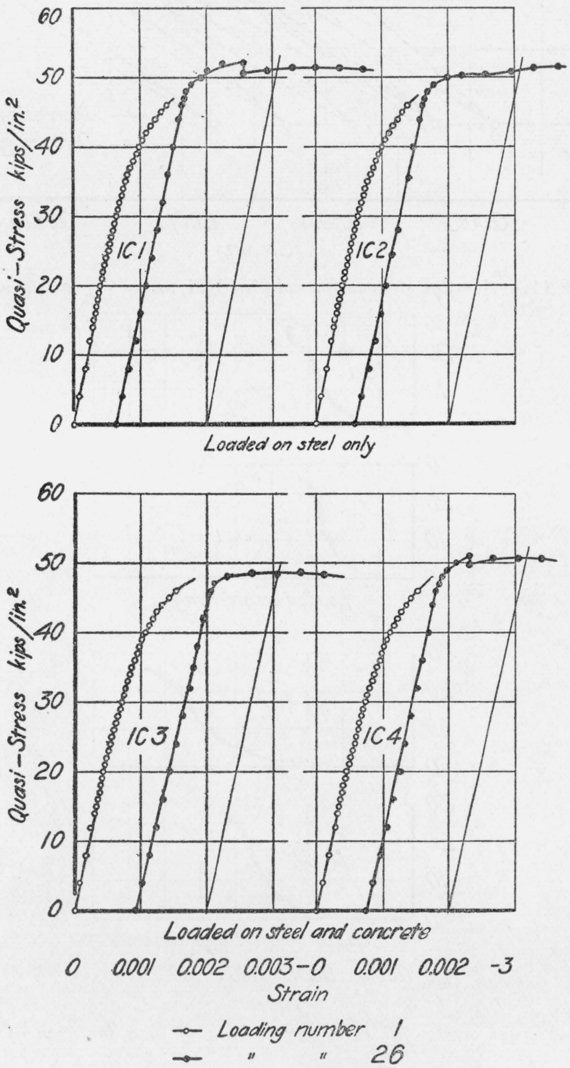

FIGURE 13.—*Stress-strain graphs for the first and twenty-sixth application of load for each of the columns.*

were considerable differences in the strains for the individual gage lines for a given load. Probably more consistent average strains would have been obtained if readings had been taken on more of the rods.

The stress in the rods increased with an increase in the quasi-stress in the column. Although the stress in the rods under the first application of load was less than one-fourth the proportional limit of the steel, the stress in the rods did not decrease to zero when the load on the column was decreased to zero.

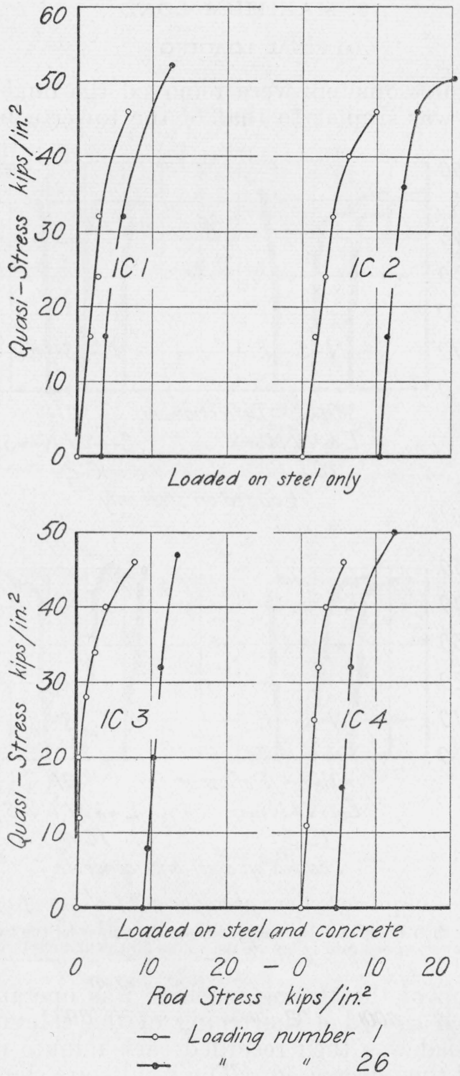

FIGURE 14.—*Average rod stresses at midheight for the first and twenty-sixth application of load for each of the columns.*

7. LATERAL DEFLECTION

The lateral deflections at midheight of each column are shown in figure 15. For quasi-stresses greater than 46 kips/in.² the deflections are the values for the twenty-sixth application of load. For lower quasi-stresses they are the values for the first application of load. The deflections were very small until the load on the column approached the maximum. For each of the columns except column IC2 there was an appreciable increase in the lateral deflection as the number of loadings increased.

8. MAXIMUM LOAD

(a) FINAL LOADING

After the compressometers were removed the final loading of the incased columns was similar to that of the tower columns TC1 and

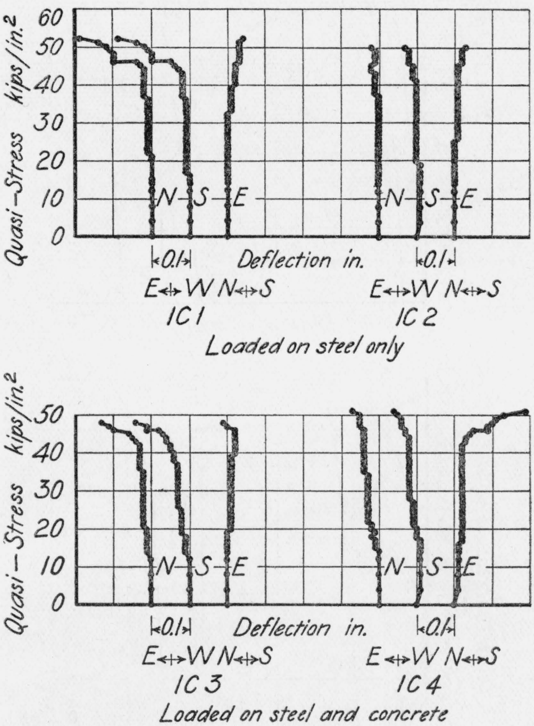

FIGURE 15.—*Lateral deflections at midheight for each of the columns.*

For quasi-stresses greater than 46 kips/in.² the deflections are the values for the twenty-sixth application of load. For lower quasi-stresses they are the values for the first application of load.

TC2. The pump of the testing machine was operated continously at a speed which caused a shortening of the column of about 0.1 in./min. The load was then recorded each minute until it reached a maximum and then decreased. The results are shown in figure 16. The maximum load on column IC1 was applied before the compressometers were removed. Pumping for more than an hour did not induce a greater load. For column IC4 pumping was stopped about 3 minutes after the maximum load was recorded. The lateral deflection at midheight was about 6 inches. Large pieces of concrete were falling from the column and it was considered inadvisable to carry the test further. The graphs for each column show several more or less pronounced maxima. Each of the columns showed either "pick-up" or "hang-on."[7]

When cracks suddenly appeared in the concrete or large pieces of concrete fell from the column there was an appreciable decrease in

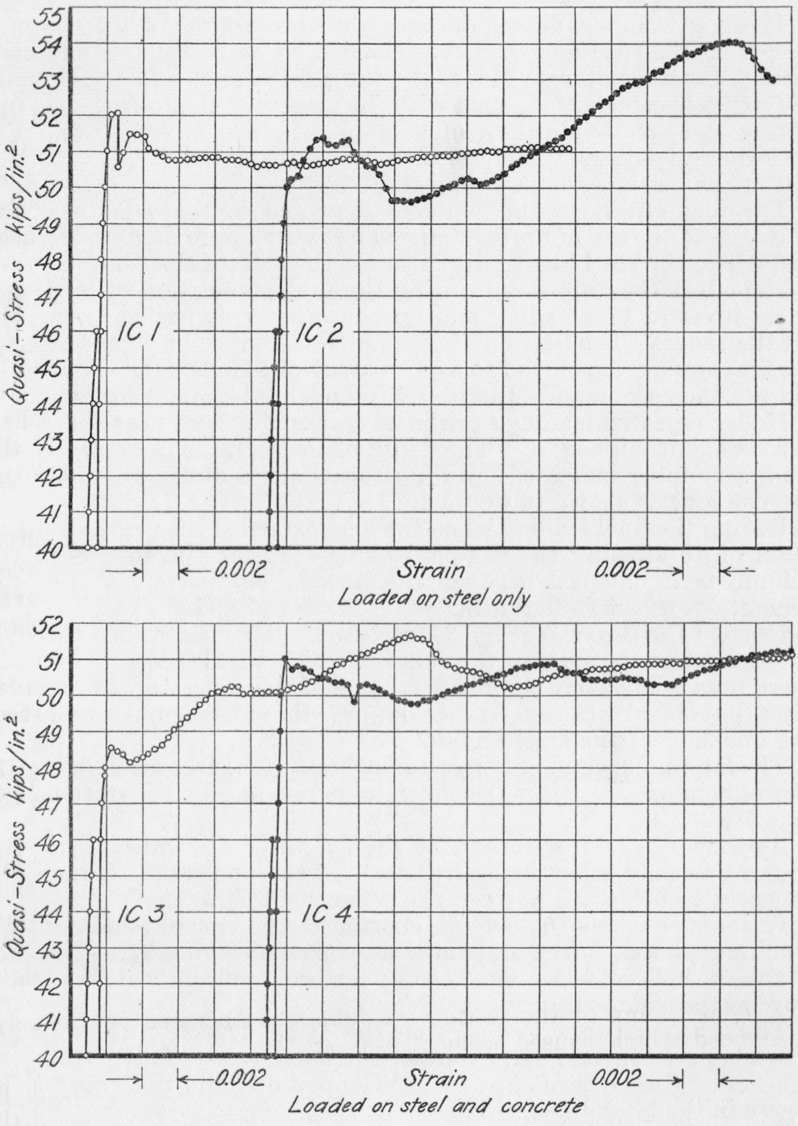

FIGURE 16.—*Final loading for each of the columns.*

the load. It is probable, therefore, that the properties of the concrete had a much greater effect upon the graphs shown in figure 16 than the properties of the steel members. The slenderness ratio of

[7] J. Research NBS **15**, 317 (1935) RP831; BS Tech. Pap. **21**, 59 (1926) T328.

the incased columns was less than that of the tower columns. The incased columns were therefore more likely to show pick-up or hang-on than the unincased columns.

(b) DESCRIPTION OF THE FAILURE

There was no significant difference in the failures of the columns loaded on the steel only and those loaded on both the steel and concrete. For the columns loaded on the steel only it is probable that the concrete came into contact with the platens of the testing machine during the first application of load (quasi-stress 46 kips/in ^{2}). For subsequent loadings, therefore, there was no essential difference in the way in which the load was applied.

The first cracks in the concrete appeared in the sides near the bottom and the top of the column. They were approximately vertical and where the steel angles were nearest the side of the column. For column IC4 they appeared during the first application of load at a quasi-stress of 17 kips/in.^{2}, and on the other columns at somewhat greater loads. The bottom of column IC1 is shown in figure 17 after the first application of load to a quasi-stress of 46 kips/in ^{2}. The cracks did not increase in width appreciably under subsequent loads.

Under repeated loading horizontal cracks appeared near the A and B rods about midheight. The width of these cracks increased as the loading number increased until horizontal strips of concrete fell from the column, as shown in figure 18.

During the final loading when the load exceeded the previous maximum of 46 kips/in.^{2} the C rods (see fig. 2) near the corners of the column bent outward between the A rods, causing spalling of the concrete as shown in figure 18.

None of the B rods (see fig. 2) fractured. They restrained the steel angles nearest the sides of the column, which buckled locally between these rods as shown in figure 19. Some of the concrete was removed after the column was tested so as to expose the steel members and show the buckling of the steel angles.

The lateral deflection after test of column IC2 is shown in figure 18, and of column IC4, after the concrete on one face had been removed, in figure 20.

As shown in figures 19 and 20 the length of a buckle in the steel angles was only a few inches, whereas in the unincased columns the length of a buckle was the distance between diaphragms.

In some cases, as the column shortened, the concrete between the diaphragms was forced outward, as shown in figure 21. The ends of the short D rods (see fig. 2) came into contact and were bent outward as the column shortened.

The tests were discontinued when the lateral deflection at midheight was about 6 in. and large pieces of concrete had fallen from the columns. Removal of the last load apparently did not cause a decrease in the lateral deflection.

For future tests of columns incased in concrete which are to be loaded on the steel only the steel members should extend beyond the concrete so far that they will not come into contact with the platens of the testing machine at the maximum load.

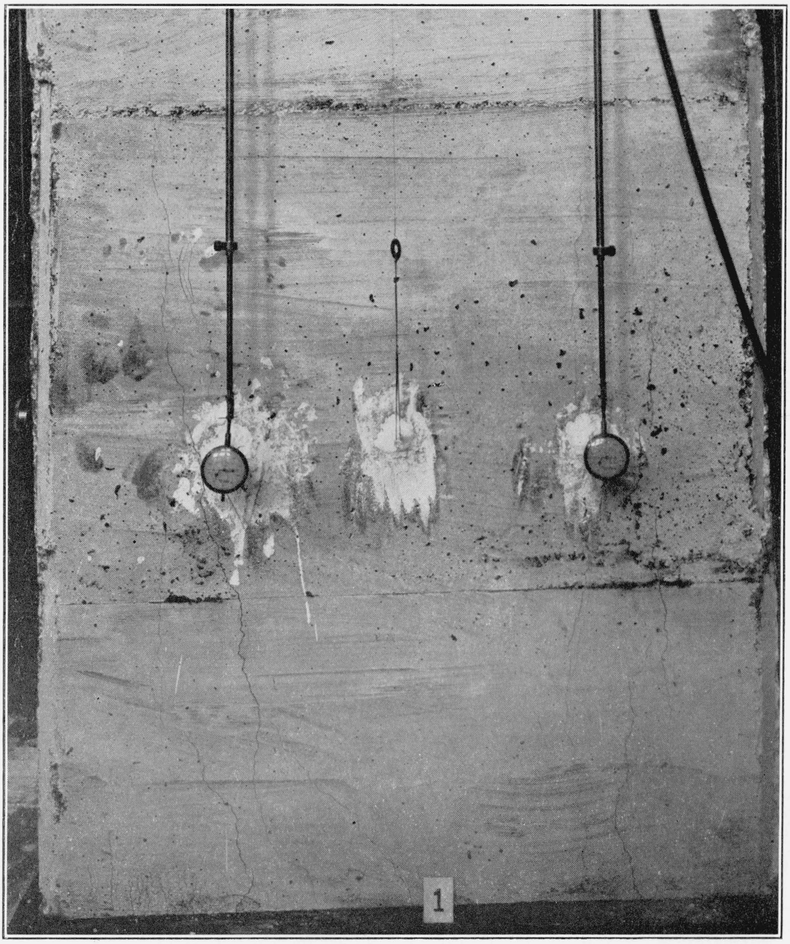

FIGURE 17.—*Bottom of column* IC1 *after the first application of load to a quasi-stress of* 46 kips/in^2.

The cracks are where the steel is nearest the side of the column. These cracks appeared on all four sides of the column, both near the bottom and near the top. They were the first cracks to appear.

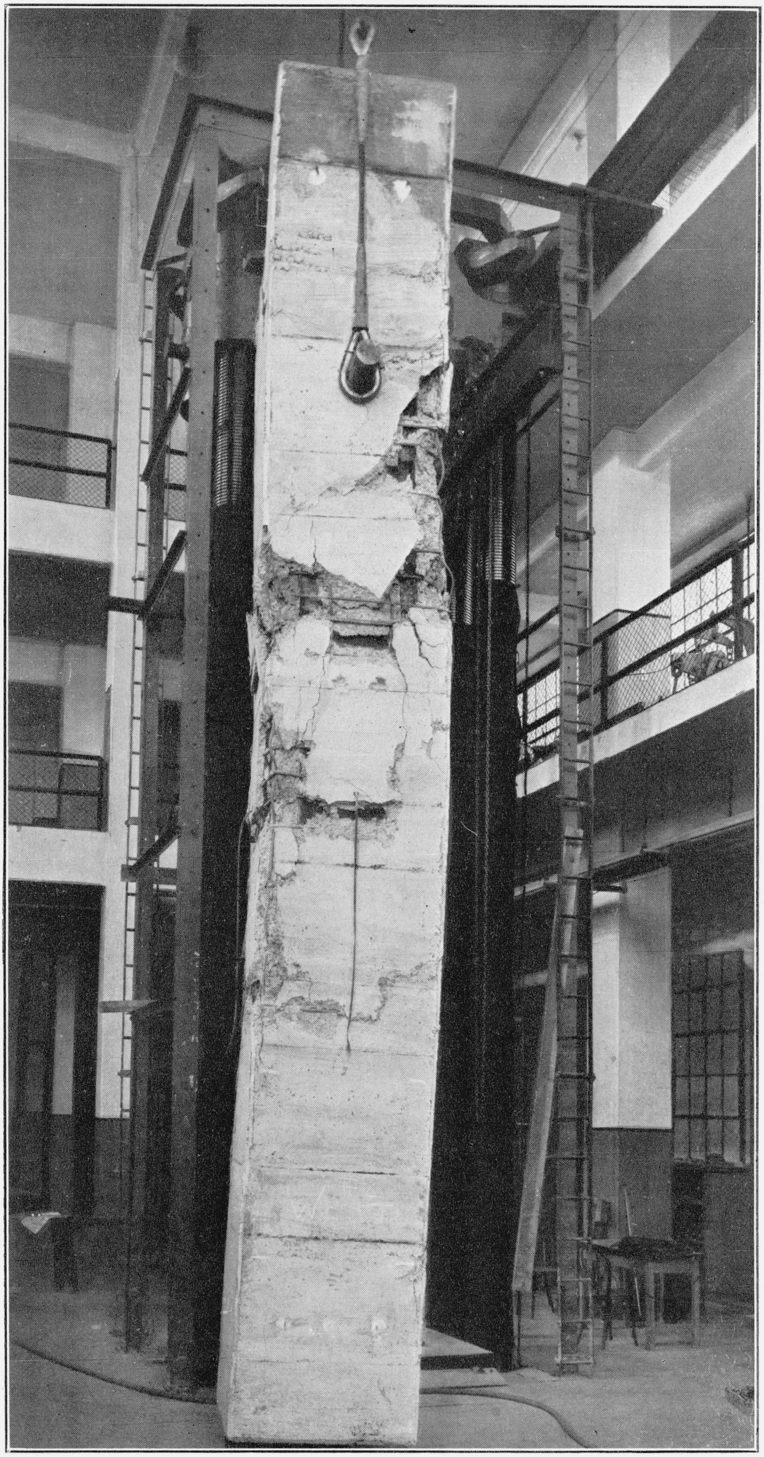

FIGURE 18.—*Column* IC2 *after test.*

About midheight horizontal strips of concrete have fallen from the column and the rods C near the corners of the column have bent outward causing spalling of the concrete.

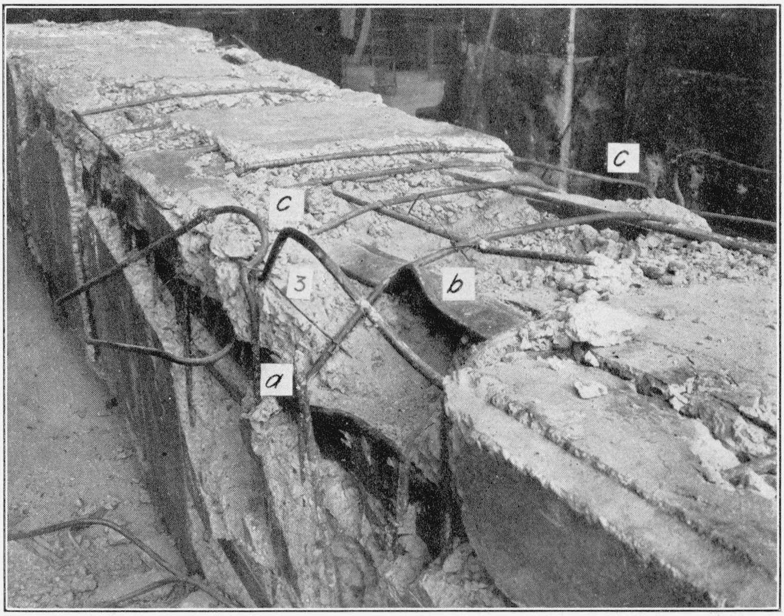

FIGURE 19.—*Column* IC3 *after test.*

Some of the concrete was removed to expose the steel members. (*a*) One A rod was fractured at the corner. (*b*) None of the B rods fractured. The steel angles nearest the sides of the column buckled locally between the B rods. (*c*) The C rod near the corner of the column bent outward between the A rods.

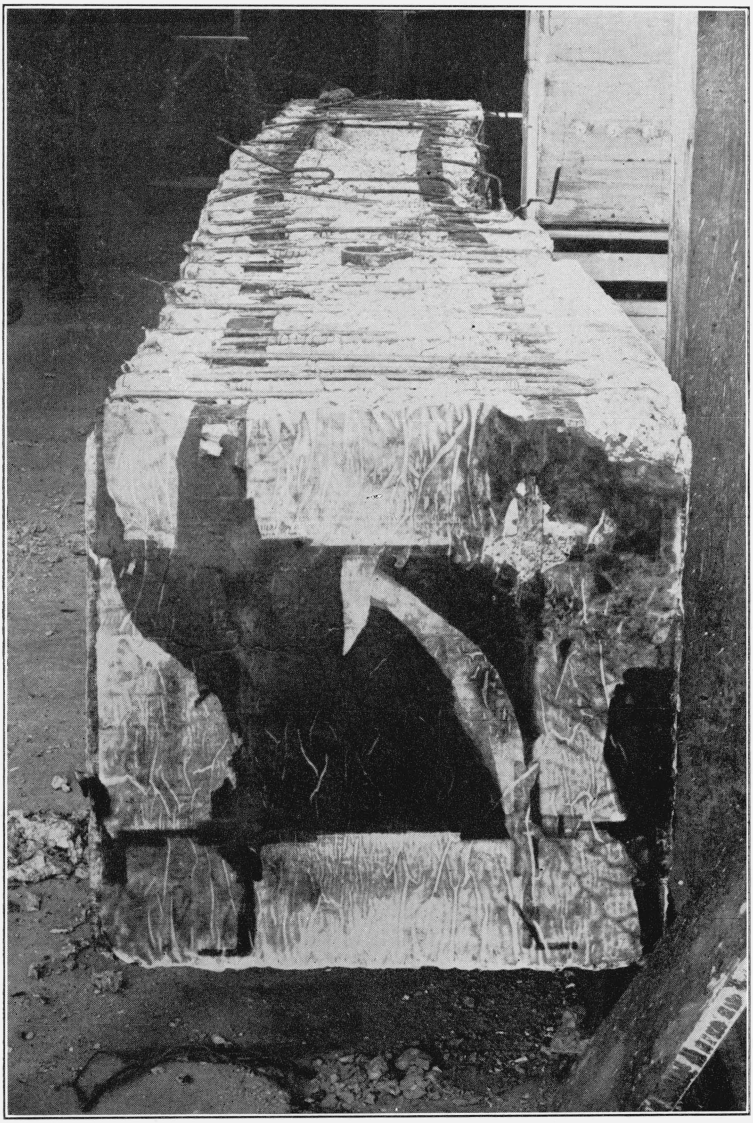

FIGURE 20.—*Incased column* IC4 *after test.*

The concrete on one side has been removed. The steel angles in the incased columns did not buckle between the diaphragms as they did in the unincased tower columns.

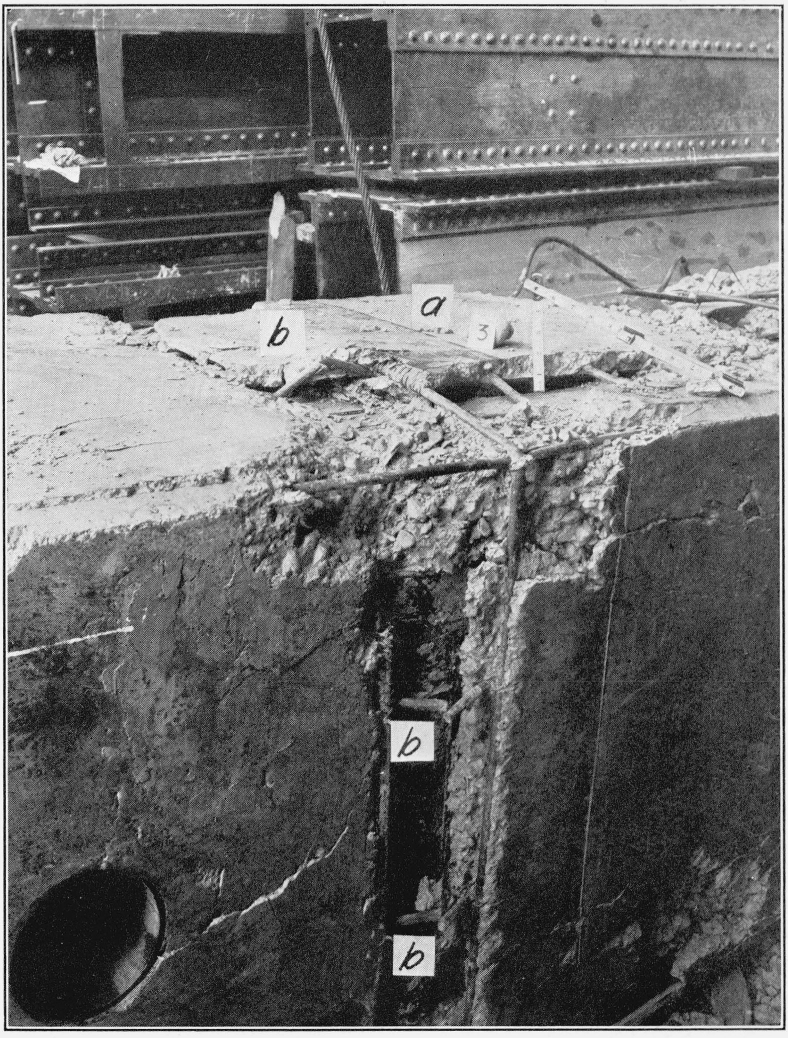

FIGURE 21.—*Column* IC3 *after test.*

As the column shortened (*a*) the concrete between the diaphragms was forced outward; (*b*) the ends of the short D rods came into contact and were bent outward.

(c) STRENGTH

The column strengths given in BS Technologic Paper T328 were the values of the first maximum stress. It was stated that "the practically definite first maximum stress, occurring before any appreciable lateral deflection of the column, and fairly reproducible when the column material and test conditions are reproduced, should furnish a good measure of the strength of the column in practical use. This justifies the practice followed in this report of recording the first maximum stress observed in a column test as the 'column strength' under the given test conditions. However, as was previously pointed out, this would not be justified in case no maximum were observed before the column was badly deformed."

With regard to the procedure that should be followed when no maximum is observed before the column is badly deformed, it was stated that "the best criterion could only be determined by a series of tests on columns in this range, in which the stress deformation curves were carefully determined."

In tensile tests of steel which do not show a definite yield point, it has become customary to define a yield strength in terms of the stress necessary to produce a definite strain (usually 0.002) in the coupon in excess of the computed elastic strain. It seemed probable that a similar definition of a column yield strength would be satisfactory for columns for which no definite first maximum load is observed, and for this reason the column yield strengths were computed on this basis.

The strengths of the incased columns are given in table 6. Although the column yield strengths and the maximum loads for the columns

TABLE 6.—*Strength of the columns*

Column	Loaded on	Cross-sectional area		Column yield strength	Final maximum load	Final maximum quasi-stress
		Steel	Concrete			
		in.^{2}	in.^{2}	kips/in.^{2}	kips	kips/in.^{2}
IC1	Steel only	159	1,245	51.2	8,278	52.0
IC2	do	159	1,245	51.0	8,587	54.0
IC3	Steel and concrete	159	1,245	48.6	8,211	51.6
IC4	do	159	1,245	50.7	8,179	51.4
Average				50.4	8,314	52.29

loaded on the steel only were somewhat greater than those for the columns loaded on both the steel and concrete, probably the difference is not significant because differences in the strength of the concrete may have had more effect on the strengths of the columns than the differences in the way in which the load was applied. The strengths of the four columns, therefore, were averaged. A comparison of the average strengths of the incased and of the unincased tower columns follows:

Average of the final maximum loads for the incased columns	8,314 kips.
Average of the final maximum loads for the unincased columns	5,853 kips.
Increase in the average of the final maximum loads due to the concrete	2,461 kips (42%).

Total area of concrete	1,245 in^2.
Strength of the concrete based on total area	1.98 kips/in^2.
Concrete area within A rods	1,190 in^2.
Strength of the concrete based on this area	2.07 kips/in^2.
Concrete area within B rods	1,140 in^2.
Strength of the concrete based on this area	2.16 kips/in^2.
Strength of concrete cylinders at age of 2 months	2.96 kips/in^2.

The strength of the concrete in the incased columns may be computed by dividing the increase in the strength of the incased columns due to the concrete by the total cross-sectional area of the concrete. The concrete outside the rods spalled before the maximum loads were applied. The strength of the concrete was, therefore, also computed by dividing by the area within the rods. These strengths are given above. The computed strength of the concrete in the incased columns is about two-thirds the strength of the concrete cylinders.

This difference in strength may have been due, at least in part, to the differences in the temperature and the humidity during aging.

V. CONCLUSIONS

1. STRESS IN THE STEEL MEMBERS

The stresses in the steel members were obtained from the readings of telemeters attached to the steel members and were designated "telemeter stresses." The telemeter stress was always less than the quasi-stress, indicating that the concrete carried a portion of the load. The quasi-stress was obtained by dividing the load on the column by the cross-sectional area of the steel members. As the load increased, the steel members carried a greater proportion of the load.

The telemeter stresses at midheight were less than those near the top and the bottom of the column for the columns loaded on the steel only. For quasi-stresses greater than about 35 kips/in^2. the telemeter stresses increased rapidly. The maximum stress in the concrete near the top was 2.62 kips/in^2. when the quasi-stress was 46 kips/in^2. At midheight it was 2.56 kips/in^2. when the quasi-stress was 46 kips/in^2., and near the bottom it was 1.40 kips/in^2. when the quasi-stress was 36 kips/in^2.

There was not much difference between the telemeter stresses for the columns loaded on steel only and those loaded on both the steel and concrete.

2. AVERAGE STRAIN

For an average strain of 0.0007 under the first loading the quasi-stress was 30 kips/in^2. for the incased columns, and 20 kips/in^2. for the unincased columns, indicating that for the same shortening (strain) the load was about 50 percent greater on the incased columns than on the unincased columns.

For each application of load on the incased columns the strain increased, but the increase was smaller and smaller as the loading number increased.

The strains in the incased columns loaded on the steel only were less than those loaded on both the steel and concrete.

3. STRESS IN REINFORCEMENT

The stress in the reinforcing tie rods increased with an increase in the quasi-stress in the column. The greatest stress was about 10 kips/in^2.

4. LATERAL DEFLECTION

The lateral deflection of the incased columns was very small until the load on the column approached the maximum. There was an appreciable increase as the number of loadings increased.

5. FAILURE

At or about the maximum load the incased columns having a length of 24 ft. deflected about 6 in., and large pieces of concrete fell from the column. Each column showed either pick-up or hang-on.

There was no significant difference in the failures of the columns loaded on the steel only and those loaded on both the steel and concrete.

6. STRENGTH

At the column yield strength the load on the incased columns was 51 percent greater than the load on the unincased columns. At the final maximum load the load on the incased columns was 42 percent greater than the load on the unincased columns.

The program and testing procedure were prepared by O. H. Ammann, chief engineer, L. S. Moisseiff, consulting engineer, and R. S. Johnston, research engineer, of the Port of New York Authority, and by L. J. Briggs, L. B. Tuckerman, and H. L. Whittemore, of the National Bureau of Standards. The following members of the staff of the Port of New York Authority assisted in making the tests and obtaining the data: A. H. Baker, F. J. Hinners, S. K. Hoppen, B. A. Lefeve, L. D. Mork, R. B. Morris, and G. A. Woods.

The chemical compositions of the steels were determined by the Chemistry Division of the Bureau.

The mix of the concrete was supervised by C. W. Ross of the Masonry Construction Section of the Bureau. He also placed the thermocouples for measuring the temperature in the columns.

WASHINGTON, November 1, 1935.