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ENGINEERING EXPERIMENT STATION  
Series 28

# REINFORCED BRICKWORK

by

MASON VAUGH



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ISSUED FOUR TIMES MONTHLY; ENTERED AS SECOND-CLASS MATTER AT THE  
POSTOFFICE AT COLUMBIA, MISSOURI—2500

OCTOBER 1, 1928



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### ACKNOWLEDGMENTS

I wish to gratefully acknowledge the help received from many different people in designing and carrying out this work. My thanks are especially due to Mr. L. B. Lent of the Common Brick Manufacturer's Association for his interest and for financial help in securing material; to Prof. A. Lincoln Hyde for his help in designing the experiments and in evaluating the results; to Prof. H. A. LaRue for guidance in testing the specimens and for the very cordial way in which he placed the entire facilities of the testing laboratory at my service; and also to Prof. J. C. Wooley for his interest and guidance and for sharing his research funds for the early parts of the experiments.

I wish also to acknowledge the help received from Prof. F. E. Richart of the University of Illinois; Mr. J. W. McBurney of the Bureau of Standards, Washington; the Portland Cement Association, Chicago; the Dickey Clay Products Co., Kansas City, and others who supplied information and material without which the preparation of this paper in its present form would have been impossible.



## PROPERTIES AND USE OF REINFORCED BRICK

### INTRODUCTION

My interest in reinforced brickwork was aroused through experience in India during six years occupation as Agricultural Engineer, during which period I carried out considerable building work. Brick offers several distinct advantages in comparison with practically any other material commonly used, and each of these other things have distinct limitations. The purpose of this paper is to point out these factors and to report the results of certain tests on various types of reinforced brick construction and the possibilities of utilizing some of these constructions in the ordinary types of building operations.

A short description of conditions in India may serve as a background for this report. The poorer classes of buildings are universally made of mud. In some cases this mud is molded into sun-dried brick much like adobe but usually only for small buildings. The mud, worked to a stiff consistency, is piled up to a height of two feet or more and allowed to dry sufficiently to carry the weight of another section which is then added in the same way and so on to a height of eight or nine feet. The walls are trimmed smooth and to a fairly uniform section and the roof put in place. Those who can afford a slightly better construction use burned brick usually set with a mud mortar, except around doors and windows and sometimes in the foundations where a natural cement or hydraulic lime is used. Even though the brick is of very good quality, the walls are usually excessively thick, 18 inches being not unusual for single story buildings and two to two and one-half feet for two story construction. Roofs in the drier sections are usually nothing more than four to six inches of mud on a support of rough boards, reeds in bundles, or the stalks of certain crops tied in bundles and supported by rough beams, or, frequently merely the crooked limbs of trees. Better construction utilizes sawed timber. In the sections having higher rainfall, thatch, which is unsanitary and dangerously liable to fire, or small tiles laid on bamboo mats, are used. These tile, when carefully placed, make a fairly tight roof but they are easily displaced and thousands of houses in India fall every rainy season because tiles become displaced and let water into the mud walls. While the first cost of these roofs is low, the recurring charges are very high. The annual upkeep may be fifteen to twenty per cent of the first cost. This of course is ruinous. As an alternative to the tile, masonry roofs of various types are used. Two general types are common: so-called "jack arch" or segmental brick arches on steel beams and brick, tiles or flat stones supported by wooden beams and joist or by steel beams and T-iron sections and covered with about four inches of concrete made from broken brick and a natural cement. If the terracing of concrete is sufficiently thick, made of good quality materials and well placed, this type of roof is satisfactory but is costly, and is very heavy. If it is on wood beams, the beams are certain to sag after a few years and cause cracks and leakage. The steel beams are better but very expensive and difficult to handle where facilities are poor. Spanish tile on steel trusses are also used, but are as expensive as the masonry roofs and in many ways less satisfactory.

Recognition of the above factors led to a search for something better. Shingles are expensive because suitable wood is scarce and sawing facilities are limited. Any type of asbestos shingle or asbestos cement sheet or other manufactured covering is about as expensive as the masonry roofs when the necessary wood support is taken into consideration. In addition, none of these are suitable to the climate unless a ceiling is added beneath for appearance and for protection from the heat. This further adds to the cost. A flat roof is also desirable because it is used a large part of

the year as an outdoor sleeping place. Reinforced concrete offers the advantages of a flat roof, fairly light in comparison with segmental arches and thick enough to protect from the heat of the sun but has the disadvantages of requiring careful form work and a degree of skill in locating and in handling concrete rarely found available. Portland cement requires prompt handling in placing while the natural cement in common use is better if soaked up and worked over for two or three days before placing. Perhaps the greatest difficulty in using concrete is the almost total lack of form lumber—that is, lumber sawed to accurate sizes and mill planed—at a price that is not prohibitive. Hand sawed lumber, unsurfaced, rarely costs less than \$150 per thousand feet and is often hard to get at that price. Suitable stone is not available locally in most sections of north India. These factors practically exclude concrete except in certain specially favored localities. Some concrete is made, using broken brick as coarse aggregate but it is expensive and requires a degree of supervision not often available, so it is rarely used. However, its many advantages led engineers to search for a substitute that would have the advantages of reinforced concrete without its limitations and near the end of the world war definite experiments were taken in hand with reinforced brick. This proved so satisfactory in use that it has been widely adopted, and is now a standard method of construction over large areas of North India.

About April, 1924, I roofed a small building of five or six rooms with a five-inch slab. I had not at that time been able to secure any design data on the process other than to follow ordinary reinforced concrete practice. The trial was successful and the roof is still in good condition. When I later secured the design data available, it seemed to me to be lacking in some details and to depend on one quite comprehensive set of experiments. Apparently the results have not been checked by experiments either in India or elsewhere. It therefore seemed desirable to check these results and to try to fill in the gaps in our knowledge.

## CHAPTER I

## WORK PREVIOUSLY DONE ON REINFORCED BRICKWORK

A survey of the available literature shows that the use of reinforced brickwork has been scarcely touched outside of India. Some work, mainly on the reinforcing of walls against lateral deflections in bins, etc., and some use of reinforced construction where it was desirable to suspend a wall between end supports has apparently been done in England and reference to one such test in Canada was found in the Canadian Engineer for February 19, 1914. Apparently other tests were made but only one is reported with data and no record of further use or test has come to light. The only other test of this sort revealed by extensive reading, correspondence with the several trade associations likely to be interested, the Bureau of Standards and the leading engineering institutions in the country, as well as much personal discussion with engineers, was a series of tests of reinforced lintels made by the Case School of Applied Science under the direction of Prof. Danforth. He tested twenty-two beams with the results shown by the following data furnished by the Common Brick Manufacturers' Association.

## REPORT

May 5, 1924

The Common Brick Manufacturers' Assn.,  
Cleveland Discount Building,  
Cleveland, Ohio.

Gentlemen:

We submit herewith a set of tables and charts which constitute our report on a series of bending tests on reinforced brick lintels upon which we made a preliminary report on January 30, which report contained dimensioned diagrams of the various specimens.

It would appear, from a study of the results of these tests, that if, in the field, workmanship can be obtained equal to that used on the test specimens, a four foot lintel sixty days old will safely carry over ten times the weight of the superimposed wall, eight inches thick, which will normally come upon it; that a six foot lintel sixty days old will, without danger, carry at least three times the probable superimposed wall load; and that an eight foot lintel will carry more than one and one-half times the weight of the superimposed eight inch wall. These figures are based on the usual assumption that the lintel must carry a wedge of the wall above it, of height equal to half the width of the opening.

It also appears, although the small number of tests does not give full assurance, that a six foot lintel thirty days old will safely carry about one and one-half times the probable superimposed wall load.

Tests Number 3 on four foot span, Number 7 on six foot span, and Number 18 on eight foot span, show what is possible with this method of construction, with thoroughly first-class workmanship. These specimens had the steel well placed, the bottom of the rod being about three-quarters of an inch above the bottom of the brick, and the steel being completely covered on all sides with mortar. The adhesion of mortar to brick in these specimens was good, showing that the brick had been thoroughly wet before laying, which seems to be of nearly if not quite equal importance with the adhesion of the mortar to the reinforcing rod.

The specimens tested at age thirty days were laid up at a later date than the sixty day specimens, and the lack of adhesion of mortar to brick seems to indicate a lack of care in wetting the brick, resulting in a drying out of the mortar rather than a true setting.

It seems to us that if, either by careful education of the workmen or by thorough inspection during construction, a few simple but absolutely essential details can be thoroughly carried out, this new form of lintel construction can be used without the slightest doubt or hesitation on the part of the architect, the builder or the owner. These details are:

1. Thorough wetting of the brick
2. Thorough covering of steel with mortar
3. Placing of the steel as low in the joint as will insure complete covering with mortar on the lower edge.

It appears to us advisable to determine by means of special tests the normal bond strength of cement lime mortar on steel bars, as one of the essential factors in computing beams of this sort. We would recommend that before making any further tests on complete lintels a series of tests of this nature be undertaken.

Trusting this report contains all the information you require, I remain

Yours very truly,

(Signed) R. H. Danforth

Professor in Charge

Materials Testing Laboratory

SUMMARY OF RESULTS OF TESTS ON BRICK LINTELS  
FOR THE  
COMMON BRICK MANUFACTURERS' ASSOCIATION  
CLEVELAND, OHIO

Beam No.	Space ft.	Steel sq. in.	Yield Point Load, lbs.	Maximum Load, lbs.	Stresses at Yield Point lbs. per sq. in.		
					Steel	Brick	Bond
1	4	.3125	5100	7400	14,900	405	115
2	4	.3125	3500	9890	10,450	275	88
3	4	.3125	8100	11,490	22,500	570	195
4	6	.3125	5150	5,910	21,400	524	124
5	6	.625	4800	6,550	10,950	430	69
6	6	.625	5200	7,220	12,780	526	81
7	6	.3125	5600	10,000	24,100	610	130
8	6	.625	3300	5,150	8,800	388	56
9	6	.3125	3900	5,440	15,400	410	90
10	6	.3125	4000	5,380	17,600	456	103
11	6	.3125	4650	7,240	18,400	438	107
12	6	.3125	2650	5,220	9,570	224	62
13	6	.3125	5600	6,830	24,100	613	140
14	6	.3124	3700	6,570	15,000	354	87
15	6	.3125	4000	7,700	17,480	427	101
16	6	.3125	6600	9,120	29,100	736	169
17	8	.3125	3000	6,420	16,700	413	72
18	8	.3125	2600	6,100	14,950	378	65
19	8	.3125	3200	5,090	18,050	477	79
20	6	.3125	(a)	1,240	-----	---	-----
21	6	.3125	1420	1,600	6,280	159	36.3
22	(b)	.3125	1400	3,400	5,240	134	36.3

SUMMARY OF RESULTS OF TESTS ON BRICK LINTELS  
FOR THE  
COMMON BRICK MANUFACTURERS' ASSOCIATION  
CLEVELAND, OHIO

Beam No.	Stresses at Maximum Load lbs. per sq. in.			Remarks
	Steel	Brick	Bond	
1	21,600	586	167	Brick crushed
2	29,500	776	249	Steel slipped
3	31,900	808	275	Steel slipped, on bottom at one end
4	24,500	600	143	Steel slipped
5	14,950	587	94	Steel slipped, on bottom both ends
6	17,750	732	111	Steel slipped
7	43,000	1,090	231	Tension in steel, yield point of steel about 41,000 # sq. in.
8	13,720	605	87	Brick crushed, joints poorly filled
9	21,500	572	126	Compression of mortar, joints poorly filled, steel on bottom both ends
10	23,700	614	138	Steel slipped, on bottom one end
11	28,650	684	166	Brick crushed, steel on bottom both ends
12	18,850	442	123	Steel slipped, mortar did not cover steel
13	29,400	748	171	Brick crushed
14	26,700	630	154	Steel slipped, on bottom one end
15	33,600	820	194	Steel slipped
16	40,200	1,020	233	Brick crushed
17	35,800	885	155	Steel slipped
18	35,100	687	153	Brick crushed
19	28,700	760	125	Steel slipped
20	6,480	180	37.5	Mortar failed in shear
21	7,080	179	41.0	Mortar failed in shear
22	12,750	326	88.2	Mortar failed in shear and steel slipped

SUMMARY OF RESULTS OF TESTS ON BRICK LINTELS  
for the  
Common Brick Manufacturers' Association  
Cleveland, Ohio

(a) Only two deflections were taken on beam No. 20; the failure occurring almost immediately upon application of the load. As these two points would not give a satisfactory curve, no yield point could be determined.

(b) Due to the falling off of a course of brick from the end of beam No. 22, it was tested on a somewhat shorter span than 6 ft.; deflections, however, have been reduced to a 6 ft. basis in order to compare with lintels previously tested.

In computing the above stresses the modulus of elasticity, (E) for brick masonry was assumed as 1,300,000 lb. per sq. in., which is approximately the average of the results obtained in the Columbia University tests. This value for E gives a value of 23 for  $n$ .

All lintels were tested at the age of 60 days, with the exception of Nos. 20, 21, and 22, which were tested at the age of 30 days.

Tested at Materials Testing Laboratory,  
Case School of Applied Science, Cleveland, Ohio.  
By: R. H. Danforth and H. C. Plummer

January 8, 1924

Common Brick Manufacturers' Association,  
Cleveland Discount Building,  
Cleveland, Ohio.

Gentlemen:

The following three tables give the results of the tests made on the common brick which you submitted to us and from which the brick beams which we tested during the holiday period were made. These tests were made in accordance with the specifications of the American Society for Testing Materials.

ABSORPTION TEST

Mark	Dry Weight grams	Weight after 5 hours boiling	Gain in Weight grams	Per Cent of Absorption
1	2116	2688	572	27.0
2	2119	2620	501	23.7
3	2118	2627	509	24.1
4	2132	2678	546	25.6
5	2169	2658	489	22.5
			Average.....	24.58

CROSS BENDING TEST

Span, 7 inches

Mark	Width in inches	Thickness in inches	Center Load in pounds	Modulus of Rupture in lbs. per square inch
1	3.89	2.35	640	313
2	3.94	2.35	685	334
3	3.96	2.31	390	194
4	3.91	2.35	630	307
5	3.96	2.35	530	255
			Average.....	281

COMPRESSION TEST

Mark	Area in square inches	Total Load in pounds	Ultimate Strength in lbs. per square inch
1	9.96	33,590	3,370
2	9.20	18,620	2,020
3	8.78	12,720	1,450
4	9.80	15,670	1,600
5	8.93	15,060	1,690
		Average.....	2,026

Measured by the specifications of the American Society for Testing Materials, a copy of which is enclosed, this specimen of brick would be classed as soft.

Signed—R. H. Danforth  
Professor in Charge,  
Materials Testing Laboratory.

Tested by H. C. Plummer and S. B. Folk,  
Materials Testing Laboratory,  
Case School of Applied Science,  
Cleveland, Ohio.

Since the foregoing seems to be the only series of tests made under laboratory control in this country, the data is given practically in full, with Prof. Danforth's report. The mortar was common cement-lime-sand (1-1-6) and the steel was round rods and flats.

One use of reinforced brickwork that has been common in this country for some time, and is now well accepted, is the circular silo and storage bins reinforced with bands of wire, round rods or flat strips laid in the mortar joints. These have been built of brick, clay blocks of various types, concrete blocks and other materials and it is surprising that no one has gone from this application to beam and slab construction.

On the ordinary use of brick there has been much work done. The Building Code Committee of the United States Department of Commerce has compiled the data on 708 tests made previous to March 1926 on brick piers, walls and columns under varying methods of construction and with various mortars. An analysis of these tests shows that of all individual brick tested, over 97 per cent showed a compressive strength above 2,500 lbs. per sq. in. In nineteen separate investigations comprising 367 individual tests, the average ultimate compressive strength of all specimens (consisting of specimens laid with cement and with cement-lime mortars in various strengths) was 1,804 lbs. per sq. in. For 292 tests where specimens represent some degree of commercial workmanship, the average ultimate compressive strength was 1923 lbs. per sq. in. It should be noted that this is 96 per cent of the customary 2,000 lbs. per sq. in. expected of commercial 1-2-4 concrete.

Since 1926 the U. S. Bureau of Standards at Washington has carried out a long series of tests on brick walls laid in different ways, using different mortars and with varying qualities of workmanship. These tests were unusual in that they were made on structures nine feet high and six feet wide, giving results really comparable to actual construction. The results of these tests will be discussed in some detail later. There have also been other investigations, notably a series of tests at Columbia University, New York, reported in their Engineering Bulletin No. 12.

The only work in print on the subject of reinforced brickwork which treats the subject at all exhaustively is Technical Paper No. 38 of the Public Works Department, Government of India, "Notes on Reinforced Brickwork", by A. Brebner, C. I. E. printed at the Government Press, Calcutta, India, 1923. It is in two volumes, Volume I including Notes and Volume II, illustrations, Data Tables, Comparative Tables, Plates, Curve Tables and Plans of certain Buildings. Since my investigations were largely based on principles discussed in this Technical Paper, and since it is not readily available in this country for reference, a very complete abstract of Volume I follows: The quotation is Mr. Brebner's description of various forms of reinforced brickwork.

"The similarity between reinforced brick and reinforced concrete structures has already been referred to. The principles of reinforcement are identical in both, the aim of the designer being to place the reinforcement in such a position that it will

take up certain stresses; for this purpose in reinforced brickwork rods are well embedded in the mortar joints of the masonry in suitable positions. Experiments have demonstrated that the steel and the masonry surrounding it act as one compact mass in almost exactly the same way as the concrete and reinforcement in reinforced concrete work.

"At first sight it would seem that brickwork could not be a homogeneous mass in the sense that concrete is, and that the regular joints in the work would present planes of weakness along which failure would readily take place. In practice, however it has been found that this is not so. On the contrary it has been proved that this factor is so insignificant that it can be neglected. It has also been established that there is no reason why reinforced brick structures should not be as successful as reinforced concrete ones of a similar nature, provided ordinary precautions are taken in designing and carrying out the work."

In further discussion it is pointed out that the main principles of steel reinforcement in concrete are well established, and that this is a form of construction long since past the experimental stage. But reinforced concrete has not been common in India due to the comparatively high price of cement. On the other hand the prices of brick, tile and other clay products are low so that this material assures a cheaper form of construction of high quality, as well as one more easily and cheaply super-vised.

Neither Indian masons nor laborers can be trusted to do good concrete work without the most careful supervision to secure the construction of centering and correct placing of reinforcement. In reinforced brickwork construction these difficulties largely disappear. It will be found that the cost of this type of construction is lower than any other form of more or less permanent nature. Reinforced brickwork has commonly been used in lintel construction and in partition walls but only recently in floors, roofs, staircases, etc.

The system was first introduced in the construction of the New Capitol for Bihar and Orissa at Patna, and proved economical and successful so that the method has been held to be suitable for work at New Delhi and elsewhere. A number of instances are given where this type of construction has been used and the items enumerated are: simplicity of construction; good permanent work with low repair charges; fire proof; neat and artistic appearance; low cost.

The type of construction is such that it may be done by ordinary Indian workmen using only bricks, cement, sand and ordinary mild steel rods. The cost of centering is low since it may be of rough character and may be used repeatedly. Even the Indian bricklayer can do good brick work, though he cannot do good concrete work. Therefore less supervision is required. The reinforcement is inserted as the work proceeds and is not liable to displacement as it might be in concrete.

In the practical execution of reinforced brickwork the main requisites are a proper centering, good materials and careful work.

The simplest type of centering and that most generally used consists of a platform of planking at the required level, supported on beams and covered with a thin layer of well beaten earth finished off with fine sand. This process will assure a centering having the following properties: Rigidity, simplicity of construction, ease of slackening and removal, and a smooth surface on which to lay the slab.

For materials, only the best bricks, complying with the usual first-class specifications, should be used in reinforced brickwork. Hardness is a desirable quality but brittleness or a smooth glaze on the surface are both undesirable. Sand should be clean and well graded and preferably sharp, although this point is not essential. In every particular the sand should correspond to a specification that might be used for a

first-class concrete. Mild steel should be used for reinforcement and only circular or square sections should be used. Flats and angles should be avoided.

In summing up the results of his experiments the author claims that the following conclusions are justified:

1. Reinforced brick slabs may be designed according to reinforced concrete theory. In the barrack type of building commonly met with in India, the limiting stresses may be taken as high as 20,000 pounds per square inch for steel in tension and 350 pounds per square inch for brick in compression.

2. Patent stone may be considered as having a strengthening effect if done along with or soon after the reinforced brickwork.

3. In cantilevers the stresses in steel should not exceed 16,000 pounds per square inch.

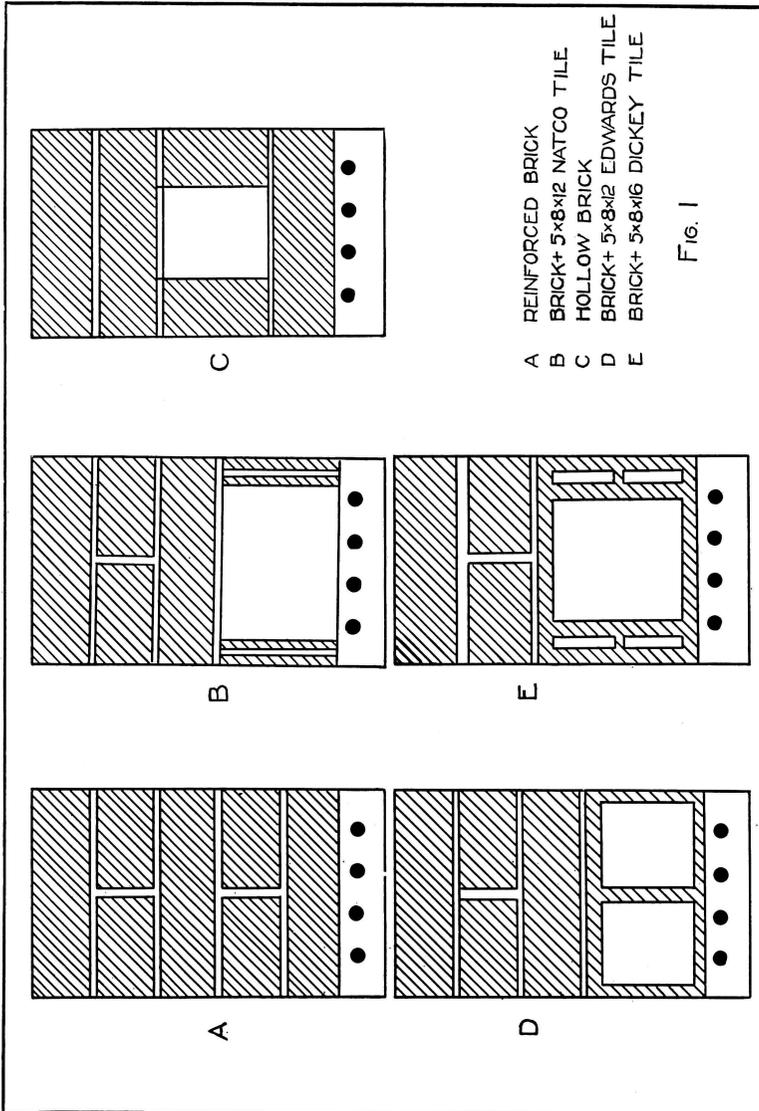
4. Reinforced brick beams may be designed according to reinforced concrete theory. The limiting stresses should be 16,000 pounds per square inch for steel in tension, 250 pounds per square inch for brickwork in compression, 80 to 90 pounds per square inch for adhesion between steel and mortar, and 60 pounds per square inch for shear in brickwork.

5. It is best to combine reinforced concrete with reinforced brick in the construction of beams. The modulus of elasticity of brick appears to be about one fortieth of the modulus of elasticity for steel, although a variation of as much as 50 per cent in the value of this ratio does not alter the stresses materially.

6. Temperature stresses may be neglected in all ordinary structures.

CHAPTER II  
PRELIMINARY PLAN OF FIRST SERIES OF TESTS

The first series of five beams were tentative and were designed to give information on two phases of the subject. In the first phase it was desired to get all the information possible as to the behavior of such beams as a guide to future design; to get some definite idea of the loads the beam could be expected to carry, the type of failure to be provided for, the magnitude of the deflections to be expected and, if possible, information on the modulus of elasticity of brickwork. Fig. I shows the



cross section of the five designs built. The beams were uniformly about thirteen feet long and were tested on twelve foot centers with the loads applied at third points by an Olsen beam testing machine of 50,000 pounds capacity.

The second phase of the problem dealt with decreasing the weight of such beams by making them hollow in the region of the neutral axis. In the smaller sizes, T-beams do not fully meet the need since the web section is insufficient to support the required steel reinforcement. Therefore, the four hollow types were constructed by making one all-brick and three other types using various styles of tile as shown.

Longitudinal steel used consisted of four, one-half inch round steel rods. In all but the solid beam they were merely laid straight without anchoring hooks. No difficulty was experienced in testing from the steel slipping which indicates that the bond was sufficient. Assuming the working stresses in the steel to be 16,000 lbs. and in the concrete or brick, 650 lbs. per sq. in., the steel required for this size of beam in ordinary concrete practice is 0.77 sq. in. Four, one-half inch rods gives 0.79 sq. in. or a slight excess for a depth of  $d = 12$  in. No vertical stirrups were used as concrete practice does not call for stirrups in this size of beam. Steel was placed in a bed of cement-sand-broken stone (1-2-4) concrete for convenience, on which brick or tile was laid after bedding with mortar.

On the basis of the information available, a cement sand (1-3) mortar was used with hydrated lime equal in weight to ten per cent of the cement added for workability. A local sand with a fineness modulus of 1.6 was used. This sand was too fine to make the best mortar. All joints were filled and brick were laid on a flat mortar bed. The brick were thoroughly wet at all times and the mortar was made soft to facilitate working.

The brick used were made locally from an excellent grade of shale, well burned and hard. They showed a compressive strength (average of 20 tests) of 5,953 lbs. per sq. in., and a modulus of rupture of 1,698 lbs. The absorption was not determined but it was quite low as the brick were burned to incipient vitrification.

Fig. 2 shows the testing machine used with a beam in place. It was originally planned to use an Olsen beam apparatus but this was found to be impossible with the equipment available when the tests on the first series were made. Therefore, only loads and deflections were taken. The load was applied at one-third points as shown using the slowest speed of the machine. Fifteen to twenty minutes were required to apply the full load including the time the machine was stopped for readings.

Fig. 3 shows the failure in the solid brick beam. It is a typical diagonal tension failure showing a greater tendency for the parts of the beam to separate vertically than for slip to take place horizontally. The tendency to push the end off should be noted. Later determinations show the load of 13,000 lbs. to be near that required to stress the steel to its elastic limit. The following table No. 1, gives the data as taken during the test.

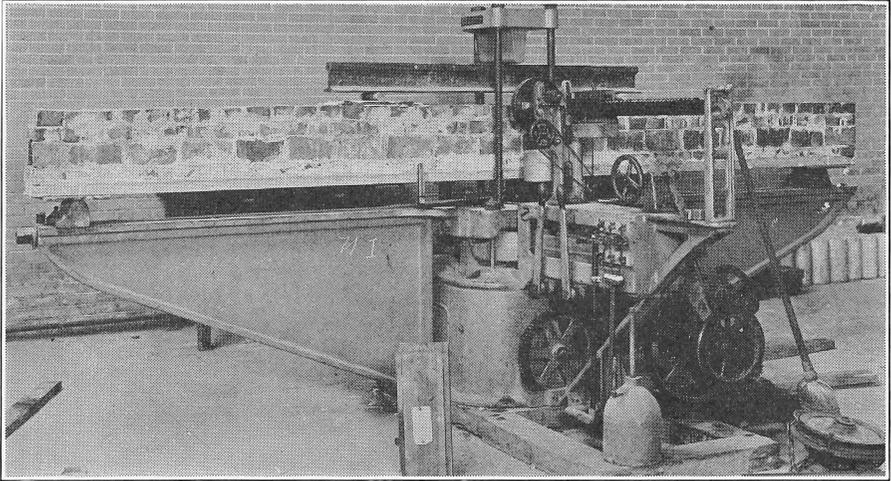


Fig. 2

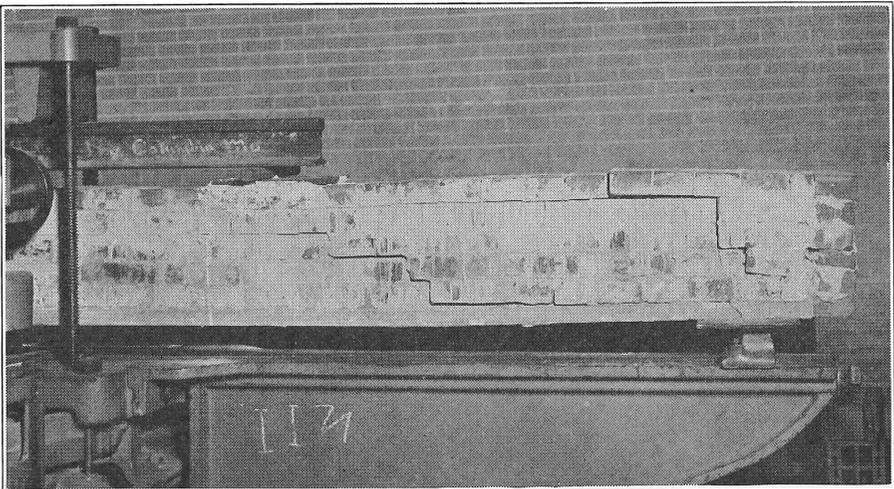


Fig. 3

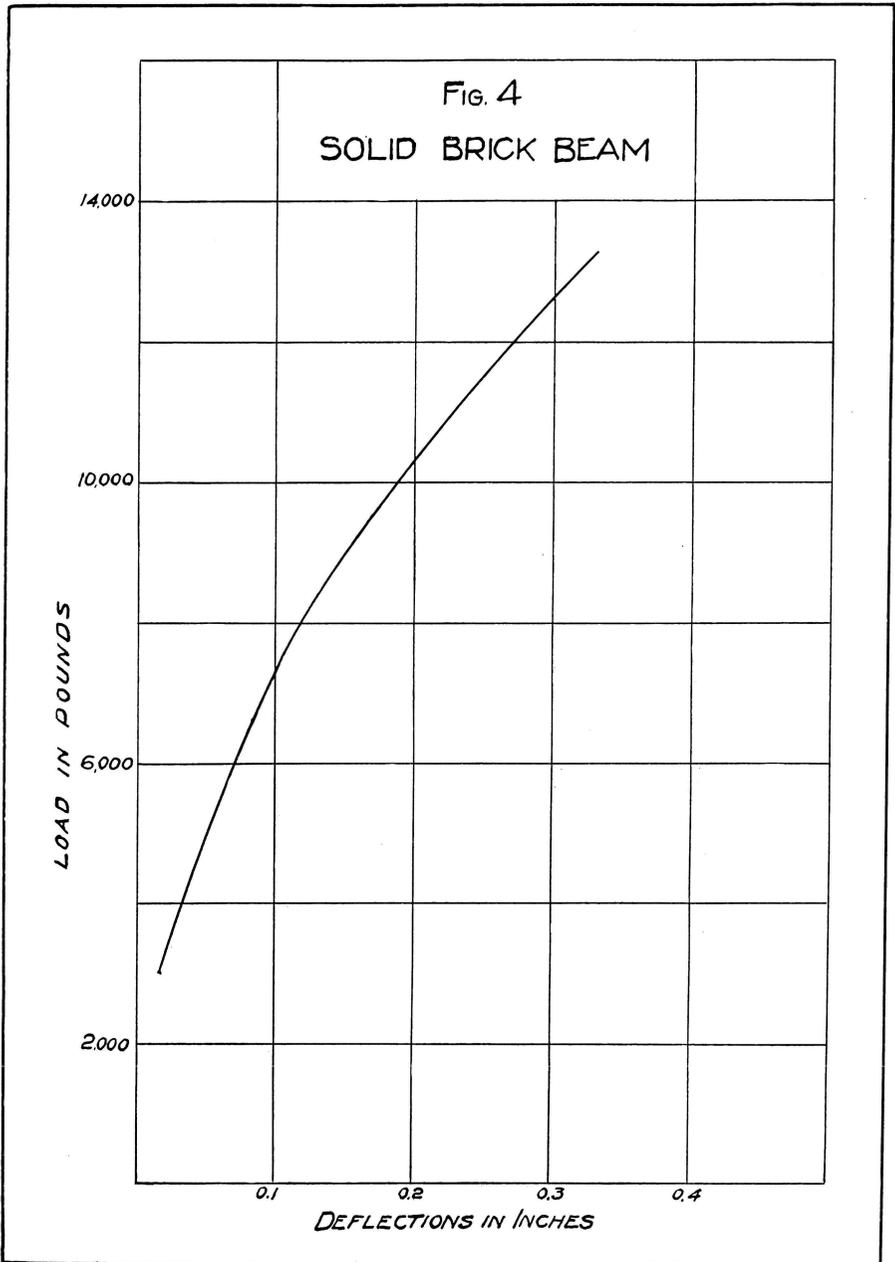


TABLE NO. 1  
ALL BRICK BEAM

Load pounds	Deflection, inches		
	East	West	Average
1,750	.001	.000	.0005
3,000	.028	.013	.020
4,000	.042	.035	.038
6,000	.078	.068	.073
8,000	.140	.127	.133
9,550	.187	.171	.179
10,000	.206	.191	.198
11,000	.240	.224	.232
12,000	.273	.263	.268
13,000	.313	.304	.308

In order to compute the modulus of elasticity of such a beam, calculations were made from the homogenous beam deflection formula

$$D_{max} = \frac{W a}{12 E I} (3/4 l^2 - a^2)$$

when W is the total load applied in two equal parts at third points on beam, l is the total span, and a is the distance from support to point of application of the load. For a load of 6,000 lbs. this gives a value for E of 2,000,400 lbs. per sq. in. and for a load of 12,000 lbs. a value of 1,300,000 lbs. per sq. in. To meet objections which have been made against the validity of such computations under the assumption that measured deflections were not reliable, the observations on twenty-four beams of varying design showed a regular deflection curve and the values determined in this way closely check other figures. Fig. 4 shows deflections plotted against loads.

Beam No. 2 was made with Natco end-construction hollow tile in the lower part of the beam. Fig. 5 shows quite clearly construction of the beam and the type of

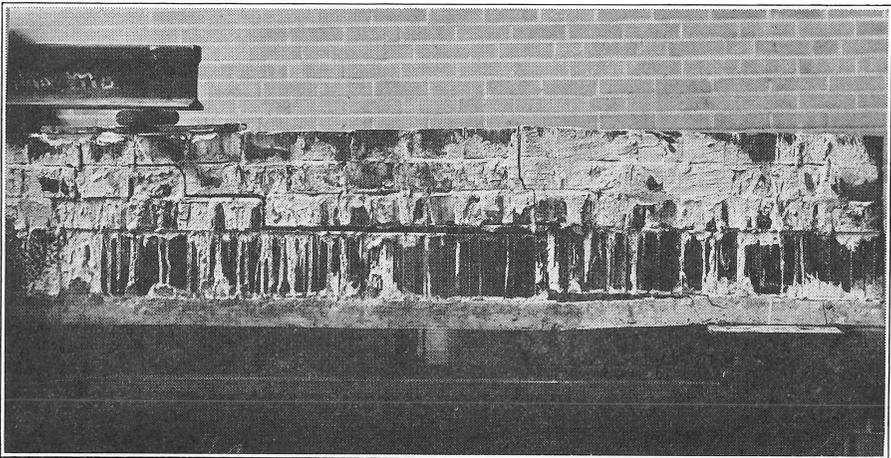


Fig. 5

failure. Unfortunately, the dead weight of the beam was not recorded, but table No. 2 shows the data secured and Fig. 6 shows the deflections plotted against the loads.

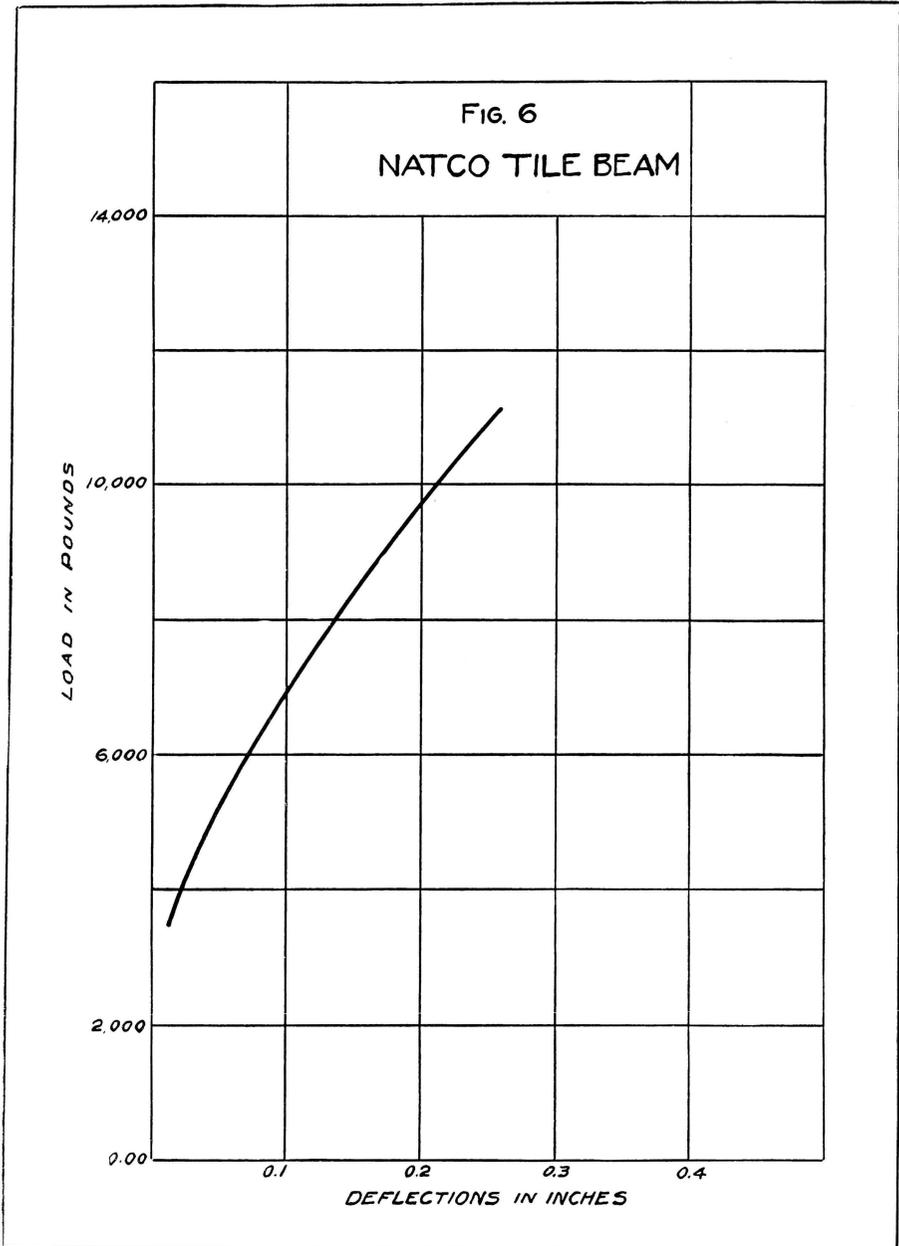


TABLE No. 2  
NATCO TILE BEAM

Load pounds	Deflection, inches		
	East	West	Average
3,640	.002	.001	.001
5,000	.050	.039	.044
6,000	.078	.067	.072
7,000	.110	.097	.103
8,000	.144	.133	.138
9,000	.176	.166	.171
10,000	.216	.208	.212
11,130	.267	.264	.266

Beam No. 3 was of hollow construction but utilized only brick in its construction. A course of headers was laid on the concrete and parallel rows of brick on edge were set flush with the sides of the beam and covered with two courses of headers. This gave a hollow of about four inches square in the center of the beam which resulted in a considerable reduction in the dead weight of the beam but also in a corresponding reduction in the load carried so there was no real gain. Fig. 7 and 8 show the types of failure on opposite sides of the beam and Table 3 gives the data secured. Figure 9 shows the deflection plotted against loads.

Beam No. 4 was made with a two cell tile in the lower half. This tile was good in that it laid up quickly but the adhesion to the mortar seemed poor and failure was sudden as shown in Fig. 10. Table No. 4 gives the data.

TABLE No. 3  
HOLLOW BRICK BEAM

Load pounds	Deflection, inches		
	East	West	Average
2,000	.0	.0	.0
3,000	.012	.014	.013
4,000	.032	.037	.035
5,000	.058	.065	.062
6,000	.094	.103	.099
7,000	.139	.149	.144
8,000	.184	.197	.191
9,000	.225	.242	.233
10,000	.267	.290	.279
11,000	.330	.362	.346

TABLE No. 4  
EDWARDS TILE BEAM

Load pounds	Deflection, inches		
	East	West	Average
2,450	.0	.0	.0
3,000	.009	.009	.009
4,000	.028	.028	.028
5,000	.054	.053	.053
6,000	.083	.083	.082
7,000	.116	.116	.116
8,000	.155	.157	.156
9,000	.190	.194	.192

Beam No. 5 was similar to No. 4 except that it was made with an 8"x8"x16" tile with double side walls no partitions in the middle. It did not get a fair test as the wooden braces put in place to prevent displacement of the machine while placing the beam on the machine were not removed due to oversight. While the detailed data is of no value and so not given, general observations indicated that this was slightly stiffer and perhaps slightly better in every way than the other tile beams tested. Figure 11 shows the failure and the construction of the beam quite clearly.

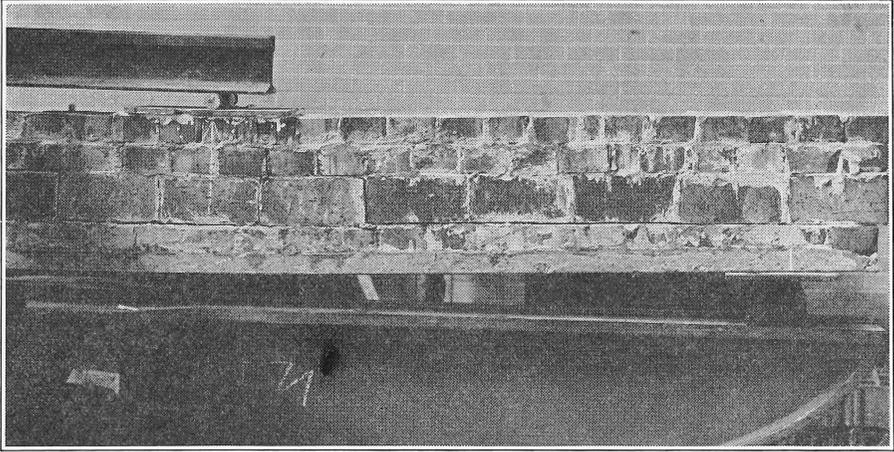


Fig. 7

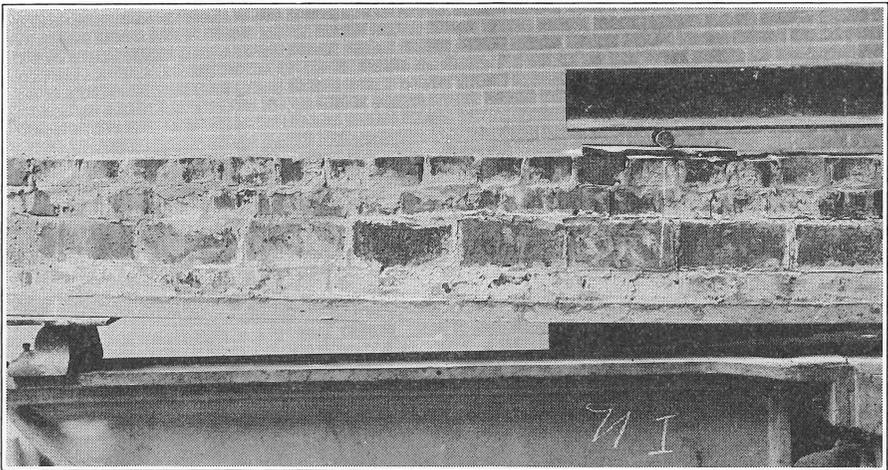
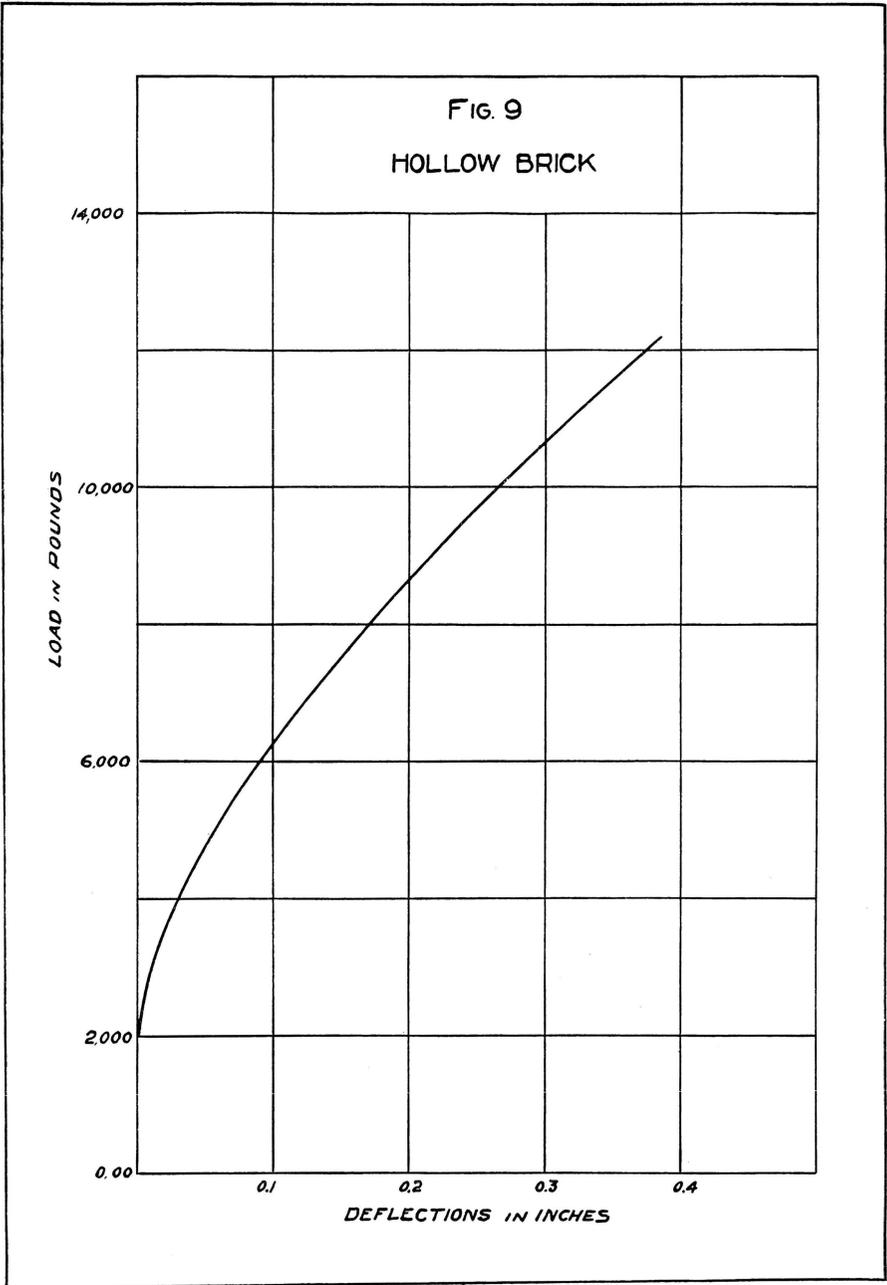


Fig. 8



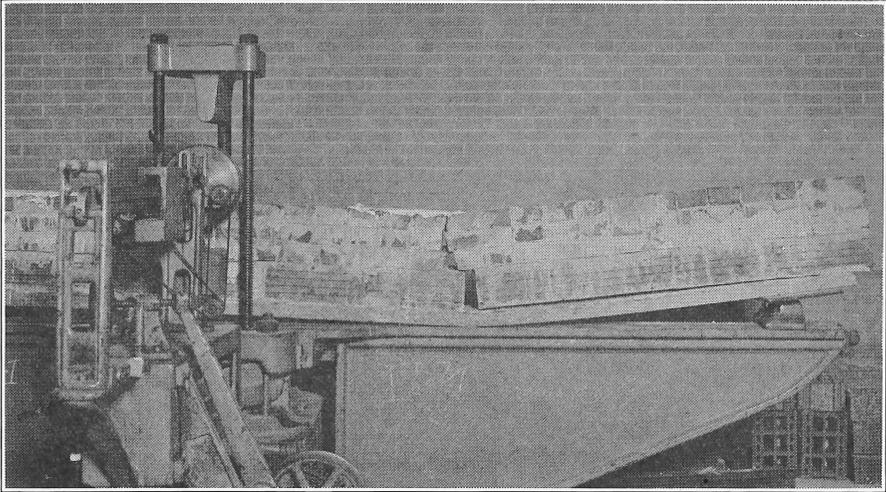


Fig. 10

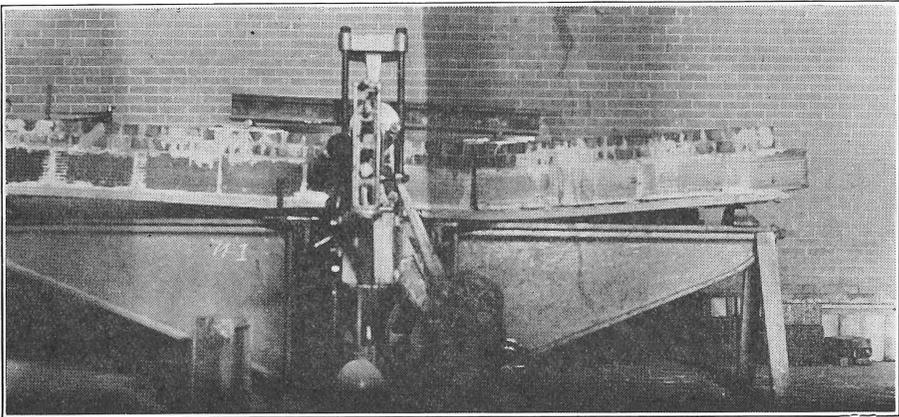


Fig. 11

## CHAPTER III

## DISCUSSION OF RESULTS OF FIRST TESTS

From these tests several problems emerged. The most outstanding problem seemed to be that of a better mortar, or at least a better adhesion of the mortar to the brick or tile as the case may be. A careful study of the tests made in the past indicates that a good 1-3 cement sand mortar, with or without the addition of 10 per cent lime, with grade A brick and with good workmanship, will easily give crushing strengths in excess of 2,000 pounds. The adhesion of the mortar to the brick seems to be definitely less than the tensile strength of the mortar as in practically all cases the mortar tended to pull away from the brick rather than to crack through the mortar. The crushing strength of brickwork is very much in dispute but it is believed that existing data with the results here recorded indicate that with reasonable inspection and control—no more than is required for good concrete work—the figure given can be met.

Several factors affect the strength and adhesion of mortar. The richness of the mortar in cement, the quality, particularly the fineness modulus of the sand, the addition of lime, the amount of water used in the mortar, the absorption of the brick and the extent to which this absorptiveness is satisfied before placing, and the workmanship used in placing the brick are the commonly considered factors. The effects of these different factors are not fully understood and the commonly accepted views on some of them seem to need revision.

The literature on the strengths of mortar seems to be quite limited. Mills', "Materials of Construction", devotes about five pages to the whole subject of mortars and this seems about representative of the space given the subject by other authors. A request addressed to the Bureau of Standards for information on mortars brought some general information and the statement that an investigation had just been started on the whole subject but that they were not prepared to make recommendations at present. The Portland Cement Association gave substantially the same reply. The National Lime Association quotes Mills.

All authorities seem to agree that compressive and tensile strengths increase in proportion to the percentage of cement carried by the mortar. However, the ratio of increase in strengths is not in proportion to the increase in cost. The cost increases so much more rapidly than the strength that a mortar stronger than 1-3 is rarely used commercially. On page 165, Mills gives a graph showing average strengths of 1-2 and 1-3 (cement-sand) mortars which shows average strengths of 1-3 mortars of about 1,100 lbs. per sq. in. at 7 days and over 2,000 lbs. per sq. in. at 28 days. The National Lime Association has collected on page 17 of its bulletin on "Uses of Lime in Construction", data to show the same strength practically for 1-1-6 (cement-lime-sand) and 1-2 (cement-sand) mortar and of course a lower strength for 1-3 (cement-sand). While a 1-1-6 mortar is undoubtedly suitable for straight wall work, it is questionable if these figures can be accepted fully in this interpretation. Certainly the bond strength of 1-1-6 with steel requires investigation and no record of such investigation has been found. It seems that a mortar of 1-3 is the best commercially practicable and efforts to improve it by slight variations offer most promise of results.

Attention to the quality of sand is certainly important. Spaulding, Hyde and Robinson give on page 47 of their "Masonry Structures", a table showing the effect of density, gradation of particles, of sand on the tensile strength of mortars which shows that a good 1-3 mortar may be distinctly superior to a 1-2 made with a less satisfactory sand. A sand density of 0.66, in a 1-2 mortar gave a tensile strength of

223 lbs. per sq. in. at 28 days against a strength of 443 lbs. per sq. in. for a sand having a density of 0.75 in a 1-3 mortar. The densities given are the densities of the mortars and not of the sands. A coarse sand with well graded particles will give the highest strength with all proportions of cement. This seems to apply to adhesion as well as to compression and tension. The effect of adding lime is also in doubt. Unquestionably, the addition or substitution of lime, either paste or hydrate, increases the workability of the mortar on the trowel. This has been much emphasized in the past, probably too much so, as with the increasing use, it is found that it is possible to get straight cement mortars used satisfactorily where formerly they were strenuously objected to. The data referred to above on the substitution of or addition of lime was selected from various sources and so did not represent the results of a controlled test on this point. On page 59 of this same N. L. Assoc. bulletin, Professor MacGregor of Columbia University is quoted as giving the following figures as the results of averages of three specimens in each case.

TABLE No. 5

Mortar Composition			Ultimate Crushing Strengths, lbs. per sq. in.			
			Face Brick		Common Brick	
cement	lime	sand	7 days	28 days	3 months	28 days
1.00	.00	3	2,630	2,840	2,840	1,170
.90	.10	3	3,080	3,170	4,435	1,189
.85	.15	3	2,890	3,230	4,300	1,340
.75	.25	3	3,120	3,470	4,170	1,685
.50	.50	3	2,670	3,100	3,820	1,300
.25	.75	3	1,945	2,370	2,720	1,032
.00	1.00	3	1,535	1,870	1,950	-----

Proportions by volume and method of curing not stated.

The conclusions drawn are that the addition of lime prevents too rapid drying of the mortar and so facilitates proper curing. The following table is given by the kindness of Mr. J. W. McBurney of the U. S. Bureau of Standards from a paper to be presented at a meeting of the A. S. T. M. They are the averages of some 1,000 cylinders tested during the recent investigation on strength of masonry.

Mixture by Volume	Cured in Water; lbs. sq. in.	Cured in Air; lbs. sq. in.
1 C — .25 L — 3 S	-----	90.
1 C — 1.25 L — 6 S	1,070	500
1 C — 1.00 L — 1 S	1,100	750
1 C — 0.10 L — 3 S	3,260	1,950
1 C — 3 S	3,580	1,460

This partially confirms the other data as far as air curing goes. The conditions in brickwork are probably somewhere between the two conditions so that ten per cent of lime is at least not detrimental as to compressive strength. Personal observations indicate that it does not seriously affect the bond strength, though no figures can be quoted.

Workmanship is of greater importance than is usually realized. Mr. McBurney, quoted above, makes the following statements in an article: "Effect of Workmanship

on Strength of Brick Masonry", American Architect, Vol. CXXXII, No. 2532, pages 613-18, Nov. 5, 1927. "Workmanship characterized by filled vertical joints and unfurrowed horizontal beds will give an increase of 24 to 112 per cent wall strength over the results of workmanship characterized by unfilled vertical joints and furrowed horizontal beds.

All other things being equal, the thinner the mortar joint the stronger the masonry for solid brick walls.

The percentage increase in strength of solid walls secured by using filled and unfurrowed joint workmanship over the other type diminishes with increase in strength of the brick.

This statement seems to fully sum up the facts. It is unnecessary to say that the above applies even more fully to beam action than to wall action.

The question of absorption of moisture from the mortar by the brick is one which seems to need thorough investigation. In the past, authorities have almost universally advocated thorough soaking of the brick. Prof. Danforth in the report quoted in the first part of this paper emphasizes this point, Brebner advocates soaking in a tank for some hours. Spaulding, Hyde and Robinson on page 101 of their "Masonry Structures", say that "Bricks should be thoroughly wet before being laid, in order to prevent the water being absorbed from the mortar by the brick. Good adhesion cannot be had between mortar and dry, porous brick." While it is not to be disputed that an absolutely dry, soft burned and porous brick in very hot weather may take too much water from the mortar, there is some indication that a certain amount of absorption is desirable and that with hard burned brick with low absorption, very little wetting may be needed at most times. The Technical News Bulletin of the U. S. Bureau of Standards, No. 126, dated October, 1927 makes the following statement:

"BOND BETWEEN CONCRETE AND HOLLOW TILE." An investigation of the bond between concrete and hollow tile has just been completed. The factors covered by the tests included six different types of hollow tile and concrete mixtures of several consistencies and proportions. The tiles used in making the specimens were in a dry, saturated, or in a semisaturated condition. The specimens were cured either in dry or in damp storage. All specimens were tested when twenty-eight days old, the damp-storage specimens being allowed to dry out fourteen days prior to testing. Concrete control cylinders six by twelve inches were made from the same batches as the concrete in each bond specimen.

The test specimens represented sections from a hollow tile concrete floor, each made up of two tiles joined by a concrete block four inches in thickness. The testing consisted in loading the concrete blocks by a heavy bearing block, the tiles forming the lower base. The desire was to obtain a shearing failure between the concrete. The results obtained indicate that the bond depends largely on the strength of the concrete, but this relation may be disturbed by using saturated tiles or damp curing. The condition least favorable to a good bond was found to exist when specimens were made from saturated tiles and then cured damp. This particular tile had an absorption of about ten per cent. In general, damp curing did not increase the bond strength.

Specific factors which affected the bond strength were:

1. Strength of the concrete, the stronger the concrete the greater the bond.
2. Absorption of the tile, no general law being observable.
3. Amount of water in the tile, the greatest bond being developed by dry tile, slightly less by sprinkled tile, and the least bond by saturated tile. For tiles of the lowest absorption, about 3.1 per cent, there was no material difference in strength

of bond, whereas the greatest bond difference was recorded for the tiles with five per cent absorption. The absorption ranged up to twenty-one per cent.

4. Curing conditions. The dry-cured specimens developed a slightly higher average strength than the damp cured.

As a practical guide in construction, based on the results of these tests, it is recommended that the hollow tiles be sprinkled only enough to work off the dust and loose particles, and that the concrete contain the minimum amount of mixing water necessary for its proper placement."

Also, Technologic Papers of the Bureau of Standards, No. 291, "Tests of Hollow Tile and Concrete Slabs Reinforced in One Direction" by Douglas E. Parsons and Ambrose H. Stang, gives the following statement on page 474: "Experiments have shown that the removal of some of the excess mixing water before the initial set has taken place increases the strength of the concrete by reducing its volume, its water-cement ratio, and its porosity." While both of these references are to hollow tile rather than to brick, the same factors seem to be likely to operate in both cases. From this it would seem that for hard burned brick at least, light sprinkling, not soaking, would be advisable. This is in line with personal experience. The only failure or excessive deflection that has occurred in some forty separate operations of removing forms from reinforced brick slabs after four days, was one slab that was made with very thoroughly soaked brick and a mortar thin enough to flow into the joints as grout. When only partially saturated brick and reasonably dry mortar were used it has been possible consistently to remove the forms at four days. This is in line with knowledge of the water-cement ratio law in concrete. The same thing may be done with concrete except that it is physically impossible to place a concrete dry enough to give this high early strength around reinforcing and where ramming is not permissible. The economy in form material where it can be used once a week instead of once in three weeks is obvious.

The objection will be raised that such control of the moisture in the brick will be difficult. On most concrete jobs of any importance the moisture in the aggregate is determined frequently and accurately and the control of the moisture in brick seems likely to offer no greater difficulty. Another decided advantage, from the standpoint of convenience in laying brick, of allowing some absorption is that it tends to set the mortar slightly and to make it easier for the bricklayer to lay and hold a true wall. The whole subject needs careful tests made on a large number of specimens under accurate control to determine the amount of absorption which will give the best adhesion.

Aside from the question of adhesion and mortar strength, the next most important problem arising seems to be that of the modulus of elasticity of brickwork under such conditions. Brebner, quoted before, says that the ratio,  $n$ , is relatively unimportant and that he used a value of 40. Various engineers consulted suggested values of  $n$  ranging from 20 to 30 and even higher. In order to determine just what the effect of an untrue value of  $n$  would be, Table No. 6 was prepared. Assuming four one-half inch round rods as used in the beams tested, the area of steel  $A_s$  is 0.79 sq. in. With the effective area of the beam 100 sq. in. (8 in. x 12½ in.),  $p$  is 0.0079. From the formula,  $k = \sqrt{2pn + (pn)^2} - pn$ , values of  $k$  can be computed for assumed values of  $n$ , and  $j$  can be calculated for corresponding values from  $j = i - k/3$ . Assuming values of 15, 20 and 30 for  $n$ , the values shown were calculated, the stresses calculated on the basis of a concentrated load of 6,000 lbs. applied at the third points of a twelve foot span. While the unit stresses in the steel does not vary so greatly, the unit stresses in the masonry varies 23.5 per cent between values of 15 and 30 for  $n$ .

Because the strength of the masonry is the most controverted point in this problem and because the increase in dimensions due to the necessity of providing for lower

TABLE No. 6

Value of n	k	j	Pc in lb. sq. in.	Ps	v
n = 15	.382	.873	692	16,724	34.3
n = 20	.425	.858	632	16,950	35.
n = 30	.528	.824	529	17,680	36.4

stresses is undesirable if it can be avoided, it seemed highly desirable to determine this value more accurately. For this purpose, six brick columns, (8 in. x 8 in.) square and approximately 24 inches high were built, three with 1-0.1-3 mortar and three with 1-0.1-2 mortar. These were tested in direct compression and the deformations measured. As they were not ready for test until after it was necessary to go ahead with the construction of the beams, the results of these tests will be discussed with the beam results.

Computations for the second series of beams therefore had to be based on published values. Published values for the modulus of elasticity of brickwork are very few and no author has been found who gives any discussion of the factors affecting this property of brick or brickwork. Mills, in his "Materials of Construction", gives no value for the modulus of elasticity of *brickwork* but makes this statement: "The modulus of elasticity of *bricks* is not a constant for any considerable range of loading. The elastic properties as shown by the stress-strain curve for a compressive test are very similar to those of concrete and mortars. For ranges of loading not exceeding one-fourth of the compressive strength the modulus of elasticity of common bricks is about 1,500,000 to 2,500,000 lbs. per sq. in." He also gives values for other kinds of brick all higher than this. The above statement is given on p. 251 of the first edition. On page 165 of the same book, Mills gives a graph showing the modulus of elasticity of various mortars. The curve for 1-3 cement-sand mortar shows a value of 3,300,000 for the range 200-400 lbs. per sq. in. and ranging downward from that to just under 2,000,000 for the range 1,200 to 1,400 lbs. per sq. in. The range 600 to 800 shows a value of 2,800,000 lbs. per sq. in. He states that these values are "slightly higher" than average cements and mortars show. They are based on one brand of cement only. Kent's Mechanical Engineers' Handbook, 10th edition, 1923, page 404 gives a value of 2,500,000 for each grade 1 and 2 of brick from Pittsburgh district, "considered to be representative of the product east of the Mississippi River" when laid in cement mortar in brick piers ten feet high. For "cement and lime mortar" (proportions not given) and grade 1 brick, the value given is 3,500,000 and for grade 2, 1,550,000 lbs. per sq. in. Prof. Clarence W. Hudson, in his book, "Deflections and Statically Indeterminate Stresses" on page 4 in a table of general values of this sort, states that the values range from 1,000,000 to 3,500,000 with a "rough average" of 2,000,000. Bulletin No. 12 of Columbia University on "Comparative Tests of Clay, Sand-lime and Concrete Brick Masonry" by A. H. Beyer and W. J. Krefeld, gives average values of 1,300,000. Some unpublished results of the Bureau of Standards show values ranging from 831,000 to 2,506,000 where good workmanship was used.

## CHAPTER IV DESCRIPTION OF SECOND SERIES OF TESTS

Considering the discussion in Chapter II and the results of the first solid beam tested, they seem to justify using a value of  $n = 15$  as being approximately correct. On this basis, six types of beams were designed as shown in Figs. 12 and 13. Types S1 and S2 are practically the same except that type S1 is made without stirrups and with a 1-2 mortar while S2 has stirrups and 1-3 mortar. S2-A and S2-B had one-fourth inch stirrups and S2-C had stirrups of No. 9 black iron wire. All stirrups in these beams were placed at intervals of about eight and one-half inches—the length of one brick, so they could be put in vertical mortar joints without cutting or breaking the brick. This spacing is slightly larger than the allowed spacing when spacing does not exceed  $0.45d$ . Since it was desirable to know the point at which yielding under diagonal tension would occur and since for 2,000 lbs. concrete diagonal tension reinforcement would not be required, it was entirely omitted from the one series and made very light in all but two specimens of the others.

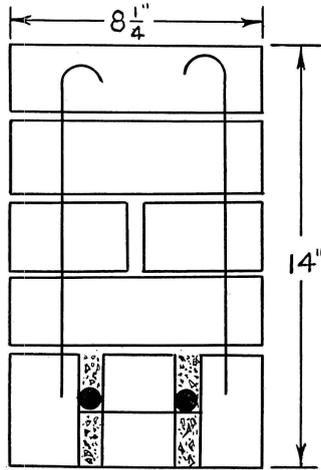
Types H1 and H2 take the same amount of material but have the hollow space located differently. H1 was also reinforced with four one-half inch bars while all other beams of this series were reinforced with two three-fourths inch round bars. The one-half inch bars were simply cut to length and put into the concrete without hooking the ends. No slipping occurred in any of the specimens made this way with one-half inch bars. S1-A was reinforced with two three-fourth inch bars not hooked. These slipped under a load slightly below that of the elastic limit of the steel, showing that it is not safe to depend on bond alone when the larger sizes of plain bars are used.

Four one-half inch round bars = 0.79 sq. in. steel area. With  $bd = 100$  sq. in. this gives a value of  $p = 0.0079$ . (Although the average length of the brick used was  $8\frac{1}{4}$  inches, the difficulty of getting full filling of the joints at the ends probably reduced the effective width to eight inches which is used as the nominal width of the beams in all these computations.) In the S1 series the value of  $d$  is  $12\frac{3}{4}$  inches, giving a value of 102 sq. in. for  $bd$ . With two  $\frac{3}{4}$  inch round rods having an area of 0.88 sq. in., the value of  $p$  would be 0.0086 and the same for the S2 series. H2 series with slightly less depth gives  $p = 0.0089$  for beam H2-C only and 0.0088 for H2-A and H2-B. The H3 series with  $bd = 96$  sq. in. has  $p = 0.0092$ . The H4 series were considerably deeper and gave a value of 0.0078 for H4-A and 0.0076 H4-B and H4-C. The original plan was to make them all as nearly 8 in. x 12 in. as practicable. The variations from this are due to different arrangements of the units and slight variations of the thickness of the mortar joints. On the basis of  $f_c$  at 650 lbs. per sq. in. and  $f_s$  at 16,000 lbs. per sq. in. Spalding, Hyde and Robinson give the value of  $p$  as 0.0077 when  $n$  equals 15. On this basis, those beams which were reinforced with  $\frac{3}{4}$  inch rods had a slight excess of steel.

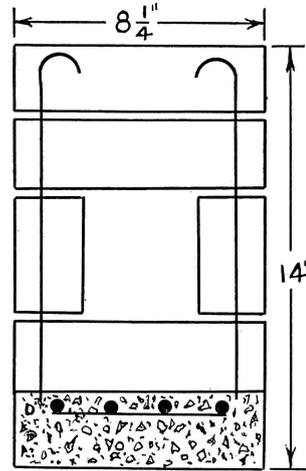
For all these types of beams, a washed river sand with a fineness modulus of 2.84 was secured. The mortar (cement-lime-sand) was 1-0.1-3 by volume. A bricklayer was secured and instructed that all beams were to be laid up with as thin mortar joints as he considered practicable, fully filled both horizontal and vertical, and with what he would consider good commercial workmanship. The mortar was mixed by volume measurements by a helper of average intelligence with only casual inspection. He was directed to make the mortar in reasonably small batches and to deliver it as dry as the masons could use it. The placing of reinforcement was supervised closely to see that it was accurately placed. The bricklayer cooperated well and it is believed that the resulting work represents what could be expected on a job with the degree of supervision usually given reinforced concrete after the masons had learned the new

Fig. 12

BEAM CROSS-SECTIONS

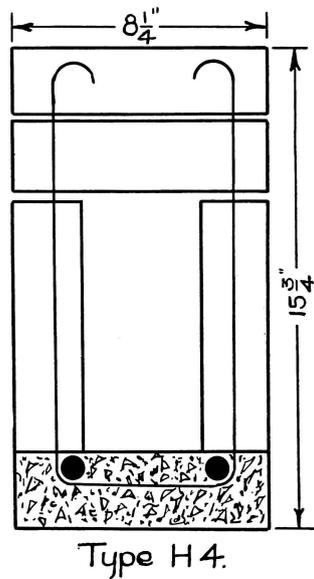
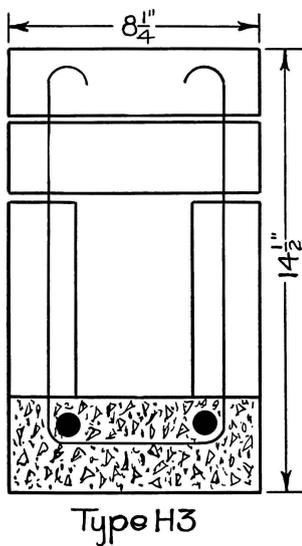
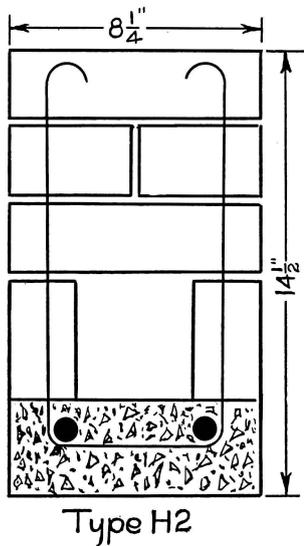


Types S1 and S2



Type H1

Fig. 13  
BEAM CROSS-SECTIONS



technique. It was difficult to impress the idea that all joints must be really full and one beam failed because of insufficient contact of mortar and steel, due to incomplete joint filling.

The brick used were the same in all of the experiments and their properties have been previously given. They were rather poorer than the average of common brick in evenness of sizing and in regularity of shape. The moisture in the brick was controlled only by judging the feel and appearance of the brick. When thought necessary they were dampened by throwing water over the pile, preferably in the evening or just before noon as so to allow the absorption to become somewhat equalized before work was resumed. The attempt was made to get some absorption from the mortar, enough to reduce the water-cement ratio to a low figure but not enough to injure the strength of the cement. So far as could be judged, both during construction and



Fig. 14

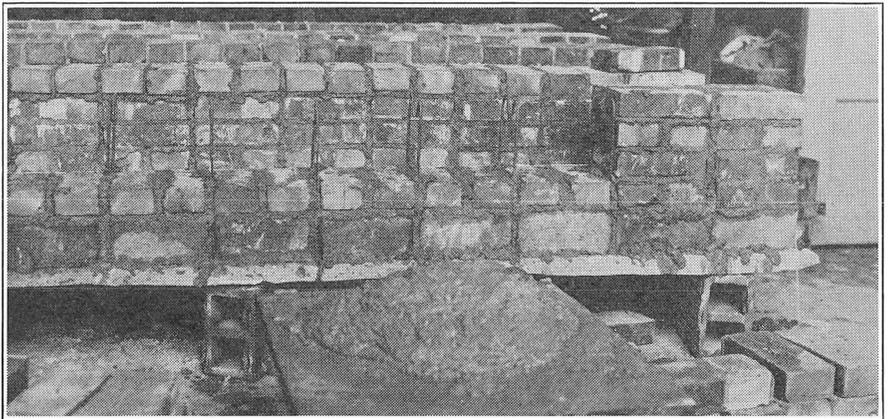


Fig. 15

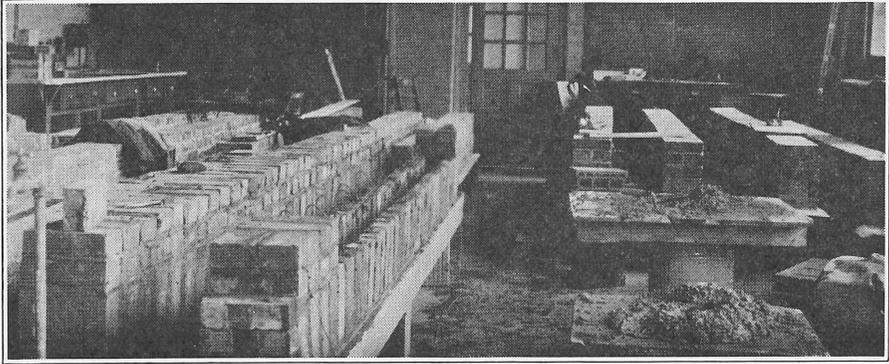


Fig. 16

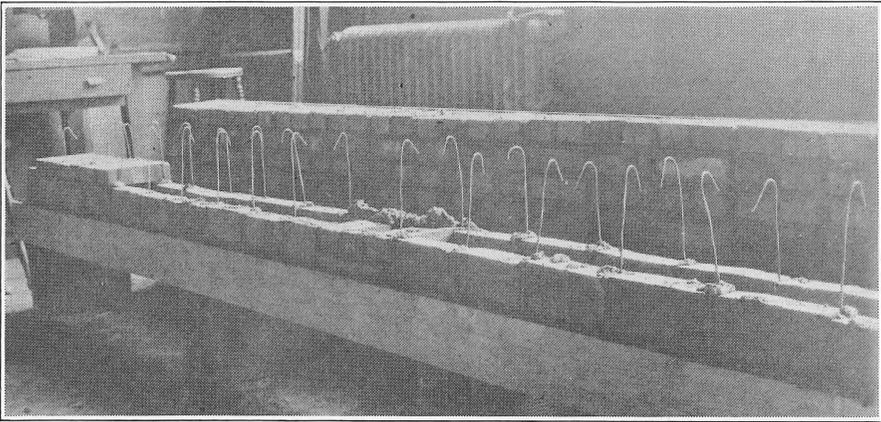


Fig. 17

during testing, the mortar was not injured by excessive absorption. Though compressive stresses reached comparatively high figures, there was no single case of crushing either brick or mortar in compression. A few bricks broke across due to transverse stress. Figs. 14 to 20 show various aspects of the construction and are clear enough without further explanation.

The beams were all constructed in a room steam heated, some distance from the testing laboratory. No special precautions were taken about curing except to prevent freezing and to maintain a normal degree of humidity. On one occasion when some of the beams seemed to be drying rather rapidly, a few buckets of water were thrown over them and on a couple of days when the air seemed dry, the floor was wet down. Outside doors were kept open much of the time. With the exception that they were protected from the sun, they were cured under approximately normal outdoor summer conditions when no rain occurs for some days.

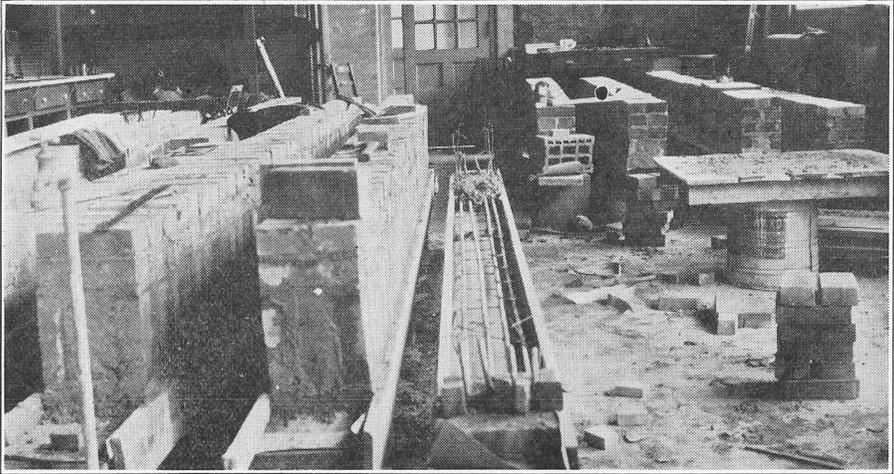


Fig. 18

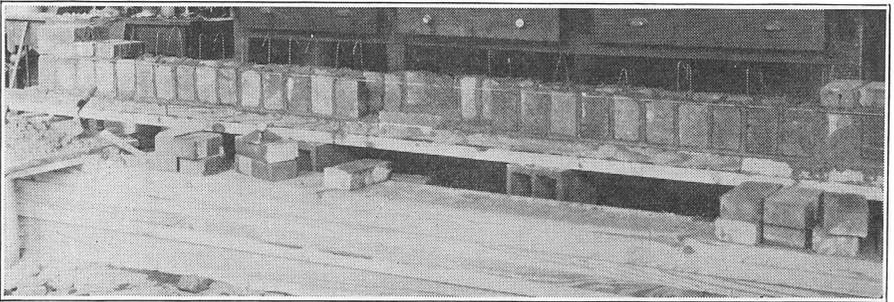


Fig. 19

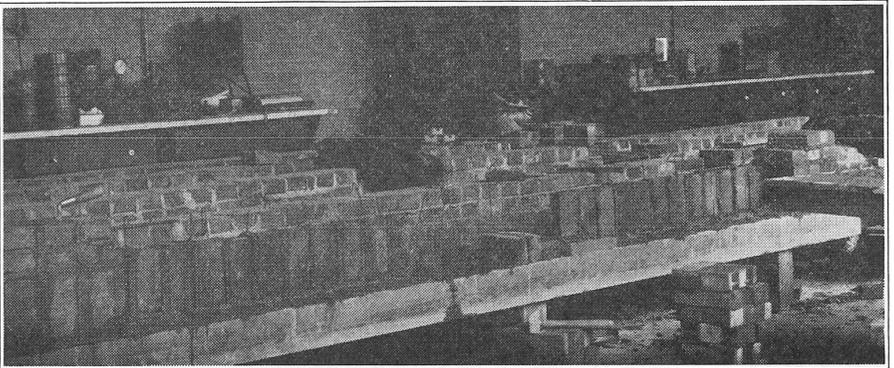


Fig. 20

Due to unavoidable delays, it was not possible to begin testing at the end of twenty-eight days. Most of the beams were tested at about six weeks age but it is believed that the strength developed is not materially different from the twenty-eight day strength.

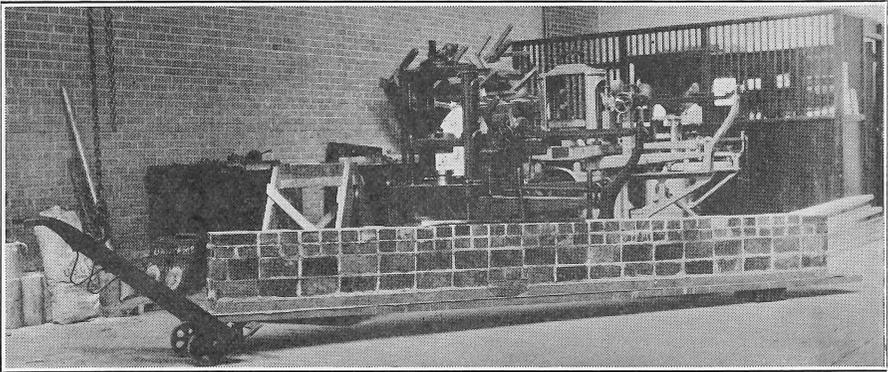


Fig. 21

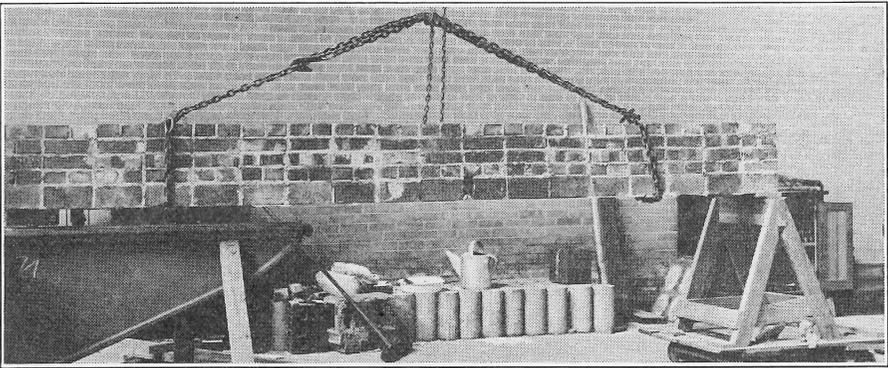


Fig. 22

Fig. 21 shows the method of transporting the specimens from the room where they were built to the testing machine and Fig. 22 shows the manner of placing the beams on the table of the machine. In this way, two men could handle a beam fairly easily. The same small iron dolly was used under the rear end of the beam when moving it on the floor and under the front end when putting it on the machine. The hoist not shown in the picture was an ordinary triplex chain block. Contrary to ordinary expectation, no particular difficulty was experienced in handling these beams in spite of the comparatively crude apparatus available. Only two were damaged in handling. One was dropped about a foot in getting it onto the trucks and cracked through a vertical mortar joint near one end. When put on the machine, this point was reinforced by wrapping some wire around the beam parallel to the crack. Failure occurred at the other end and in a similar way and under similar loads

to the other beams of the same construction. The other one damaged was cracked vertically by putting the dolly under a point near the center of the beam because the workmen thought it could be maneuvered through doors more easily so. This crack came near one of the loading points and apparently did not affect the strength of the beam at all. In one or two cases, the mason forgot to remove the brick to make space for setting the strain gauge on the steel and it was necessary to completely smash a brick on each side in order to get it out. This was done with a heavy hand hammer without any special precaution but did not result in loosening other brick. In the six beams where the brick were not supported by concrete, there was no case of brick falling away from the bottom of the beam even after failure. In every case, deflection was carried far past the point of failure in order to open up the cracks sufficiently to make them show in the pictures. In every case, after this severe treatment, the beams carried the full load given as the "safe" load for that type without further deformation. There were no cases of sudden failure in the beams. In every case the beams were removed whole from the testing machine by the same method as they were placed on it and then were wheeled aside to be broken up.

## CHAPTER V

## TESTS OF BEAMS

As before mentioned, all beams were tested on a 50,000 lb. Olsen beam machine on twelve foot centers and with the load symmetrically applied at third points. This gave a section in the center four feet long over which the bending moment was constant and therefore convenient for various measurements. With the exception of the H1 series, the following data were recorded: total load, strain in steel over eight inch length, strain in masonry both at the top and bottom of the beam and the de-

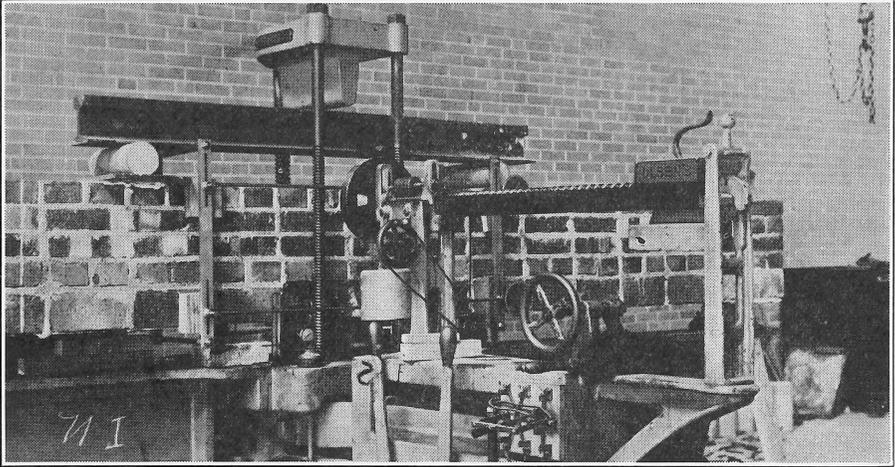


Fig. 23

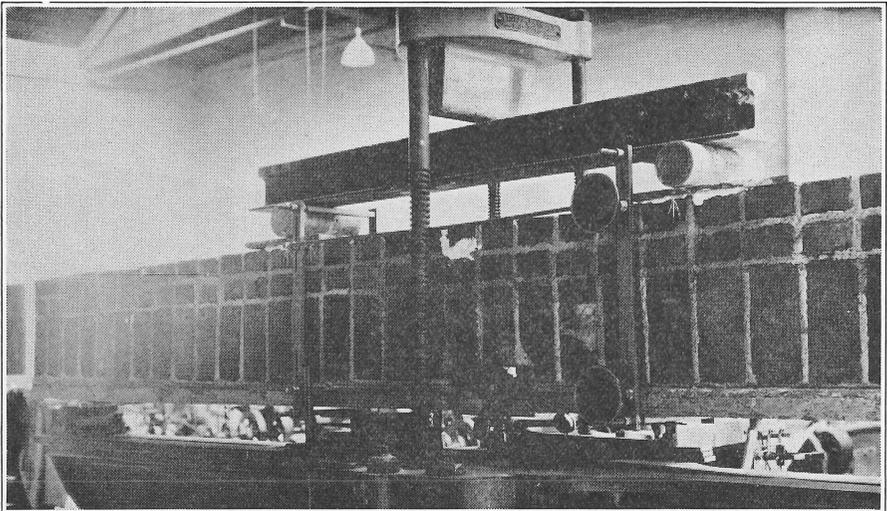


Fig. 24

flection. All measurements were made on both sides of the beam and averaged. The steel deformation was measured with a Berry strain gauge registering on an Ames' dial reading to 0.0002 of an inch direct. The masonry deformations were measured by an Olsen Beam Apparatus over a distance of 40 inches. The deflections were measured by Riehle lever type deflectometers reading by vernier 0.001 inches. The deflectometers were mounted on the bed of the testing machine and would really record not only the deflection of the beam but any tendency of the machine bed to give under the load. Therefore the deflections recorded are rather in excess than otherwise. In one or two tables where a column of data is omitted, the apparatus, failed to function for some reason. In one or two cases, the beam apparatus was accidentally struck and thrown off so that it was impossible to get readings for one or two of the last loads. Figs. 23 and 24 show the apparatus in place for tests.

The procedure was as follows: after placing the beam, it was weighted and all measuring apparatus and the railroad rail used for applying the load was put in position and everything made ready to start loading. The dead weight reading of the scale beam under these conditions was taken as the zero reading, all apparatus being set to zero under this load. This introduces some error in the deformations as the strains recorded are really those due only to the live load and not to the total combined load. However, the first load reading was such as to be in most cases practically double the dead load. The deformations recorded for this load were so small that the dead load deformations seem not important. The deflections recorded were very small but regular in amount as shown by the graphs attached. The loads were applied in 1,000 lb. increments by the slowest motion on the machine. It usually took about one and one-half to two minutes to apply 1,000 lbs. After the load was nicely balanced, the machine operator read the instruments in order and a helper recorded. After failure became definite, loading was continued to such a point that the cracks could be photographed. In some cases where the crack is too small to show in the photograph, it is outlined with chalk.

In order to get the beams out of the room in which they were made, it was



Fig. 25

necessary to test them in a different order from that in which they were built. The date of building and of testing will be given in each case. For convenience in reference the data will be given in regular order.

Beam S1-A, built March 6, 1928 and tested April 24th, gave the data shown in Table No. 7. The steel was not hooked on the ends and the mortar was not too well packed around it so failure occurred by slipping of the steel. Fig. 25 shows the crack which opened up after the steel had slipped about  $\frac{3}{8}$ ". Even after this much slipping, the beam carried over 6,000 lbs.

Beam S1-B made March 6th and tested April 19th, failed in almost exactly the same way as S1-A. The steel in it was hooked but the mortar was very poorly filled

TABLE No. 7  
BEAM S1-A

Load	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1905	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0014	.0012	.0013	.0013	.0014	.0013	.0003	.0004	.00035	.016	.020	.018
4000	.0028	.0024	.0026	.0025	.0025	.0025	.0008	.0007	.00075	.030	.035	.032
5000	.0041	.0035	.0038	.0037	.0037	.0037	.0012	.0010	.0011	.041	.049	.045
6000	.0058	.0052	.0055	.0052	.0052	.0052	.0016	.0014	.0015	.068	.069	.068
7000	.0088	.0074	.0081	.0070	.0073	.0071	.0022	.0022	.0022	.088	.100	.093
8000	.0118	.0094	.0106	.0090	.0093	.0091	.0030	.0028	.0029	.114	.129	.121
9000	.0147	.0115	.0131	.0110	.0116	.0113	.0036	.0034	.0035	.144	.161	.151
10000	.0126	.0136	.0156	.0130	.0140	.0135	.0044	.0040	.0042	.153	.195	.174
11000	.0201	.0156	.0178	.0152	.0162	.0157	.0052	.0048	.0050	.202	.226	.214
12000	.0225	.0180	.0202	.0173	.0186	.0178	.0056	.0056	.0056	.232	.256	.244
13000	.0244	.0205	.0224	.0199	.0207	.0200	.0066	.0062	.0064	.265	.289	.279
14000	.0265	.0226	.0245	.0216	.0230	.0223	.0073	.0070	.0071	.298	.320	.309



Fig. 26



Fig. 27

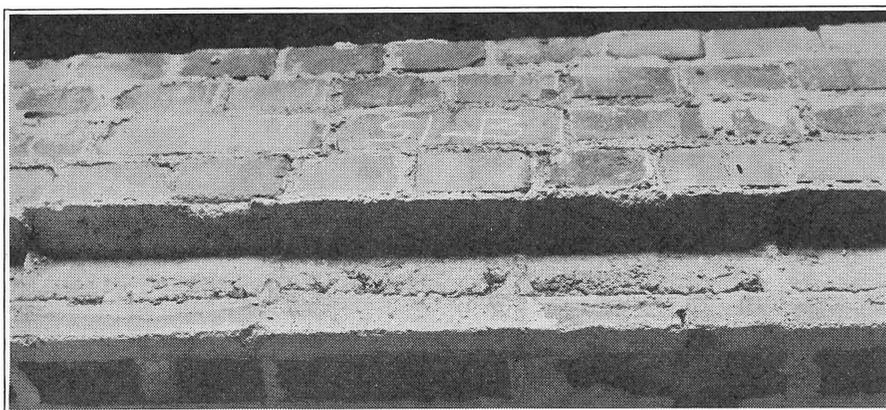


Fig. 28

around the rod. Fig. 26 shows the type of failure at the north load point and Figure 27 shows the poor filling around the rod at the hook. Fig. 28 shows the poor filling around the middle part of the rod showing that the mortar was not in contact with the rod at all below for some distance and the sides of the rod also were not covered. Table No. 8 gives the data which is applicable to neutral axis and other calculations as slipping apparently did not take place until about the time it failed completely.

Fig. 29 shows the type of failure of beam S1-C, due to shear or diagonal tension. It is typical of many of the later failures. Note how the end seems to be shoved off. This beam was made March 7th and tested April 18th. Table 9 gives the data for this beam. Fig. 30 shows the deflections of all three S1 beams plotted on one sheet and an average curve drawn. Figure 31 shows the steel deformations plotted against loads.

Beam S2-A, made March 7th and tested April 17th, failed quite definitely in

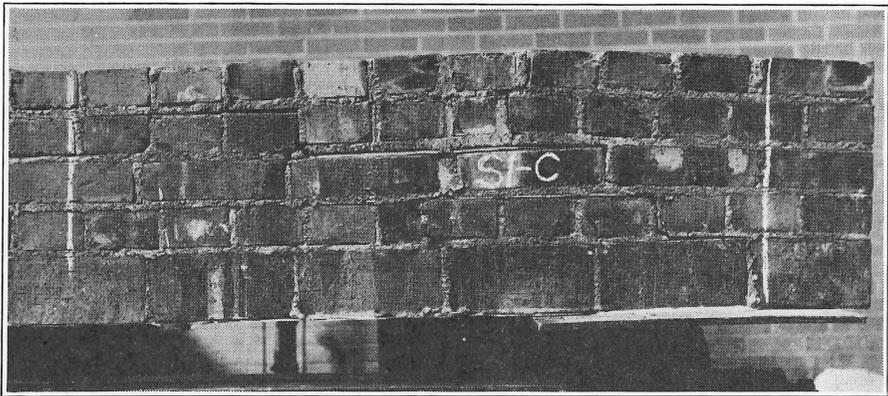


Fig. 29

tension in the longitudinal steel. It had  $\frac{1}{4}$  inch stirrups. Careful examination failed to show any sign of crushing in either mortar or brick. Table No. 10 shows the data secured from this beam.

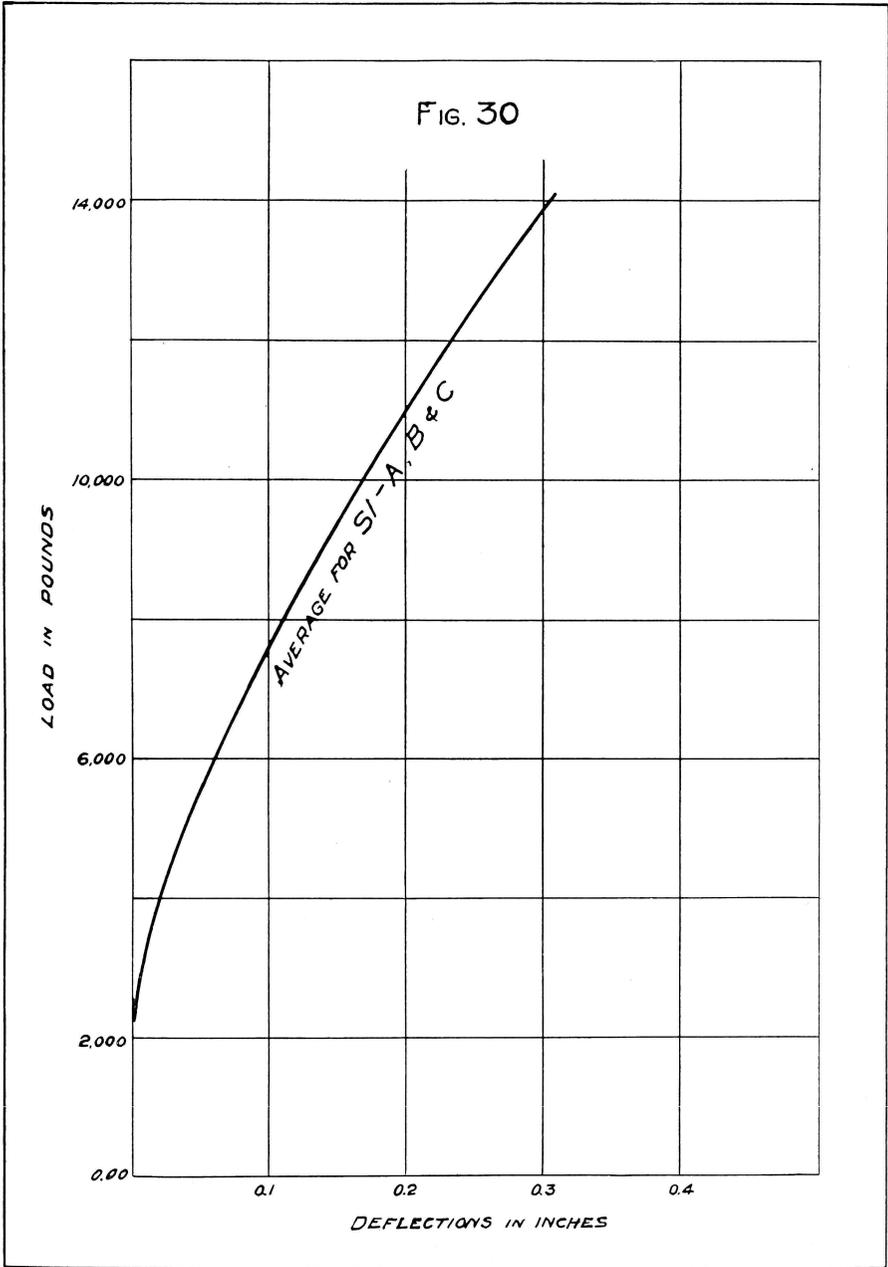
TABLE No. 8  
BEAM S1-B

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge			East	West	Av.
	East	West	Av.	East	West	Av.	East	West	Av.			
2000	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0012	.0012	.0012	.0008	.0015	.0011	0	0	0	.014	.015	.014
4000	.0025	.0025	.0025	.0027	.0035	.0031	.0007	.0010	.0008	.033	.027	.030
5000	.0039	.0038	.0038	.0045	.0057	.0052	.0011	.0012	.0011	.051	.032	.041
6000	.0054	.0052	.0053	.0067	.0080	.0073	.0016	.0016	.0016	.070	.053	.061
7000	.0068	.0065	.0066	.0090	.0170	.0130	.0020	.0018	.0019	.093	.076	.084
5700	.0035	.0060	.0047	.0070	.0088	.0079	.0019	.0014	.0016	.390	.363	.376

Fig. 32 shows the steel in beam S2-B after failure and also the single-vertical crack that opened in the mortar joint just behind the screw. Failure in this beam was also by tension in the longitudinal steel. Unfortunately, struts placed to prevent displacing the table of the testing machine when putting the beam in place were not removed until the test had stressed the steel to its elastic limit. The loads as originally observed were not therefore the true loads as the steel began to stretch at an observed load of 11,000 lbs. which was obviously wrong.

After removing the struts, the load registered 18,000 lbs. The loads therefore, given in Table No. 11 are not observed loads. Loads calculated from the load against deformation of steel curve in Beam S2-A. These are believed to be approximately correct. S2-B was made on March 7th and tested April 17th.

Fig. 33 shows the failure of beam S2-C after the longitudinal steel had been stretched enough to open the crack near the center of the beam to about  $\frac{3}{8}$  inch. This beam was loaded repeatedly before failure. The load was applied in increments of 1,000 lbs. up to 12,000 lbs. and then released with the faster speeds of the machine.



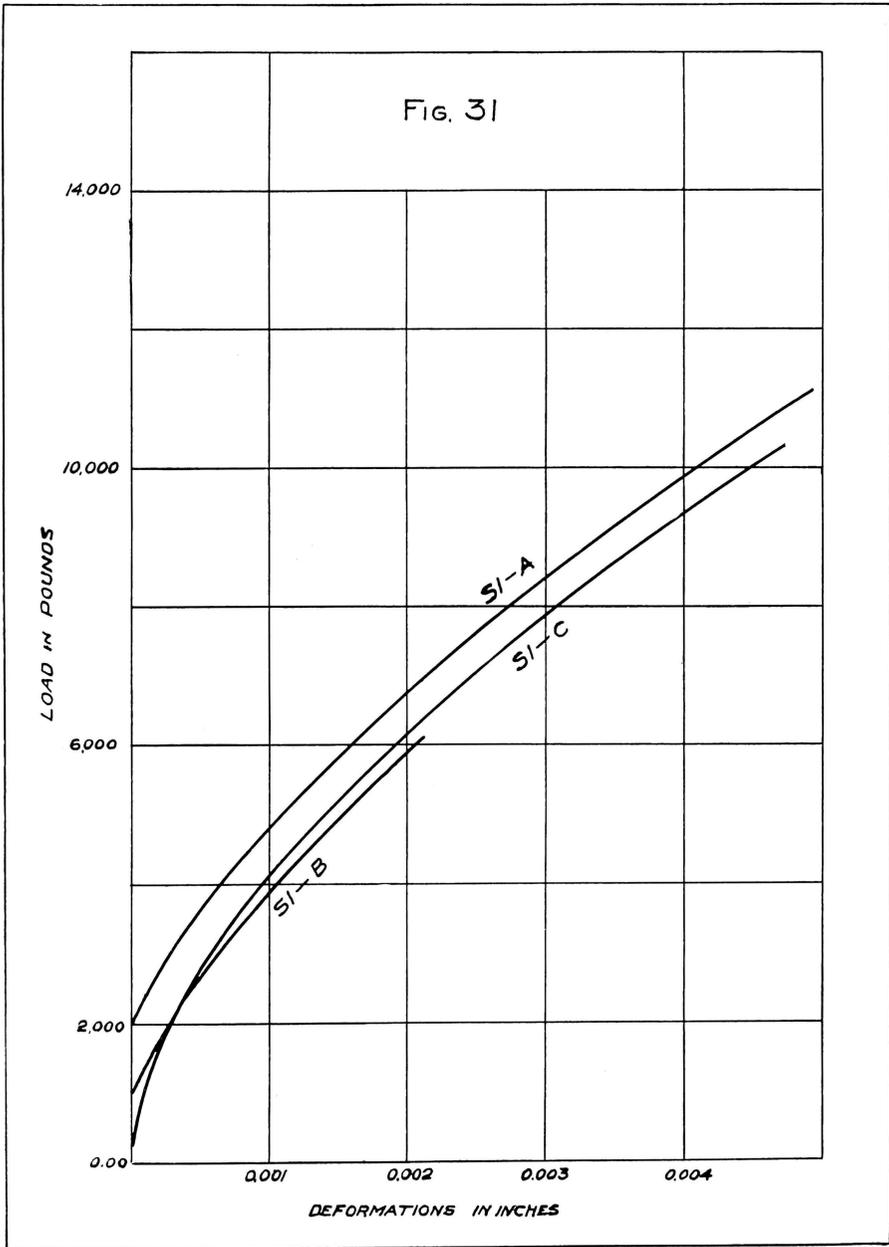


TABLE No. 9  
BEAM S1-C

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
2000	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0016	.0008	.0012	.0007	.0025	.0016	.00004	.00004	.00004	.011	.014	.022
4000	.0033	.0030	.0031	.0024	.0053	.0038	.0004	.00004	.00022	.024	.027	.025
5000	.0050	.0030	.0040	.0040	.0084	.0062	.0008	.0018	.0013	.039	.047	.045
6000	.0070	.0042	.0056	.0055	.0125	.0090	.0013	.0019	.0016	.055	.063	.059
7000	.0094	.0052	.0073	.0070	.0155	.0112	.0019	.0026	.0022	.083	.086	.084
8000	.0117	.0065	.0091	.0098	.0190	.0144	.0025	.0027	.0026	.096	.102	.099
9000	.0140	.0077	.0108	.0125	.0223	.0174	.0032	.0027	.0029	.121	.125	.125
10000	.0166	.0080	.0123	.0153	.0264	.0208	.0038	.0040	.0039	.148	.149	.148
11000	.0194	.0103	.0148	.0178	.0293	.0235	.0044	.0048	.0046	.182	.183	.182

The load was again applied by the slow speed and again released several times. After that the load was applied in increments and released in the same way several times, readings being taken in each direction. Apparently the beam apparatus was disturbed in some way as it gave negative readings on some dials—indicating a

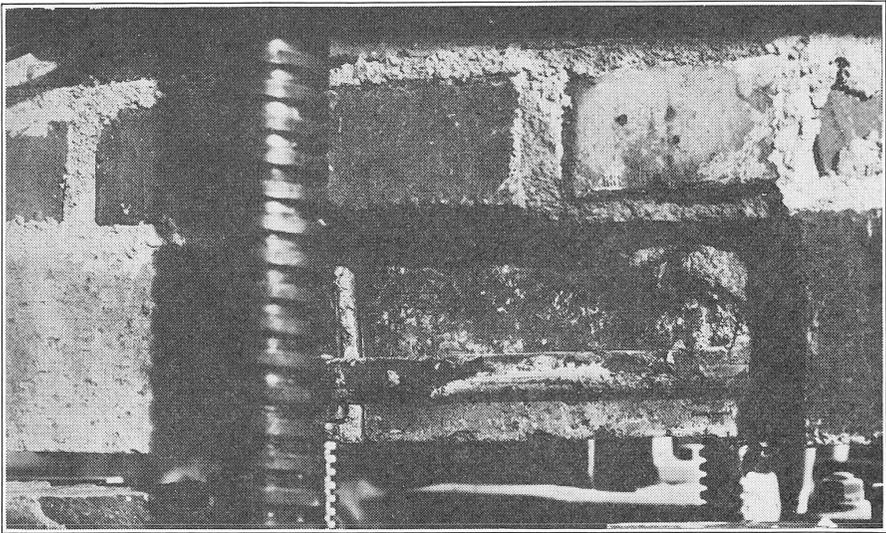


Fig. 32

recovery greater than the original deformation. This was not confirmed by the deflection and steel deformation readings so the detail data is not given here. After being loaded five times to 14,000 lbs. the permanent set in the steel was .0006 in eight inch length and the deflection was .036 with the load totally removed. The steel began to stretch at about 17,000 lbs. but the ultimate load was over 18,000 lbs. Note in the picture that the whole of the compressive load is again being carried by the one upper course of brick, a crack definitely separating the first and second

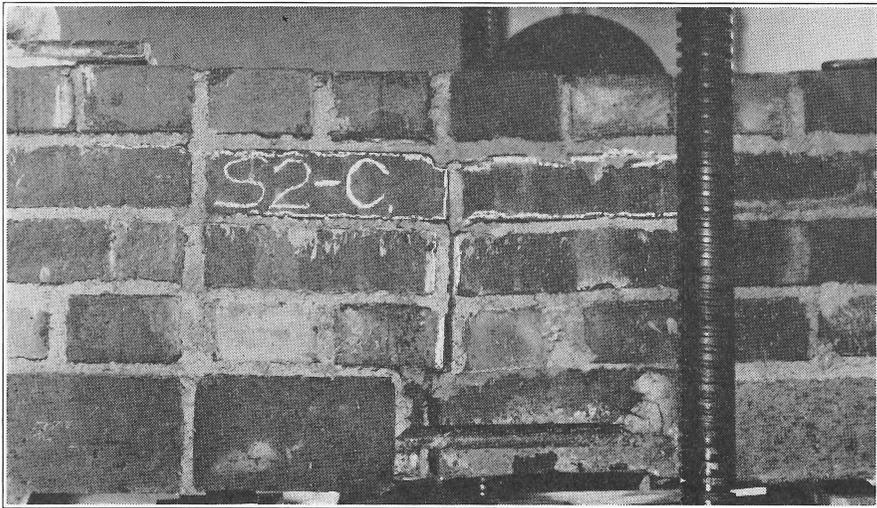
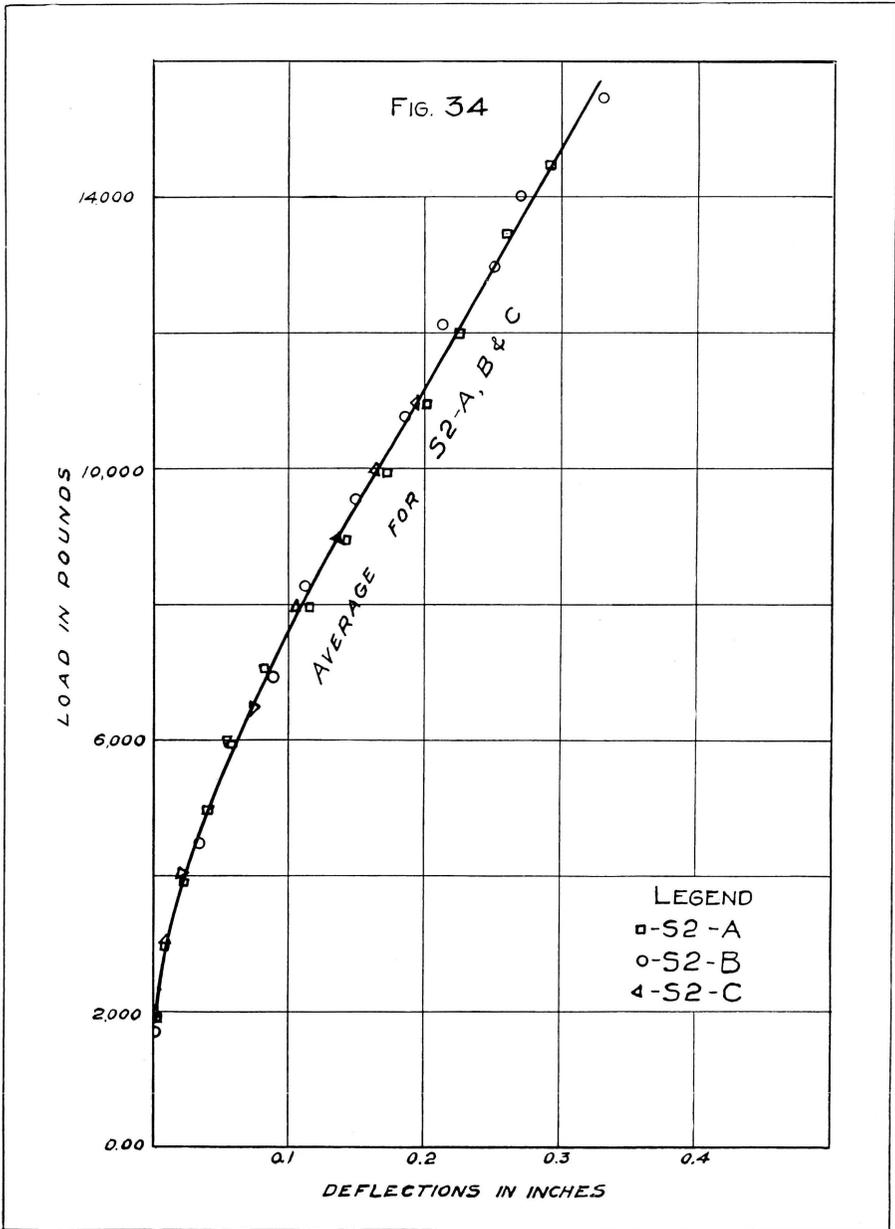


Fig. 33

TABLE No. 10  
BEAM S2-C

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
22000	0	0	0	0	0	0	0	0	0	0	0	0
33000	.0014	0	.0007	.0014	.0015	.0014	.000	.0002	.0001	.011	.014	.012
4000	.0023	.0010	.0016	.0030	.0030	.003	.000	.0005	.0002	.025	.028	.026
5000	.0035	.0016	.0025	.0040	.0045	.0042	.000	.0011	.0005	.038	.052	.045
6000	.0052	.0030	.0041	.0080	.0040	.0060	.000	.0018	.0007	.061	.068	.064
7000	.0060	.0040	.0050	.0125	.0134	.0129	.000	.0026	.0013	.087	.095	.091
8000	.0090	.0050	.0070	.0162	.0175	.0168	.0005	.0034	.0019	.115	.125	.120
9000	.0107	.0060	.0083	.0195	.0213	.0204	.0011	.0040	.0025	.141	.152	.146
10000	.0126	.0072	.0099	.0233	.0255	.0244	.0018	.0048	.0033	.171	.182	.176
11000	.0140	.0083	.0115	.0265	.0294	.0249	.0024	.0054	.0039	.201	.208	.204
12000	.0155	.0095	.0125	.0304	.0333	.0318	.0030	.0062	.0046	.228	.236	.232
13000	.0173	.0105	.0139	.0340	.0370	.035	.0037	.0069	.0053	.245	.265	.255
14000	.0192	.0118	.0155	.0395	.0408	.0391	.0044	.0075	.0064	.261	.281	.271
15000	.0222	.0130	.0176	.0385	.0445	.0415	.0050	.0082	.0066	.303	.319	.311
16000	.0235	.0144	.0189	.0420	.0486	.0453	.0058	.0089	.0073	.323	.346	.334
17000	.0252	.0157	.0204	.0435	.0560	.0497	.0068	.0140	.0104	.350	.378	.364

courses. This beam was only reinforced with No. 9 wire stirrups. Table No. 12 gives the data from the first time the load was applied up to 12,000 lbs. The value given for 17,000 lbs. is the reading taken after repeated loadings and just before failure. Fig. 34 shows the deflections in the three beams of the S2 series plotted against loads and the average curve drawn. Fig. 35 shows the steel deformation plotted against loads for this series. S2-C was made March 8 and tested April 23.



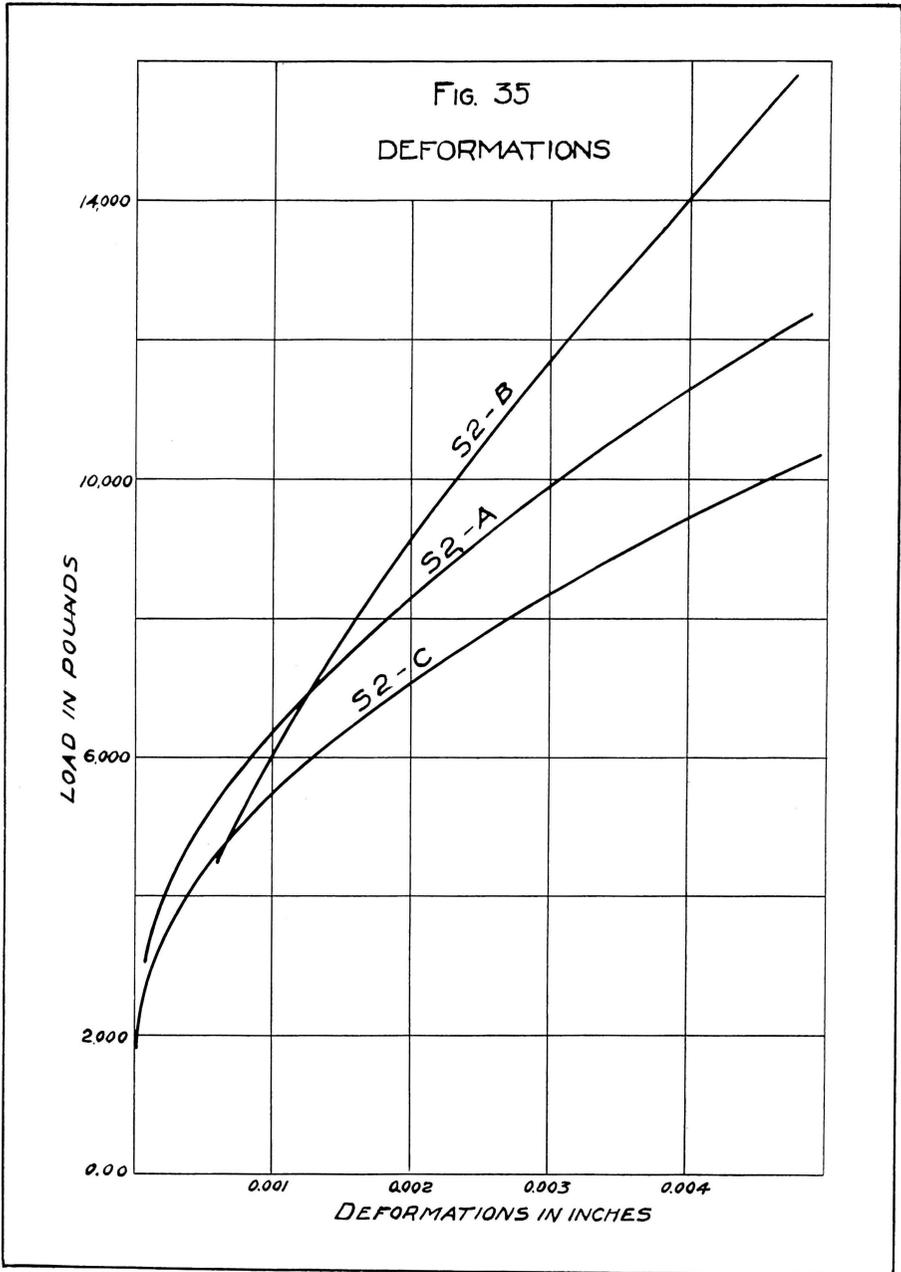


TABLE No. 11  
BEAM S2-B

Load	40" Gauge				8" Gauge		Deflection inches	
	East		West		East	West	East	West
	Upper	Lower	Upper	Lower				
1200	0	0	0	0	0	0	0	0
3000	3.3	3.6	1.0	3.2	95	94	.037	.037
4000	6.8	10.5	5.6	7.5	88	90	.075	.079
5000	8.8	16.0	7.5	11.8	82.5	85	.110	.120
6000	9.5	21.8	9.8	12.5	77.5	80	.146	.157
7000	9.0	27.8	12.5	19.2	72.5	75	.181	.194
8000	9.0	34.0	15.3	23.0	67.5	70	.233	.216
9000	9.0	39.5	17.0	27.0	62.5	64	.250	.272
10000	9.0	44.5	18.4	31.0	58.5	60	.281	.310
11000	10.0	50.0	19.0	35.0	54.5	56	.315	.349
11150	21.0	126.5	38.5	1103.5	38.0	29	.517	.533

The H1 series were reinforced with the 1/2 inch rods and so no measurements were taken of the strain in the steel. Figure 36 shows the failure of H1-A by horizontal shear. Note the vertical crack at the first vertical mortar joint. Table No. 13 gives the detail data for this beam. Unfortunately, the beam apparatus was accidentally disturbed so that the last few West readings could not be taken accurately so they were omitted. The west deflection ran off the scale of the deflectometer and so could not be recorded. It should be noted that the load carried by this beam was 2,000 lbs. higher than that carried by the best of the S1 series in spite of the fact that it was 200 lbs. lighter. H1-A was made March 12 and tested April 25.

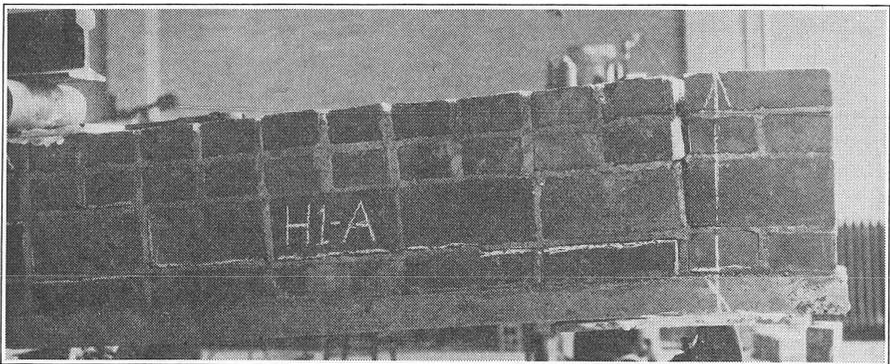


Fig. 36

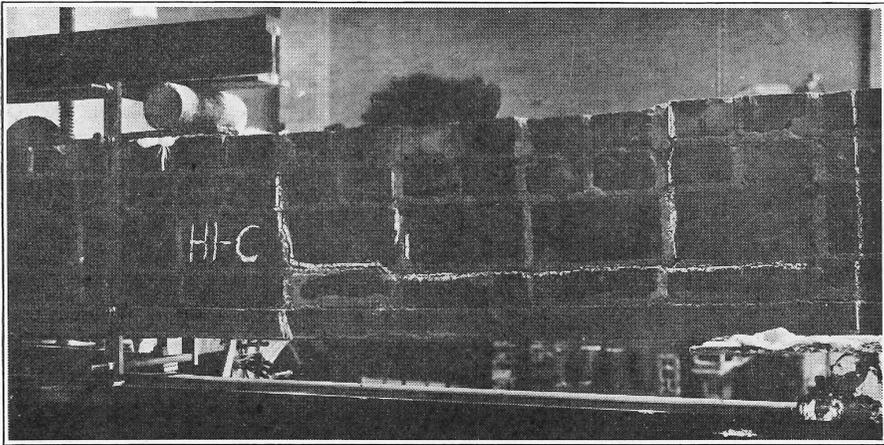


Fig. 37

TABLE No. 12  
BEAM S2-C

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1800	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0016	.0017	.0016	.0005	.0024	.0014	.0001	.0002	.0001	.014	.011	.012
4000	.0030	.0020	.0025	.0019	.0037	.0028	.0002	.0004	.0003	.028	.025	.026
5000	.0048	.0034	.0041	.0037	.0064	.0050	.0006	.0012	.0009	.045	.042	.043
6000	.0067	.0052	.0059	.0067	.0095	.0086	.0010	.0020	.0015	.065	.063	.064
7000	.0093	.0074	.0083	.0104	.0134	.0119	.0012	.0026	.0019	.090	.088	.089
8000	.0114	.0090	.0102	.0135	.0170	.0152	.0016	.0032	.0024	.113	.113	.113
9000	.0132	.0106	.0119	.0160	.0200	.0180	.0040	.0040	.0040	.142	.141	.141
10000	.0114	.0115	.0114	.0195	.0234	.0215	.0048	.0048	.0048	.172	.165	.168
11000	.0114	.0090	.0102	.0217	.0225	.0221	.0058	.0058	.0058	.210	.187	.198
12000	.0116	.0110	.0113	.0240	.0316	.0278	.0070	.0064	.0067	.241	.215	.228
17000	.0192	.0263	.0227	.0416	.0467	.0441	.0086	.0100	.0093	.395	.380	.387

Exactly the same type of failure occurred in H1-B as in H1-A. The cracks run slightly differently and the end was not pushed off as it was in the H1-A because slipping occurred all the way out to the end. The vertical mortar joints were considerably displaced without, however, breaking the stirrups. Original failure occurred at 13,000 lbs. but the load was increased to just 14,000 before slipping occurred. One joint was injured in moving the beam to the machine by dropping it about one foot. In spite of this, the result was satisfactory. Table No. 14 gives the data on this beam which was made March 12 and tested on April 25.

TABLE No. 13  
BEAM H1-A

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1645	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0037	.0026	.0031	.0	.0016	.0008	0	0	0	.020	.022	.021
4000	.0050	.0040	.0045	.0008	.0023	.0015				.035	.035	.035
5000	.0068	.0062	.0065	.0027	.0035	.0031				.053	.053	.053
6000	.0088	.0084	.0086	.0045	.0053	.0049				.074	.074	.074
7000	.0114	.0112	.0113	.0090	.0092	.0091				.105	.103	.104
8000	.0140	.0140	.0140	.0137	.0132	.0135				.145	.145	.143
9000	.0164	.0164	.0164	.0180	.0172	.0176				.181	.175	.178
10000	.0185	.0187	.0186	.0223	.0212	.0218				.218	.209	.213
11000	.0206	.0212	.0209	.0264	.0245	.0254				.252	.244	.248
12000	.0227	.0237	.0232	.0303	.0277	.0290				.288	.285	.286
13000	.0262	.0270	.0266	.0350	.0323	.0336				.347	.352	.349
14000	.0280	X	.0280	.0380	.0350	.0365				.374	.382	.278
15000	.0306	X	.0306	.0420	.0387	.0403				.417	.428	.422
16000	.0333	X	.0333	.0482	X	.0482				.460	X	.460
17000												

TABLE No. 14  
BEAM H1-B

Load in lbs.	Beam Apparatus						Deflection inches		
	Upper			Lower					
	East	West	Av.	East	West	Av.	East	West	Av.
1650	0	0	0	0	0	0	0	0	0
3000	.0026	.0028	.0027	.0014	.0008	.0011	.020	.022	.021
4000	.0052	.0052	.0052	.0025	.0015	.0019	.038	.042	.039
5000	.0086	.0082	.0084	.0038	.0028	.0033	.062	.064	.063
6000	.0120	.0112	.0116	.0057	.0052	.0055	.090	.094	.092
7000	.0164	.0150	.0157	.0120	.0104	.0112	.131	.135	.133
8000	.0213	.0755	.0484	.0163	.0160	.0161	.176	.180	.178
9000	.0246	.0786	.0516	.0205	.0205	.0205	.210	.218	.218
10000	.0289	.0815	.0552	.0245	.0252	.0248	.264	.257	.260
11000	.0316	.0838	.0557	.0280	.0294	.0287	.302	.294	.295
12000	.0350	.0867	.0608	.0317	.0337	.0327	.341	.332	.336
13000	.0393	.0903	.0648	.0300	.0387	.0343	.411	.395	.403

Beam H1-C is hardly a fair test in that apparently through some drying shrinkage in the form or other unknown cause, it was cracked vertically near the north support. It was also cracked vertically near the South loading point by being supported at this point in moving it from the room in which it was built. This latter, however, did not seem to affect the strength as the failure was at the other end. Fig. 37 shows the type of cracking that took place and Table No. 15 the data secured. Probably a slightly increased amount of stirrup reinforcement and also a rod bent up to the top would have greatly strengthened it. This beam was built on March 13 and

TABLE No. 15

Load in lbs.	Beam Apparatus						Deflection inches		
	Upper			Lower			East	West	Av.
	East	West	Av.	East	West	Av.			
1600	0	0	0	0	0	0	0	0	0
3000	.0027	.0030	.0029	.0014	.0020	.0017	.027	.024	.025
4000	.0053	.0060	.0056	.0030	.0037	.0032	.053	.051	.052
5000	.0085	.0090	.0087	.0046	.0056	.0051	.088	.087	.087
6000	.0115	.0120	.0117	.0067	.0087	.0077	.123	.122	.122
7000	.0145	.0160	.0152	.0124	.0145	.0135	.167	.165	.166
8000	.0166	.0186	.0176	.0170	.0173	.0171	.205	.198	.202
9000	.0195	.0213	.0204	.0207	.0208	.0207	.247	.235	.241
10000	.0220	.0234	.0227	.0242	.0246	.0244	.287	.274	.285
11000	.0250	.0260	.0255	.0276	.0280	.0278	.325	.307	.316
12000	-----	-----	-----	-----	-----	-----	.487	-----	.487

TABLE No. 16  
BEAM H1-C

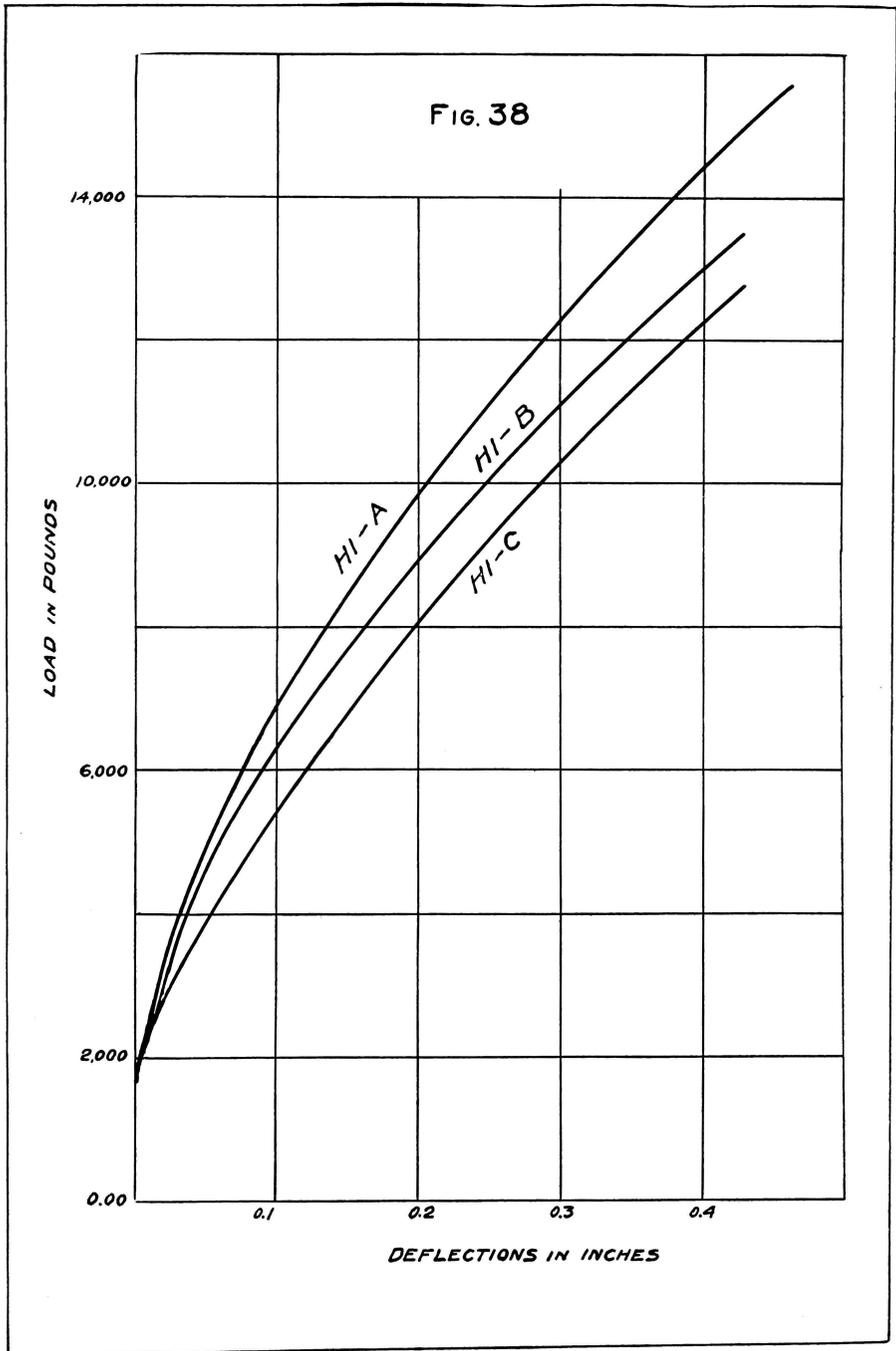
Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge			East	West	Av.
	East	West	Av.	East	West	Av.	East	West	Av.			
1645	0	0	0	0	0	0	0	0	0	0	0	0
3000	0	.0021	.001	.0030	.0030	.0030	.0006	.0008	.0007	.019	.025	.022
4000	.0015	.0037	.0026	.0054	.0045	.0049	.0010	.0014	.0012	.037	.045	.041
5000	.0025	.0050	.0037	.0073	.0064	.0068	.0012	.0020	.0016	.052	.062	.057
6000	.0070	.0075	.0072	.0110	.0096	.0103	.0020	.0030	.0025	.078	.089	.083
7000	.0095	.0100	.0097	.0146	.0130	.0138	.0030	.0036	.0033	.106	.119	.112
8000	.0074	.0125	.0099	.0180	.0163	.0172	.0040	.0046	.0043	.139	.149	.144
9000	.0090	.0137	.0113	.0215	.0192	.0203	.0048	.0054	.0051	.173	.176	.174
10000	.0110	.0170	.0140	.0250	.0224	.0237	.0056	.0060	.0055	.211	.202	.206
11000	.0127	.0194	.0160	.0284	.0260	.0272	.0064	.0068	.0066	.254	.235	.244
12000	.0145	.0220	.0182	.0317	.0290	.0303	.0072	.0076	.0074	.276	.264	.270
1645	.0009	.0013	.0002	.0050	.0040	.0045	.0006	.0006	.0006	.019	.026	.022
13000	.0162	.0260	.0211	.0380	.0337	.0358	.0096	.0090	.0093	.300	.302	.301
13500	.0207	.0377	.0292	.0753	.0653	.0703	.0204	.0152	.0178	.319	.462	.390

tested on May 4. Fig. 38 shows the deflections for all three beams of the H1 series plotted against the loads and an average curve drawn for the three.

The H2 series of beams had exactly the same material in them as the H1 series but it was disposed differently. Instead of having two courses above the hollow and one below, the hollow part was directly on the concrete and there were three courses solid above the hollow which put the part above the neutral axis practically solid. Fig. 39 shows the cracks formed during the failure which was apparently due primarily to stretch of the longitudinal steel primarily and secondarily to diagonal tension. The diagonal tension failure did not show up until the yield point of the steel had been exceeded slightly—at about 13,500 lbs. load. Note that two brick are broken across. Table No. 16 gives the data for this beam which was built March 8 and tested April 21.

TABLE NO. 17  
BEAM H2-A

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1655	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0006	.0014	.0010	.0018	.0018	.0018	.0000	.00000		.017	.018	.017
4000	.0046	.0028	.0037	.0046	.0050	.0048	.0002	.0010	.0006	.036	.039	.037
5000	.0084	.0037	.0060	.0068	.0085	.0076	.0004	.0016	.0010	.053	.058	.055
6000	.0124	.0044	.0084	.0110	.0145	.0127	.0008	.0022	.0015	.086	.085	.085
7000	.0134	.0053	.0093	.0150	.0180	.0165	.0016	.1128	.0022	.102	.119	.110
8000	.0150	.0065	.0107	.0190	.0223	.0206	.0022	.0034	.0028	.142	.143	.142
9000	.0170	.0080	.0125	.0220	.0260	.0240	.0030	.0042	.0036	.172	.170	.171
10000	.0190	.0095	.0142	.0250	.0295	.0272	.0040	.0054	.0042	.205	.198	.201
11000	.0201	.0117	.0163	.0285	.0280	.0280	.0044	.0066	.0055	.240	.232	.236
12000	.0225	.0132	.0178	.0317	.0356	.0341	.0048	.0066	.0057	.270	.264	.267
13000	.0237	.0142	.0189	.0352	.0400	.0316	.0056	.0076	.0066	.303	.299	.301
14000	.0254	.0150	.0202	.0380	.0440	.0410	.0064	.0088	.0076	.333	.330	.331
15000	.0263	.0160	.0212	.0400	.0048	.0440	.0070	.0098	.0094	.376	.379	.377



While H2-A failed primarily in tension in longitudinal steel, the diagonal tension failure is the most prominent feature in the picture. Fig. 40 shows quite definitely that failure in H2-B was from tension in the longitudinal steel. There were very fine diagonal tension cracks in the mortar joints of the north end, indicating that possibly the diagonal tension failures were partly the result of incipient failure in the

TABLE No. 18  
BEAM H2-B

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1755	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0013	.0053	.0083	.0010	.0140	.0075	.0000	.0000	.0000	.016	.019	.018
4000	.0122	.0039	.0080	.0032	.0158	.0145	.0018	.0000	.0009	.032	.033	.032
5000	.0134	.0025	.0079	.0054	.0180	.0117	.0022	.0002	.0012	.049	.051	.050
6000	.0143	.0037	.0090	.0081	.0203	.0142	.0030	.0006	.0018	.069	.083	.076
7000	.0153	.0053	.0103	.0111	.0226	.0168	.0036	.0010	.0023	.095	.101	.099
8000	.0164	.0065	.0115	.0141	.0255	.0198	.0037	.0014	.0025	.122	.131	.126
9000	.0174	.0082	.0128	.0174	.0283	.0228	.0044	.0018	.0031	.151	.175	.163
10000	.0184	.0095	.0139	.0218	.0310	.0264	.0050	.0022	.0038	.176	.196	.186
11000	.0195	.0112	.0153	.0255	.0337	.0296	.0056	.0026	.0041	.195	.233	.214
12000	.0206	.0130	.0168	.0288	.0365	.0326	.0070	.0036	.0053	.198	.264	.231
13000	.0216	.0145	.0180	.0340	.0396	.0368	.0072	.0040	.0056	.245	.301	.273

TABLE No. 19  
Beam H3-A

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1520	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0016	.0020	.0018	.0025	.0023	.0024	.0008	.0006	.0007	.019	.028	.023
4000	.0027	.0035	.0031	.0046	.0042	.0044	.0015	.0014	.0014	.035	.046	.040
5000	.0043	.0055	.0049	.0084	.0076	.0080	.0022	.0020	.0021	.061	.073	.067
6000	.0060	.0076	.0078	.0128	.0115	.0121	.0030	.0026	.0028	.091	.104	.097
7000	.0076	.0100	.0088	.0167	.0153	.0155	.0038	.0034	.0036	.122	.133	.129
8000	.0093	.0116	.0104	.0200	.0182	.0191	.0046	.0040	.0043	.150	.167	.158
9000	.0107	.0133	.0120	.0230	.0213	.0221	.0052	.0048	.0050	.181	.200	.190
10000	.0126	.0152	.0139	.0267	.0246	.0256	.0060	.0054	.0057	.215	.230	.222
11000	.0142	.0170	.0156	.0300	.0276	.0288	.0067	.0062	.0065	.246	.261	.253
12000	.0160	.0192	.0176	.0336	.0312	.0324	.0075	.0070	.0072	.285	.300	.292
13000	.0178	.0210	.0194	.0370	.0344	.0357	.0082	.0076	.0079	.336	.324	.330
14000	.0197	.0230	.0263	.0407	.0378	.0392	.0090	.0086	.0088	.385	.387	.386

longitudinal steel, especially since they did not appear in either of these two or in H2-C until near the yield point of the steel had been reached. Table No. 17 gives the detailed data on this beam made on March 8 and 9 and tested on April 21. It was started in the afternoon late and some courses laid but not finished until the next morning. Differing from concrete practice, whole courses were laid rather than breaking off in the middle and laying up the whole of one end. In spite of this it was the best of the H2 series.



Fig. 39

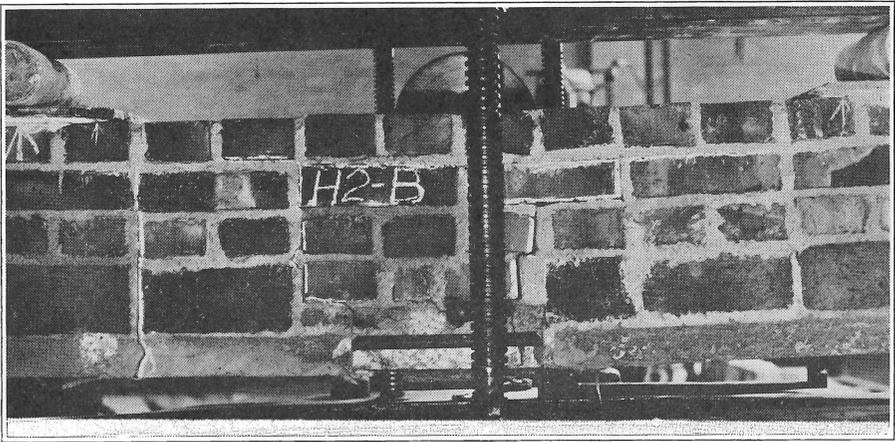


Fig. 40

Fig. 41 again shows a failure in diagonal tension very similar to that of H2-A and other beams but again not until near the yield point of the longitudinal steel. This was the beam which was dropped about one foot in loading it onto the truck at which time the crack over the arrow point was made. Apparently this crack did not affect the strength of the beam until near 13,000 lbs. load when it began to open slightly. Just as the load reached 14,000 but before readings could be taken, slipping occurred longitudinally and the beam failed. Had there been one longitudinal bar bent up to take the stress that opened the vertical joint, perhaps the load carried would have been increased. Table No. 18 gives the detailed data on this test, and Fig. 42 gives the steel deformation in the three beams separately. H2-C was made on

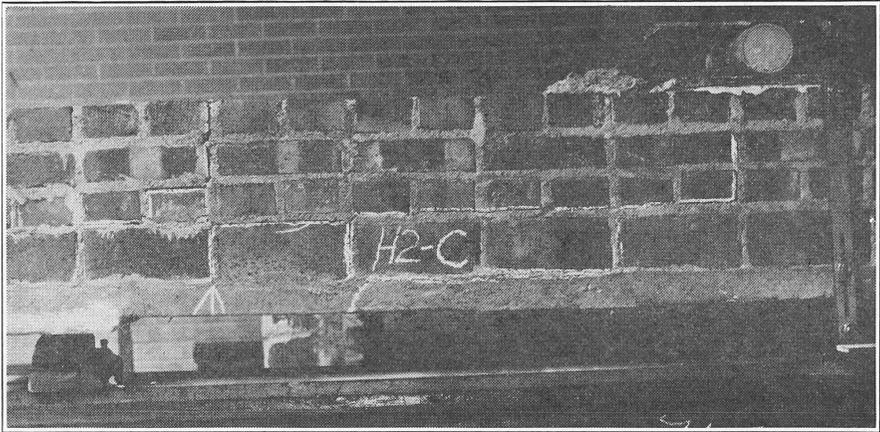


Fig. 41

TABLE No. 20  
BEAM H3-B

Load in lbs-	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge			East	West	Av.
	East	West	Av.	East	West	Av.	East	West	Av.			
1510	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0033	.0038	.0035	.0024	.0025	.0024	.0008	.0008	.0008	.027	.028	.027
4000	.0066	.0076	.0071	.0060	.0057	.0058	.0016	.0016	.0016	.054	.057	.055
5000	.0112	.0116	.0114	.0096	.0112	.0104	.0020	.0024	.0022	.090	.093	.091
6000	.0146	.0146	.0146	.0134	.0150	.0142	.0026	.0033	.0029	.124	.128	.126
7000	.0180	.0180	.0180	.0173	.0182	.0177	.0036	.0042	.0039	.157	.163	.160
8000	.0216	.0213	.0214	.0212	.0224	.0218	.0044	.0050	.0047	.198	.201	.200
9000	.0244	.0234	.0239	.0236	.0252	.0244	.0050	.0056	.0053	.229	.230	.229
10000	.0273	.0260	.0266	.0267	.0287	.0277	.0056	.0064	.0060	.265	.264	.264
11000	.0305	.0287	.0296	.0300	.0325	.0311	.0064	.0072	.0068	.301	.300	.300
12000	.0330	.0312	.0321	.0330	.0355	.0342	.0072	.0080	.0076	.336	.328	.332
13000	.0360	.0332	.0346	.0360	.0380	.0370	.0078	.0086	.0082	.372	.360	.366
14000	.0390	.0354	.0372	.0397	.0460	.0428	.0086	.0130	.0097	.427	.405	.416

March 9 and tested April 20.

The H3 series was a continuation of the efforts to lessen the dead weight. Just over the concrete, brick cut to length with a mason's chisel were set on end as shown leaving a hollow between them. Two courses of headers were laid on them to complete the beam with an effective depth of twelve inches. Fig. 43 shows the general arrangement of material in this series as well as the failure in H3-A. This series was remarkably consistent in results as measured by load carried, deflection and location of the neutral axis. It seems likely that with minor changes in the steel sections and in placing, this design would consistently carry practically the same loads as a similar sized solid beam. The final failure in H3-A was by diagonal tension but at practically the elastic limit of the longitudinal steel. Table No. 19 shows the detail data for this beam which was made March 9 and tested May 4.

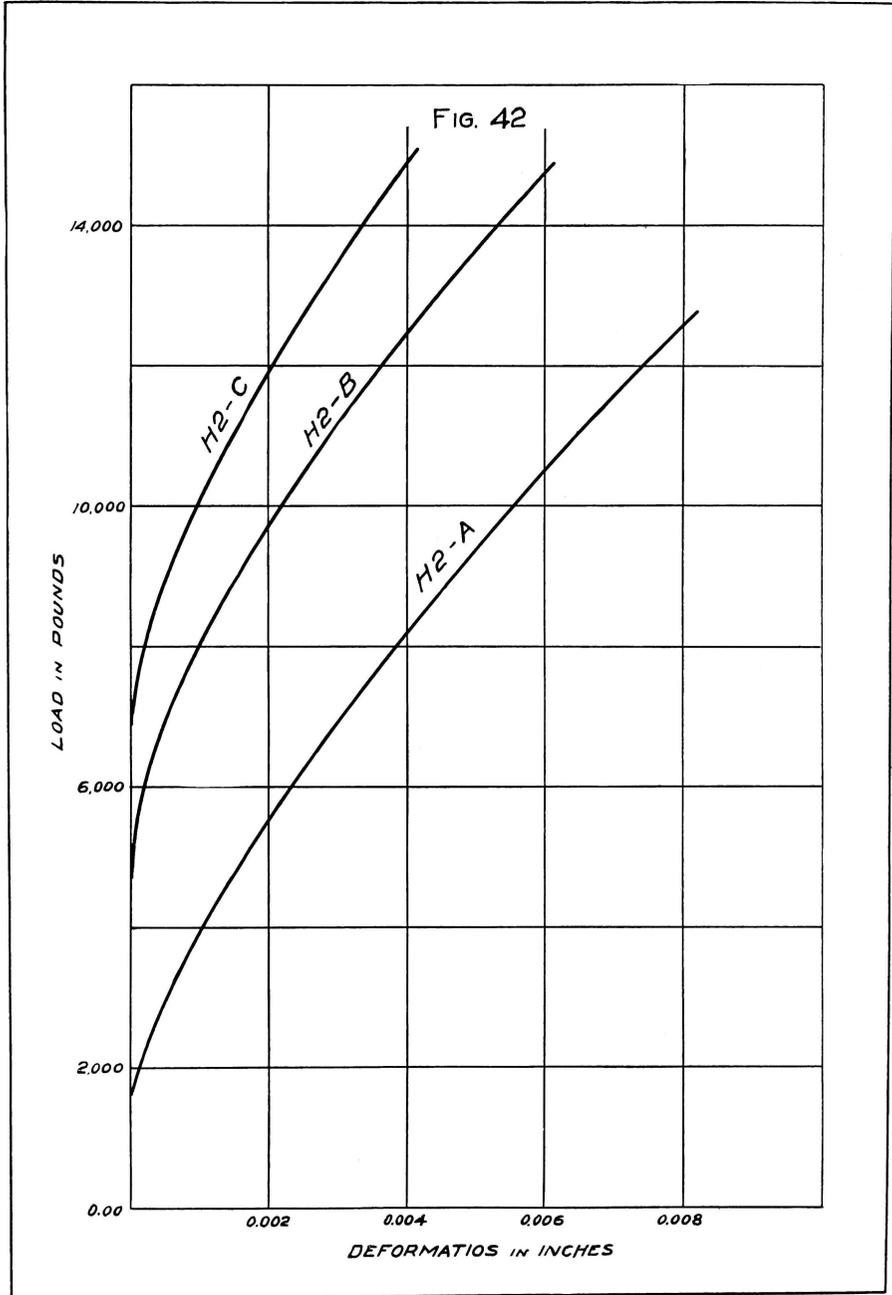


TABLE No. 21  
BEAM H3-C

Load in lbs-	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1490	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0016	.0026	.0021	.0020	.0021	.0020	0	0	0	.021	.021	.021
4000	.0032	.0046	.0039	.0046	.0037	.0041		.0008	.0008	.045	.046	.045
5000	.0050	.0072	.0061	.0085	.0065	.0075		.0014	.0014	.074	.076	.075
6000	.0070	.0095	.0087	.0124	.0100	.0112		.0022	.0022	.105	.110	.107
7000	.0090	.0117	.0103	.0162	.0135	.0148		.0030	.0030	.137	.142	.139
8000	.0114	.0137	.0125	.0200	.0173	.0186		.0038	.0038	.172	.176	.174
9000	.0135	.0157	.0146	.0236	.0213	.0224		.0046	.0046	.210	.212	.211
10000	.0162	.0180	.0171	.0273	.0252	.0262		.0054	.0054	.249	.246	.247
11000	.0183	.0197	.0190	.0204	.0287	.0295		.0060	.0060	.284	.278	.281
12000	.0206	.0220	.0213	.0340	.0325	.0332		.0068	.0068	.322	.318	.320
13000	.0227	.0240	.0233	.0375	.0360	.0367		.0076	.0076	.348	.354	.351
14000	.0255	.0263	.0259	.0416	.0396	.0406		.0084	.0084	.397	.401	.399
15000	.0263	.0324	.0293	.0553	.0506	.0529		.0188	.0188	.473	.467	.470

H3-B, shown in Fig. 44, first showed definite failure at the center by tension in the longitudinal steel excessive deformation in the steel starting at about 14,600 lbs. The point marked OF in the picture indicates the first definite crack and the point marked SF indicates the secondary failure by a longitudinal crack which

TABLE No. 22  
BEAM H4-C

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1670	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0023	.0015	.0019	.0010	.0004	.0007	.0004	.0004	.0004	.018	.018	.018
4000	.0052	.0044	.0048	.0018	.0012	.0015	.0008	.0006	.0007	.032	.033	.032
5000	.0080	.0067	.0073	.0037	.0037	.0037	.0019	.0012	.0015	.051	.052	.051
6000	.0104	.0092	.0098	.0065	.0052	.0028	.0028	.0020	.0024	.072	.078	.076
7000	.0126	.0113	.0119	.0093	.0080	.0086	.0036	.0026	.0031	.097	.105	.101
8000	.0153	.0130	.0141	.0125	.0115	.0120	.0044	.0032	.0038	.125	.136	.130
9000	.0174	.0146	.0160	.0155	.0148	.0156	.0050	.0040	.0045	.152	.165	.158
10000	.0195	.0162	.0178	.0184	.0180	.0182	.0057	.0047	.0052	.181	.195	.188
1670	.0073	.0064	.0068	.0028	.0011	.0019	.0012	.0001	.0006	.041	.041	.041
3000	.0100	.0087	.0093	.0048	.0030	.0039	.0020	.0006	.0013	.063	.066	.064
4000	.0114	.0102	.0108	.0067	.0048	.0057	.0025	.0011	.0018	.081	.083	.082
5000	.0130	.0115	.0125	.0089	.0072	.0080	.0032	.0016	.0024	.099	.102	.101
6000	.0150	.0130	.0140	.0113	.0098	.0105	.0038	.0021	.0029	.118	.123	.120
7000	.0166	.0140	.0153	.0132	.0126	.0126	.0044	.0037	.0040	.136	.142	.139
8000	.0182	.0152	.0167	.0154	.0146	.0150	.0050	.0034	.0042	.156	.163	.159
9000	.0196	.0167	.0181	.0174	.0170	.0172	.0056	.0040	.0048	.175	.184	.179
10000	.0210	.0173	.0191	.0193	.0195	.0194	.0063	.0048	.0055	.195	.205	.200
11000	.0230	.0184	.0207	.0215	.0220	.0217	.0069	.0054	.0061	.218	.229	.223
12000	.0250	.0200	.0225	.0243	.0250	.0246	.0074	.0060	.0067	.261	.274	.267
13500	.0276	.0223	.0249	.0284	.0297	.0280	.0082	.0070	.0076	.314	.334	.324
14000	.0283	.0230	.0256	.0297	.0310	.0303	.0085	.0072	.0078	.327	.340	.333
15000	.0302	.0246	.0274	.0320	.0343	.0331	.0090	.0078	.0084	.383	.389	.386
16000	.0327	.0265	.0286	.0345	.0376	.0355	.0096	.0084	.0090	.432	.436	.434
14700	.0332	.0270	.0301	.0283	.0417	.0350	.0093	.0080	.0086	.487	.490	.489

finally goes across a brick. This secondary failure did not become apparent until some time after the elastic limit of the steel was passed. The detail data is shown in Table No. 20. This beam was made March 9 and tested on April 28.

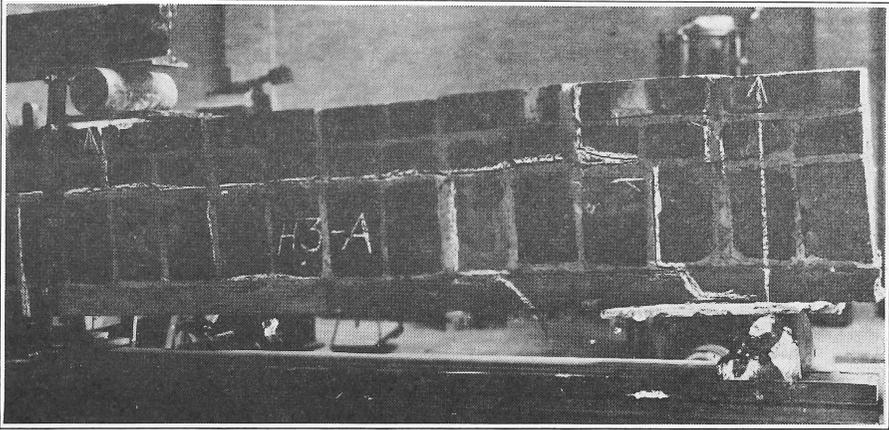


Fig. 43

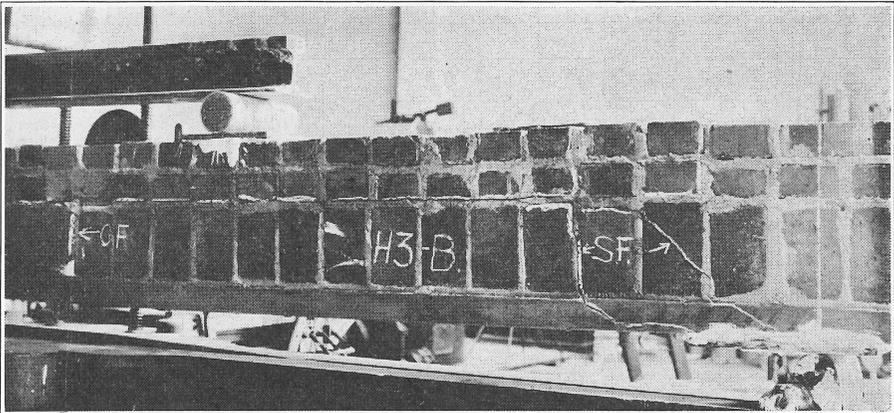


Fig. 44

H3-C shows practically the same sort of failure in Fig. 45, except that the secondary failure due to diagonal tension scarcely appeared at all. Apparently the cracks shown at the point marked x, while outside the middle third where tension cracks usually are found under third point loading, were probably due to stretch in the longitudinal steel rather than diagonal tension. Fig. 46 shows the deflections of this series plotted against loads. H3-C was made on March 9 and 10 and tested on April 27.

The H4 series again gave very consistent results, two of the series failing above 16,000 lbs. and the third above 14,000 lbs. H4-A was made with two courses of brick on edge forming the hollow part as shown in Fig. 47 which also shows the failure,

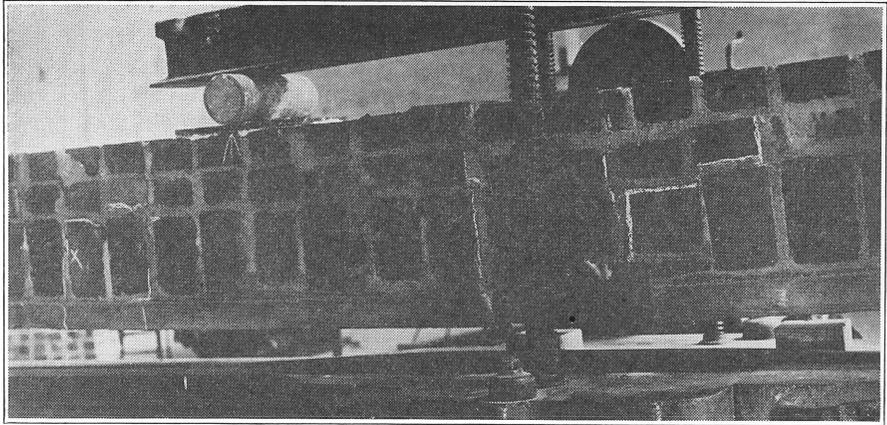


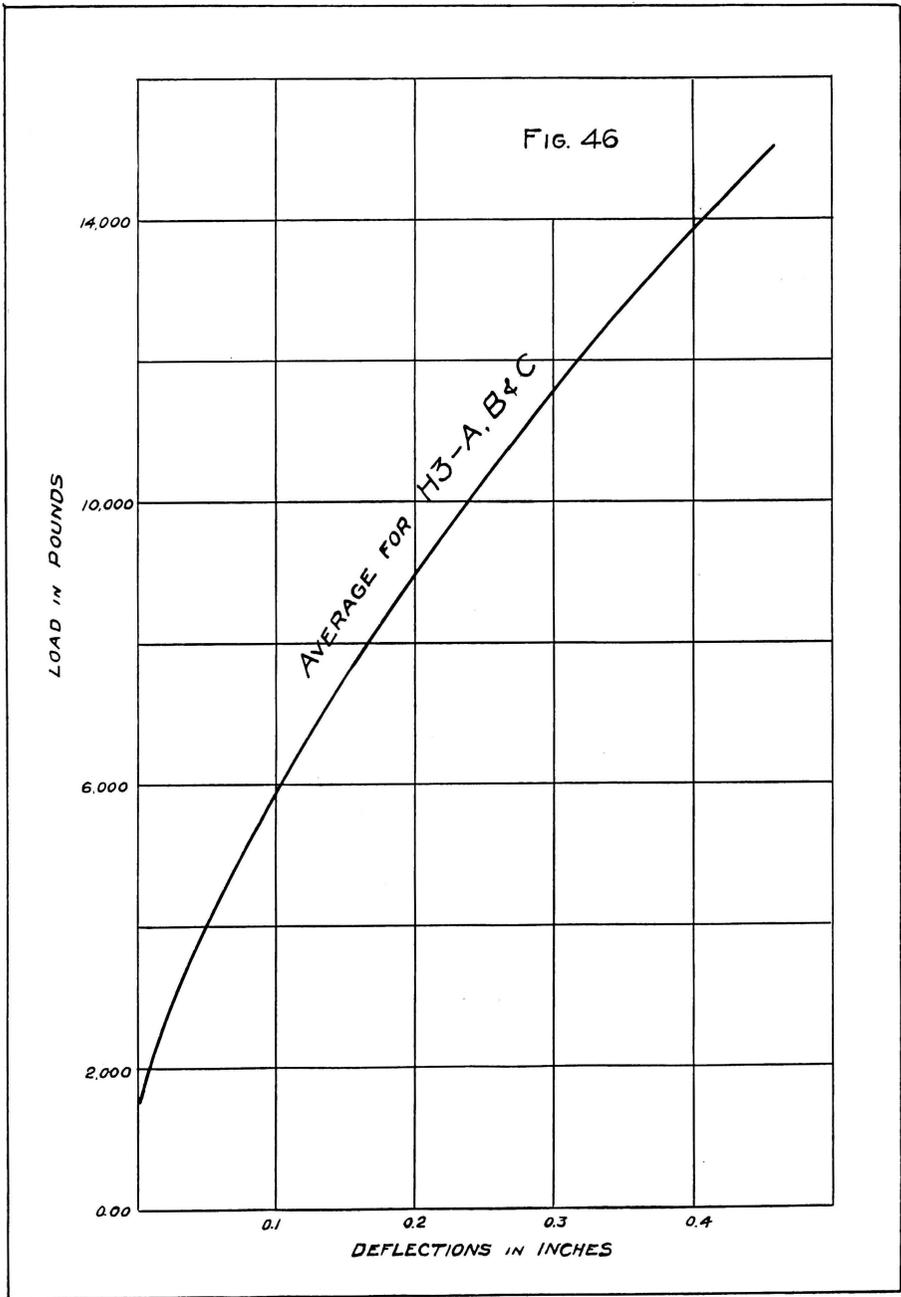
Fig. 45

TABLE No. 23  
BEAM H4-B

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1650	0	0	0	0	0	0	0	0	0	0	0	0
3000	.0005	.0006	.0005	.0015	.0014	.0014	.0000	.0000	.0000	.013	.013	.013
4000	.0014	.0017	.0015	.0035	.0033	.0034	.0004	.0003	.0001	.028	.029	.028
5000	.0028	.0035	.0031	.0065	.0060	.0062	.0005	.0005	.0005	.046	.048	.047
6000	.0048	.0062	.0055	.0102	.0094	.0096	.0018	.0016	.0017	.071	.074	.073
7000	.0063	.0080	.0071	.0135	.0122	.0128	.0024	.0022	.0023	.095	.097	.096
8000	.0077	.0095	.0086	.0167	.0152	.0159	.0032	.0028	.0030	.121	.122	.121
9000	.0092	.0113	.0102	.0197	.0182	.0189	.0029	.0036	.0032	.148	.148	.148
10000	.0104	.0130	.0017	.0230	.0217	.0223	.0046	.0044	.0045	.175	.171	.173
11000	.0117	.0147	.0132	.0263	.0242	.0252	.0054	.0050	.0052	.201	.198	.200
12000	.0132		.0152	.0300	.0275	.0287	.0062	.0054	.0058	.232	.232	.232
13000	.0144		.0144	.0330	.0300	.0315	.0068	.0060	.0064	.261		.261
14350	.0170		.0170	.0402	.0387	.0394		.0090	.0090	.464	.482	.473

while H4-B and H4-C had the corresponding part of the beam made by standing the brick on end. The variation in construction was made to determine whether there was any material difference in the difficulty of construction or not. Rather to our surprise, we found no difficulty in constructing either type. The failure in H4-A is typical of all three. Apparently at practically the yield point of the steel, stresses set up by excessive bending resulted in forcing the end of the brick apart off the hooks at the end of the longitudinal steel. This beam was loaded three times to 10,000 lbs. and the third time loading was continued to failure. The detailed data for the first and last loadings are given in Table No. 22.

Fig. 48 and Table No. 23 give the essential information on beam H4-B. There was nothing strikingly different about the test to need noticing. Also the same information for H4-C is given in Fig. 49 and Table No. 24. Fig. 50 gives the same curves as given for the other series. H4-A was made on March 10 and the other two were made on March 12. B and C were tested on April 26 and A was tested on April 27.



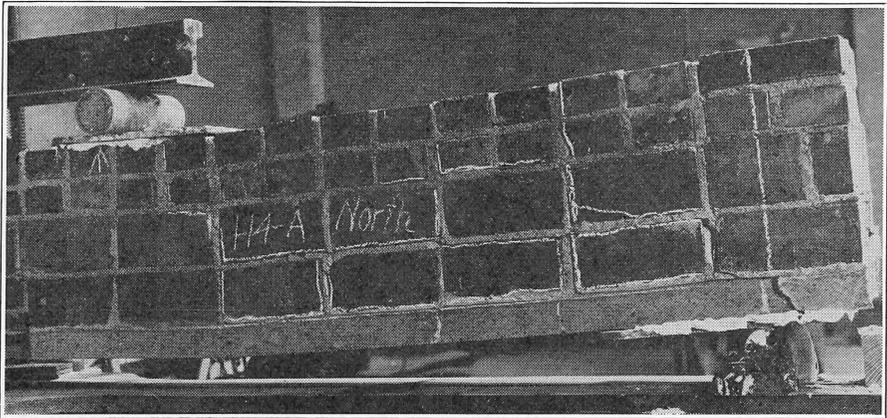


Fig. 47

TABLE No. 24  
BEAM H4-C

Load in lbs.	Beam Apparatus						Steel			Deflection inches		
	Upper			Lower			8" Gauge					
	East	West	Av.	East	West	Av.	East	West	Av.	East	West	Av.
1625	0	0	0	0	0	0	100	.000	.000	0	0	0
3000	.0023	.0017	.0020	.0016	.0021	.0018	.0006	.0004	.0005	.019	.021	.020
4000	.0023	.0048	.0035	.0086	.0038	.0062	.0014	.0008	.0011	.039	.041	.040
5000	.0040	.0069	.0052	.0107	.0060	.0083	.0020	.0014	.0017	.056	.060	.058
6000	.0055	.0078	.0066	.0128	.0088	.0108	.0026	.0020	.0023	.073	.076	.074
7000	.0070	.0092	.0050	.0155	.0122	.0133	.0032	.0024	.0058	.092	.096	.094
8000	.0087	.0105	.0096	.0188	.0142	.0165	.0038	.0033	.0035	.116	.122	.119
9000	.0106	.0117	.0112	.0223	.0175	.0204	.0044	.0040	.0042	.141	.146	.143
10000	.0123	.0130	.0126	.0258	.0205	.0231	.0051	.0057	.0054	.166	.172	.169
11000	.0140	.0143	.0141	.0290	.0235	.0267	.0056	.0053	.0054	.171	.197	.184
12000	.0157	.0158	.0157	.0324	.0262	.0293	.0062	.0059	.0060	.181	.224	.202
13000	.0183	.0187	.0185	.0374	.0305	.0339	.0071	.0070	.0075	.241	.276	.258
14000	.0194	.0193	.0194	.0393	.0322	.0357	.0075	.0071	.0073	.252	.289	.220
15000	.0208	.0207	.0208	.0424	.0350	.0387	.0080	.0076	.0078	.278	.214	.246
16000	.0226	.0226	.0226	.0460	X	.0460	.0086	.0074	.0080	.373	X	.323

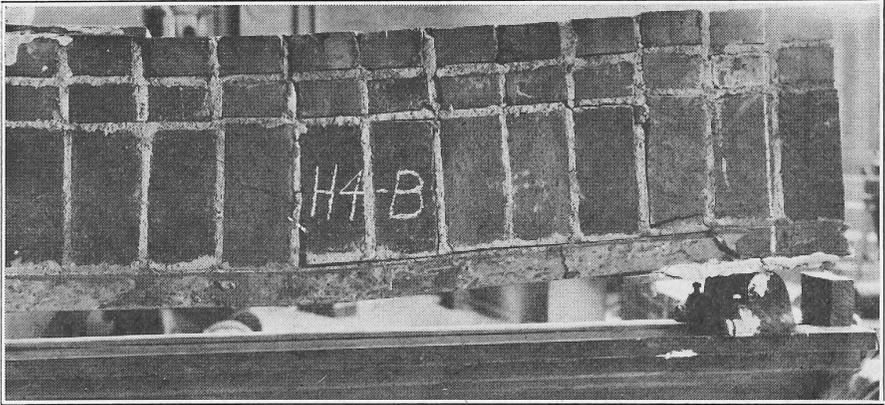


Fig. 48

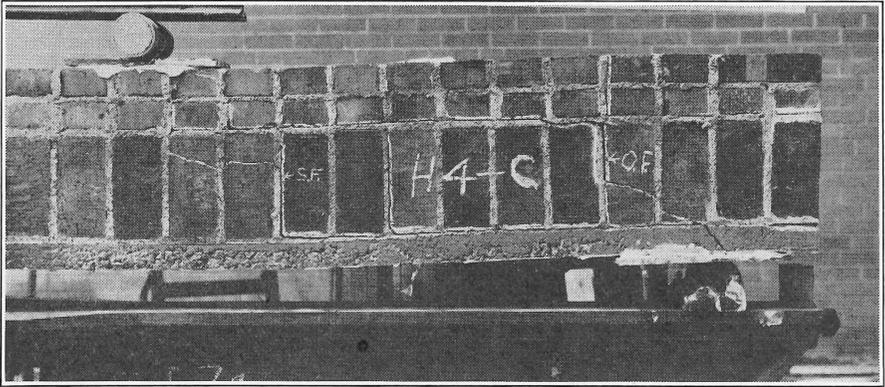
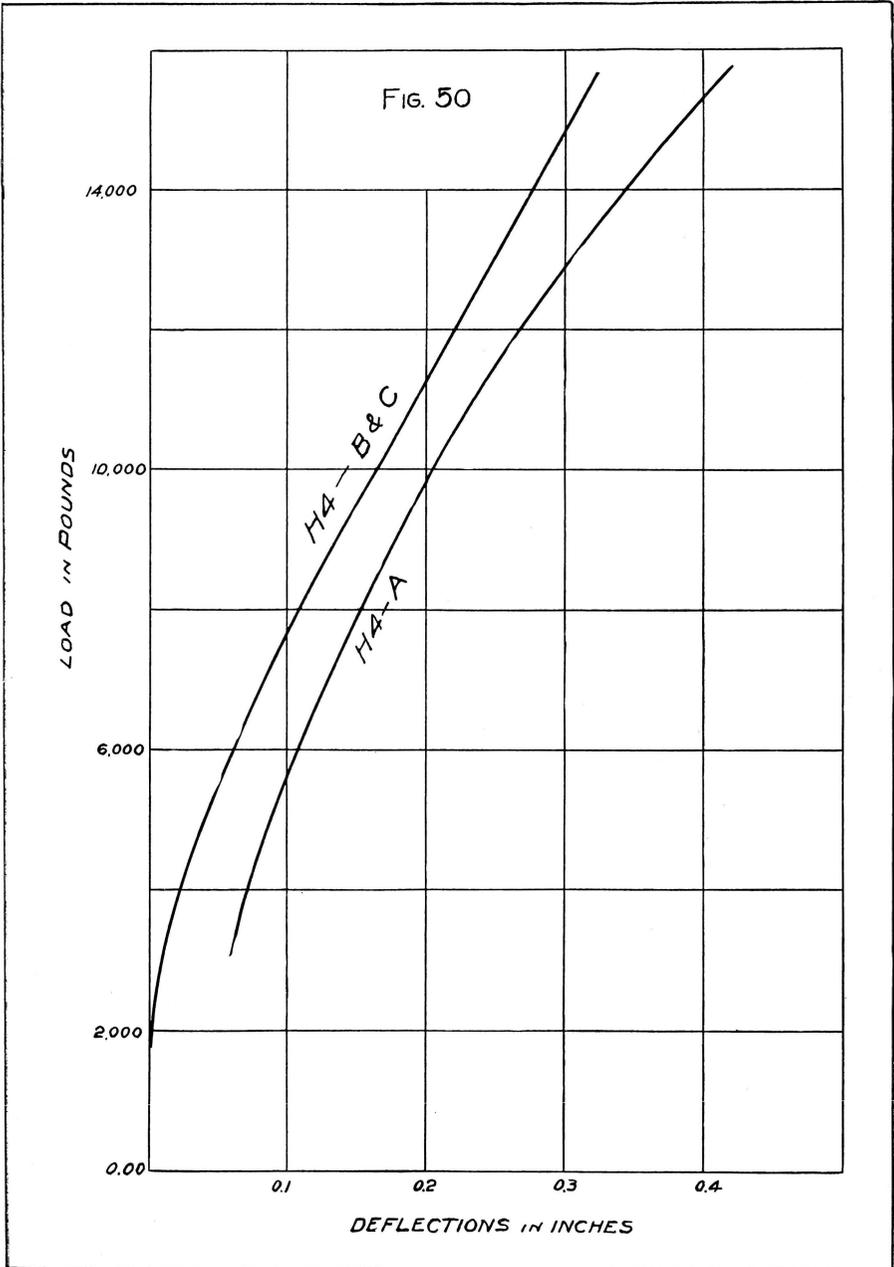


Fig. 49



## CHAPTER VI

### CONSTRUCTION AND TESTS OF SLABS

Early in this investigation, it was realized that the most obvious use of this type of construction would at least in the beginning be slabs for floors and roofs and therefore we desired to know something of the results to be expected when such slabs were actually in use. While beams offered certain facilities for making the desired measurements of deformations, it was considered desirable to build some slabs for test.

The only machine suitable for testing slabs or beams was the one used on the beams. Unfortunately, it would take only about twelve inches between the screws and, due to the type of construction, this did not seem to offer the proper conditions for representative tests. It was decided therefore, to build larger specimens and to test them in place by loading with sand, brick, etc. We appreciated that this was only an approximately accurate method of testing but all factors considered, it seemed the best available.

Since this could not wait for the results of the other tests for design data,  $n$  was assumed to be 15,  $f_s$  was taken as 16,000 lbs. per sq. in. and  $f_b$  as 700 lbs. per sq. in. The slightly higher stresses in the brick assumed for slabs over that assumed for the beams seemed to be justified by Brebner's results. Three designs were prepared as shown by Figs. 51 and 52. The first was to be built with the brick flat and with a quarter inch cement topping for finish and with a one-fourth inch rod in every longitudinal mortar joint. With plaster applied directly to the underside of the slab, this would weight about 30 lbs. per sq. ft. and should carry about 30 lbs. live load on spans up to eight feet maximum or proportionately larger loads on smaller spans with supporting beams. Fig. 53 shows a completed specimen before placing the topping and Fig. 54, the same design with the topping partly placed.

The second design called for the bricks to be placed on edge with one  $\frac{3}{8}$ -inch rod in every longitudinal mortar joint. This gave a spacing of the rods of about two and one-half inches, as compared with four inches for the one-fourth inch rods in the flat brick design. This was calculated to safely carry 30 lbs. per sq. ft. on spans of fourteen feet and up to 60 lbs. per sq. ft. on spans not greater than ten feet. These safe loads were selected from Hool and Whitney's "Concrete Designer's Manual."

Fig. No. 55 shows the manner of placing the rods in the mortar joints as the work progresses. Notice that a few brick are placed near the center to hold the rod in place and that mortar is spread along the rod to insure complete filling under and around the rod. The upper part of the joint was filled by slushing. Figure 56 shows the completed brickwork for two specimen slabs and Fig. 57 shows the topping partly placed and the ribs laid for the ribbed slab which was the third design.

The idea originally behind the third design was that the ceiling and slab would be placed as one unit. The ribs were to be laid on metal lath wired to the reinforcing bars so that the lath would carry the ceiling plaster and also contribute to the other reinforcing. At the time this slab was built, no lath was available so it was not used. The bars used were three-fourth inch round bars under the middle two ribs and three-eighths inch bars under the two outside ribs, these being used because they were available. Some little trouble was experienced in placing so thick a mortar layer around a single rod, the tendency being for the brick to slide one way or the other. It is believed, however, that the use of two smaller bars of the required combined area would remove this difficulty. Fig. 58, along with 52 previously referred to, shows the construction used. These slabs were built on March 13 and 14 and tested on April 30 and May 1, 2, 3.

FIG. 51

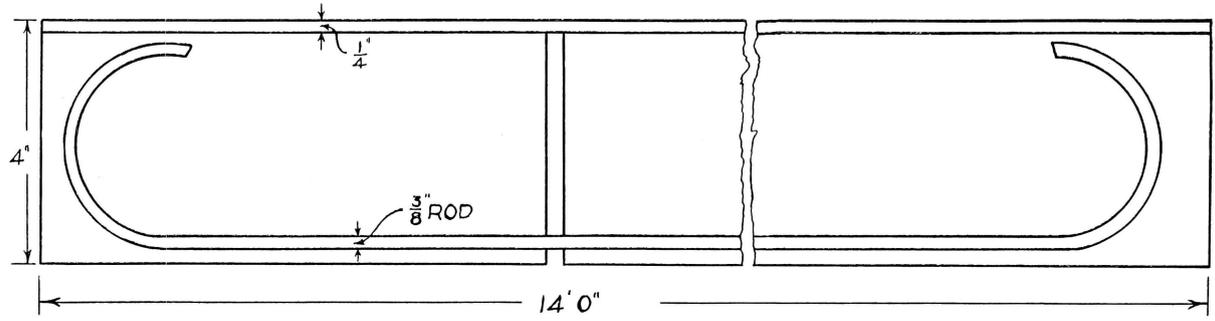
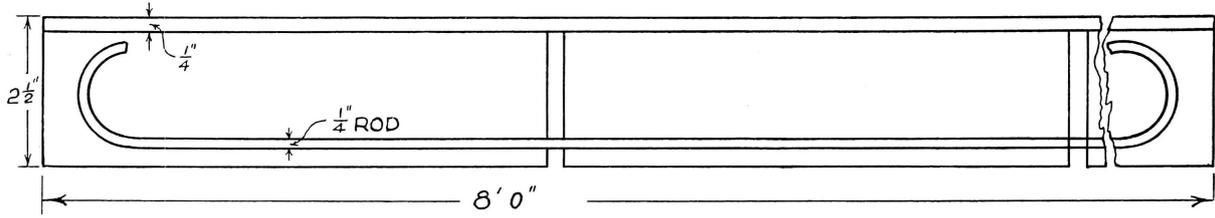
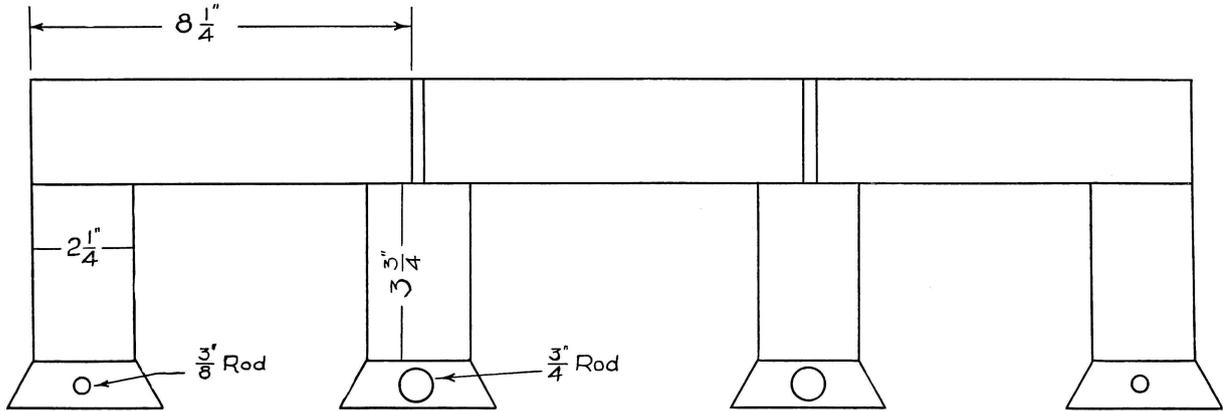


FIG. 52



RIBBED SLAB

Scale  $\frac{3}{8}$ " = 1"

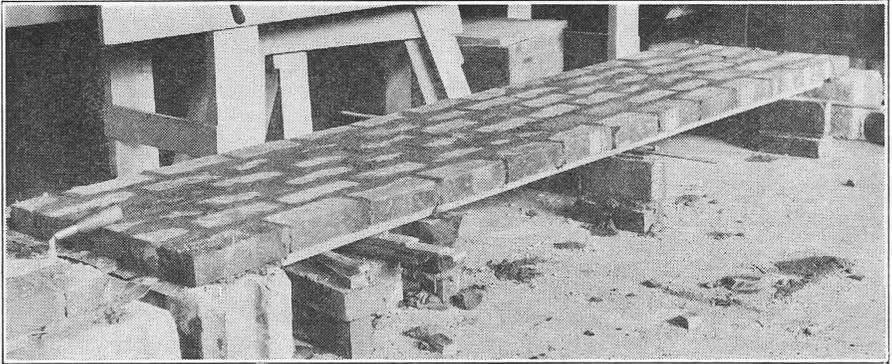


Fig. 53

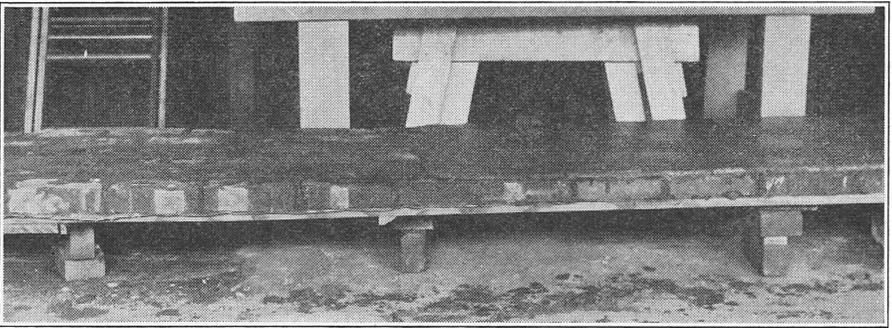


Fig. 54

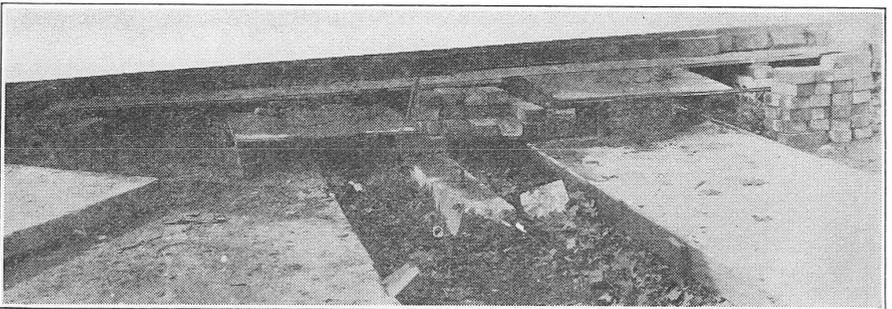


Fig. 55

Fig. 59 shows the method of loading slabs. The brick across the end, being over the supports and not carried by the slab directly, are not counted in the load. Their effect is really to cause slight fixing of the ends tending to stiffen the slabs slightly, but it is believed that their effect is negligible. In calculating the loads, an average value of the weight of sand per cubic foot is used to calculate the weight of sand. To this is added the weight of the brick between supports and this total is divided by the total area of slab between supports to get the load per sq. ft. Table No. 25 gives the data secured on this slab. All slabs of this series failed by tension in the steel at approximately the same load.

TABLE No. 25

Depth Sand inches	No. Brick	Load in pounds		Deflection inches		
		Total	Per sq. ft.	West	East	Av.
1	24	232.8	14.5	.021	.025	.023
2	24	326.4	20.4	.033	.031	.032
3	24	420.0	26.2	.030	.027	.029
4	48	652.8	40.7	.056	.052	.054
5	48	746.4	46.6	.069	.065	.067
6	72	979.2	61.2	.102	.096	.099
7	72	1072.8	67.0	.130	.113	.121
8	72	1166.4	72.9	.142	.127	.134
9	96	1409.2	88.0	.498	.553	.525

Fig. 60 shows the failure of flat brick slab No. 2 and Table No. 26 gives the corresponding data.

Fig. 61 shows the failure of flat brick slab No. 3, and Table No. 27 gives the data for it. This slab went down practically to the floor under the load but on removal of the load it, recovered somewhat and the three men shown in Fig. 62 were not sufficient to materially deflect it. It is unsupported except for the end supports in this picture and shows how even in cases of rather complete and extreme failure, these slabs do not collapse. Due to cramped space, the supports under this slab were not as good as

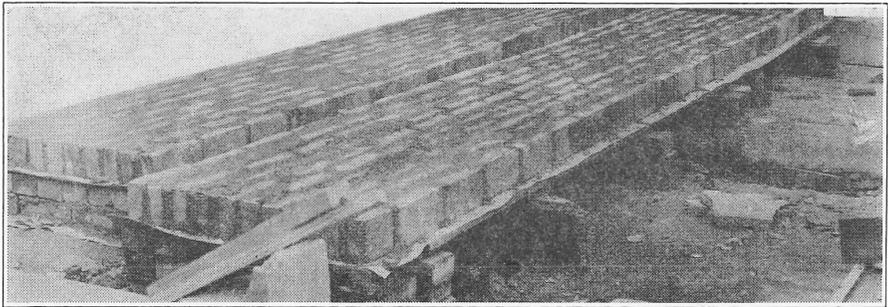


Fig. 56

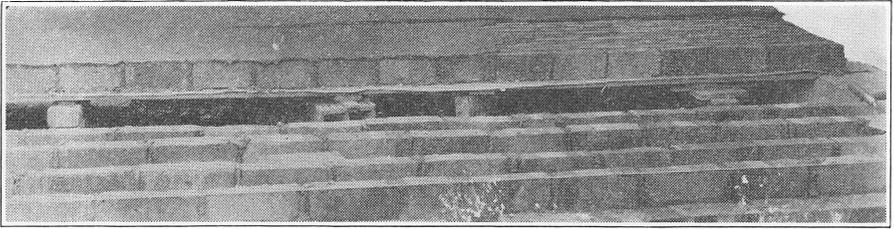


Fig. 57



Fig. 58

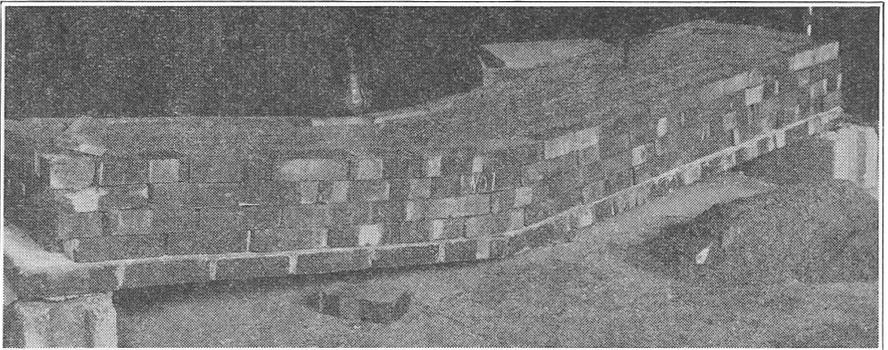


Fig. 59

under the other of the series. Fig. 63 shows the deflections of all three slabs of this series plotted against loads. With a factor of safety of three, this series of slabs would be safe for 25 to 30 lbs. per sq. ft. live load, for spans not over eight feet.

Brick on edge slab No. 1 was built inside (as were all three of the flat brick series) It was necessary to remove it to make room for other things before it could be tested to complete failure. It was loaded to the point shown by Table No. 28 and then a concentrated load of approximately 800 lbs was applied. As this exceeded three times the design load of 30 lbs. per sq. ft. live load, the test was discontinued and the slab removed.

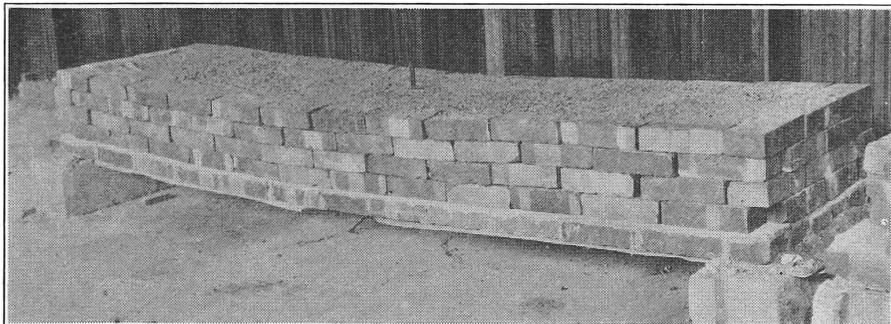


Fig. 60

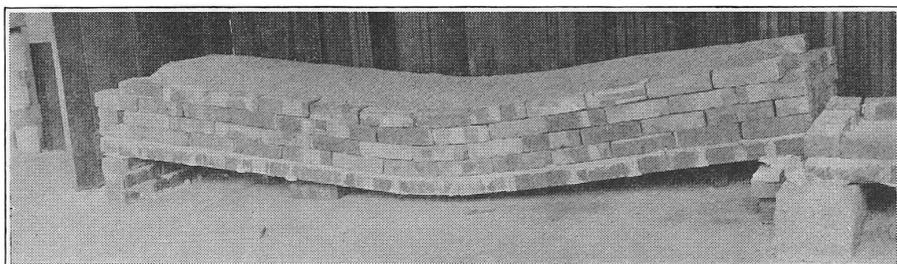


Fig. 61

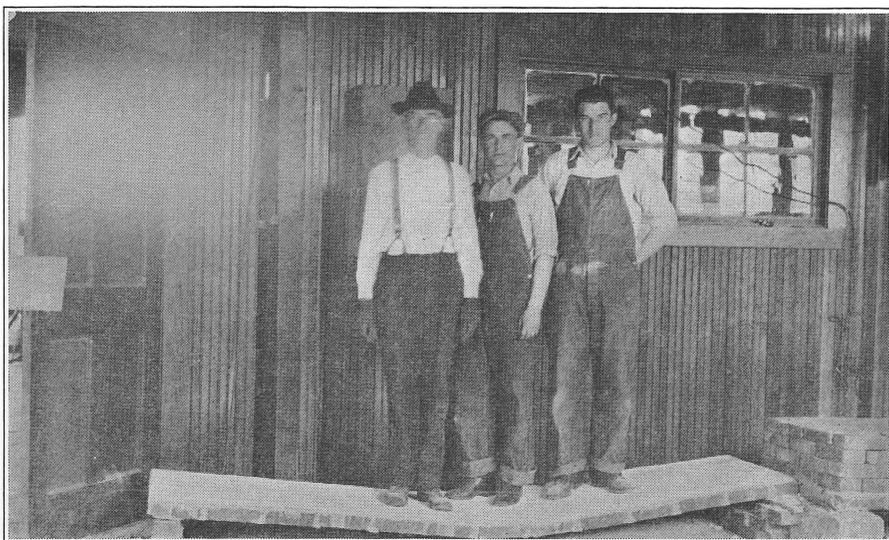


Fig. 62

TABLE No. 26

Depth Sand inches	No. Brick	Load in pounds		Deflection inches		
		Total	Per sq. ft.	West	East	Av.
1	23	237.0	14.8	.030	.028	.029
2	23	320.6	20.0	.037	.028	.032
3	46	557.6	34.8	.078	.048	.063
4	46	641.2	40.0	.093	.062	.077
5	46	737.8	46.1	.107	.075	.081
6	69	961.8	60.1	.153	.116	.134
7	69	1055.4	65.9	.171	.136	.153
8	92	1282.4	80.1	.300	.274	.287
8½	92	1376.0	86.0	.394	.383	.388
9	92	1376.0	86.0	.394	.383	.388
25 min. later			1	.520	.515	.518

Fig. 64 shows the second slab of this series. It was loaded as shown in Table 29 carrying roughly a load of 175 lbs. per sq. ft. with a deflection measured on one side only, of about 1.4 inches. As this was approximately six times the load originally

TABLE No. 27

Depth Sand inches	No. Brick	Load in pounds		Deflection inches		
		Total	Sq. Ft.	West	East	Av.
3	24	434.0	25.8	.063	.073	.068
3½	24	479.0	28.5	.076	.085	.081
4	48	671.5	39.9	.088	.097	.093
4½	48	717.6	42.7	.104	.115	.110
5	48	769.8	45.8	.112	.126	.119
5½	48	818.9	48.7	.125	.135	.130
6	72	1007.3	59.9	.149	.157	.153
6½	72	1057.4	62.9	.156	.163	.160
7	72	1105.6	65.8	.195	.191	.193
7½	72	1154.7	68.7	.208	.203	.206
8	72	1203.8	71.6	.230	.228	.229
8½	72	1252.0	74.5	.243	.237	.240
9	96	1441.0	85.9	.284	.266	.275

designed for and as the deflection was excessive, though the slab did not collapse, the loading was discontinued but the load was left in place. After a week, no appreciable increase in deflection had occurred. This indicates a safe load of about 60 lbs. per sq. ft. with a factor of safety of three. Note that at 74.7 lbs. per sq. ft. the deflection is only .464 inches while the allowable deflection ( $\frac{1}{360}$  of the span) is .466 inches showing a safe margin here. Failure was by tension in the steel.

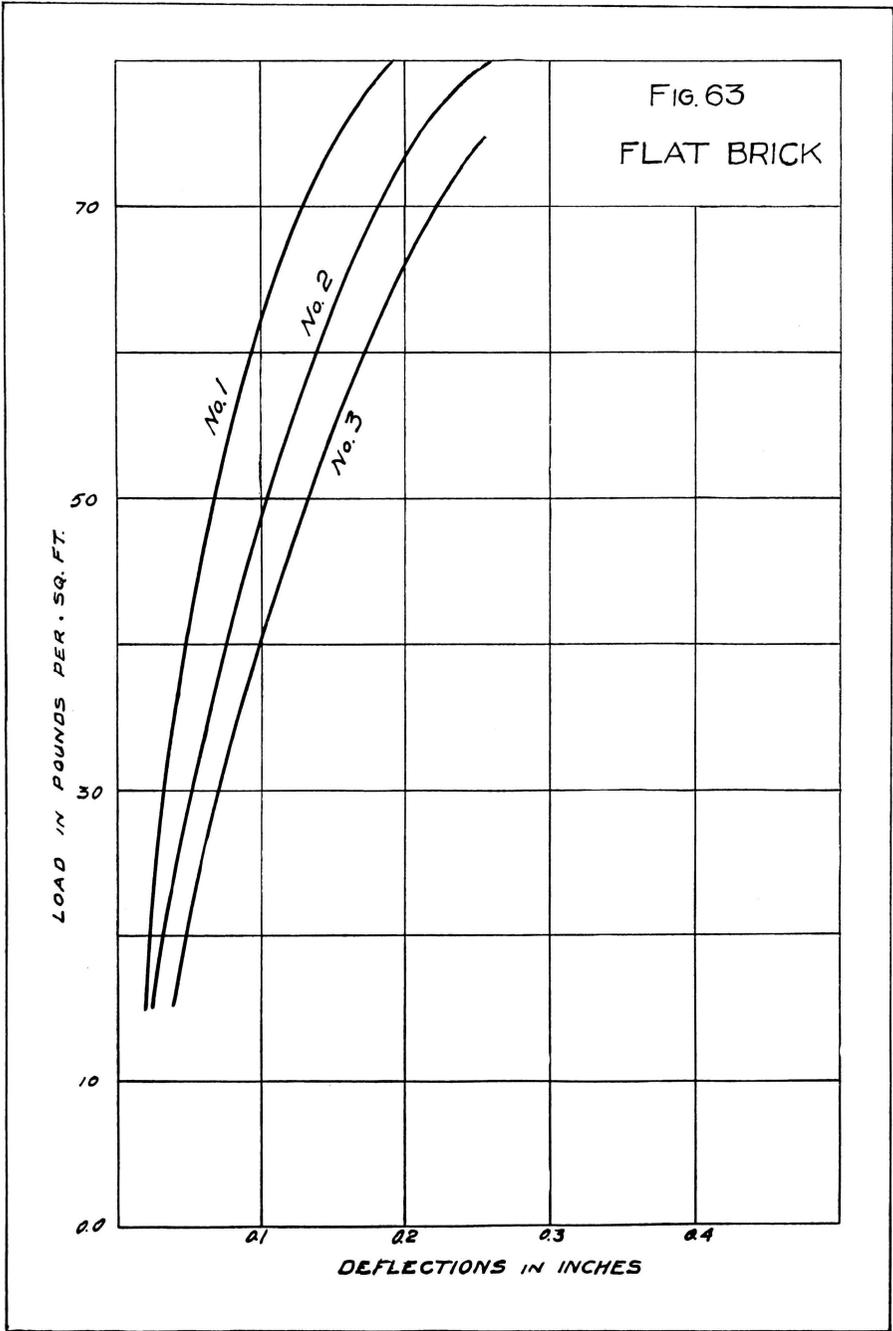


FIG. 63  
FLAT BRICK

The third slab of this series was tested in the same way but deflection readings were taken only for loads up to 118 lbs. per sq. ft. when the gauges would have had to be reset. As this could not be done accurately under the conditions prevailing and as this was well past the safe load point, the gauges were removed and loading continued until it reached 185 lbs. per sq. ft. The slab had not collapsed at this load,

TABLE No. 28

Depth Sand inches	No. Brick	Load in pounds		Deflection inches
		Per Sq. Ft.	Total	
1	39	12.4	412.62	.028
2	39	18	599.04	.040
3	78	30.4	1011.66	.080
4	78	36.03	1198.08	.085
5	117	48.4	1610.7	.130
6	117	54.02	1796.32	.148
7	117	59.65	1983.54	.182
8	117	65.5	2179.96	.197
8	156	72.3	2406.16	.282
8	156	after 30 minutes 72.3	2406.16	.321
8½	156	74.9	2489.37	.332
9	156	77.6	2582.58	.347
9½	156	80.4	2675.79	.356
10	156	83.2	2767.00	.394
10.5	156	86.0	2862.21	.459

but it was cracked across the bottom and stressed beyond safe limits so the test was discontinued. While preparations were being made to take the picture, the brick wall used to retain the loading sand collapsed. As the picture, Fig. 65 shows the extent of loading, the fallen material was merely removed and not replaced. Table No. 30 gives the data on this test. Fig. 66 shows deflections plotted against loads for this series of slabs.

TABLE No. 29

Depth Sand inches	No. Brick	Load in pounds		Deflection inches
		Per Sq. Ft.	Total	
2	40	18.7	592.36	.045
4	80	37.3	1184.72	.128
6	120	56.0	1777.08	.256
8	160	74.7	2369.44	.464
8	200	82	2601.44	.530
9	200	87.6	2781.62	.586
10	200	93.3	2961.8	.643
11	200	99.0	3141.98	.666
11	240	106.3	3373.98	.808
12	240	112.0	3559.16	.877
13	240	117.7	3734.34	.939
14	240	124.9	3914.52	.984
14	280	130.6	4146.52	1.048
15	280	136.6	4326.7	1.091
16	280	142.0	4506.88	1.161
16	320	149.4	4738.88	1.223
17	320	155.0	4919.06	1.274
18	320	160.7	5099.24	1.335
21	320	117.8	5639.78	

All slabs of this series were tested over spans of fourteen feet. The results are very much higher than was expected but the different specimens checked well with each other. According to Hool and Whitney, allowing 800 lbs. and 16,000 lbs. respectively, a concrete slab four and one-fourth inches effective depth would be required to carry the safe load indicated. No adequate explanation of this result can be offered at this time unless it be that the tensile strength of the brick really comes into play more than has been supposed before.

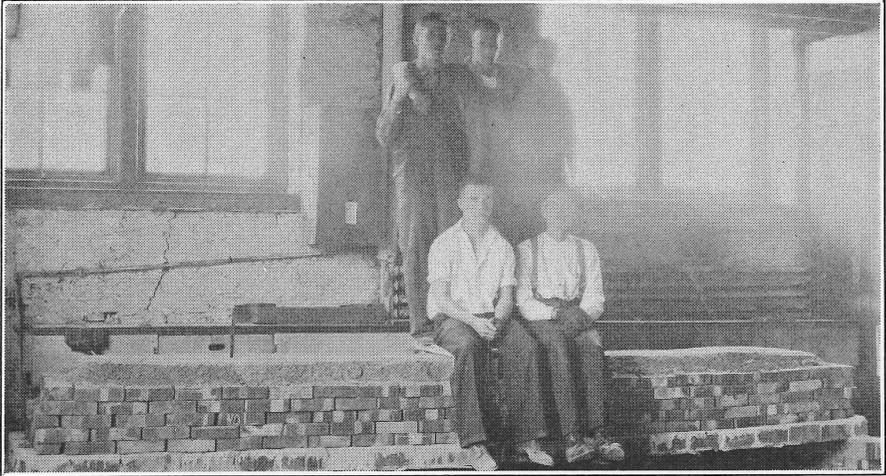


Fig. 64

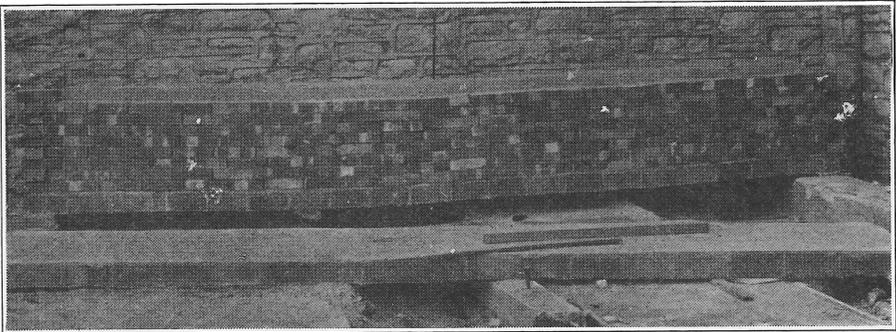
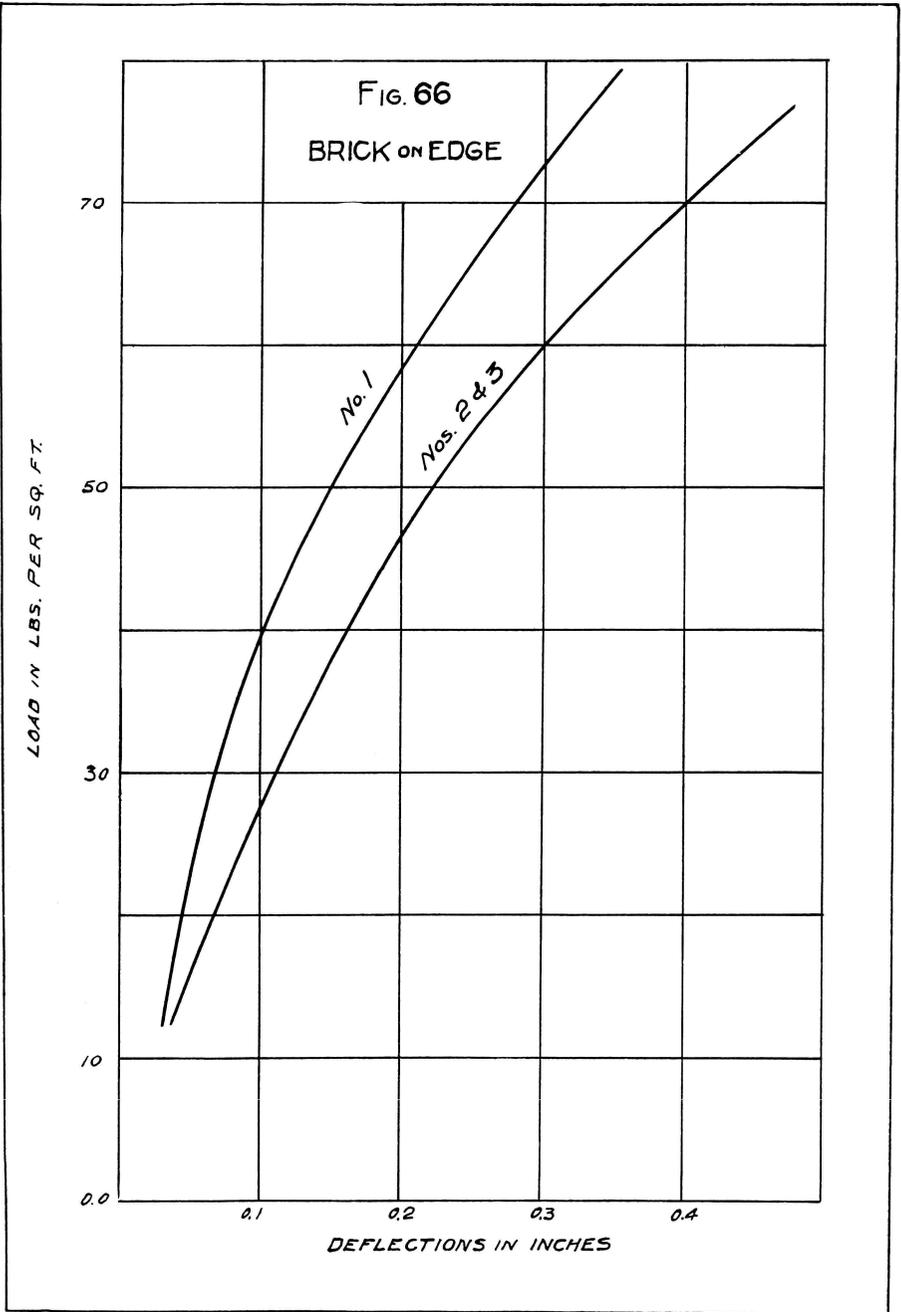


Fig. 65

The ribbed slab, shown under the failure load in Fig. 67 was loaded over a period of twenty-four hours. It was over reinforced somewhat. The load was applied as rapidly as the men working at it could conveniently handle the material. The load of 36 inch sand exhausted the sand available and was applied just before noon. About one o'clock the deflections had increased as shown in the last reading in Table No. 31



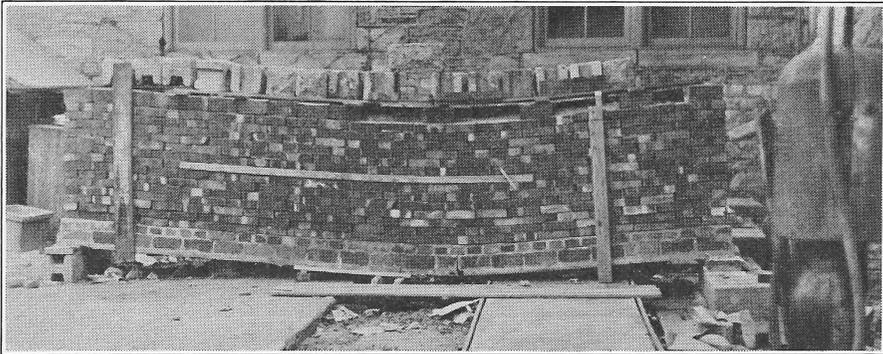


Fig 67

TABLE No. 30

Depth Sand inches	No. Brick	Load		Deflection inches		
		Total	Sq. Ft.	North	South	Av.
2	40	609	18.5	.041	.053	.047
4	80	1219	37.0	.092	.126	.105
6	120	1827	55.5	.257	.282	.269
7	160	2249	69.0	.398	.410	.404
8	160	2438	74.7	.447	.467	.457
9	200	3858	118.3	.532		.532
21	360	6052	185.6			

but had apparently come to rest. Boards were placed across the load so as to apply the load to the brick instead of to the sand and sixteen heavy concrete blocks averaging 63 lbs. each were put on. The slab showed no signs of distress until the last block was put in place when it suddenly failed in shear between the ribs and the top slab. There was no sign of crushing in any part of the slab. This result indicates a probably safe load of about 100 lbs. per sq. ft. but needs more investigation to make this definite. The addition of stirrups to join slab and ribs would also help.



## CHAPTER VII

## DISCUSSION OF RESULTS AND CONCLUSIONS

The series of tests run in an effort to determine directly the modulus of elasticity of brickwork in columns gave rather contradictory results. Two series of columns or pillars, each eight inches square and roughly two feet high, were tested, one being made with a 1-0.1-3 mortar and the other with a 1-0.1-2 mortar. The sand used in

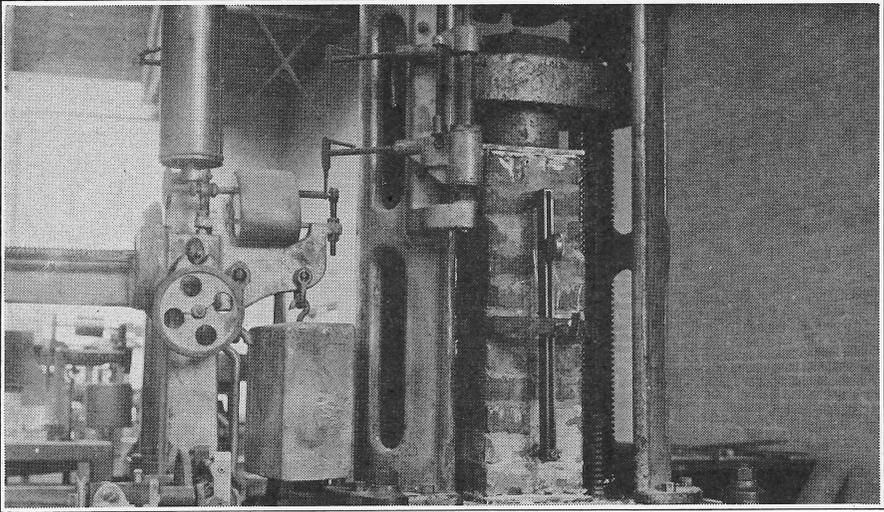


Fig. 68

these tests was the fine sand used in the first set of beams. The tests were made in a 150,000 lb. Olsen testing machine as shown in Fig. 68. Deformations were measured with a twenty inch Berry strain gauge as shown, two gauges being used on opposite sides of the pillar.

The 1-3 pillars gave rather consistent results;  $E = 3,333,000$ ,  $2,959,000$  and  $3,221,000$  with an average of  $3,171,000$ . The first column tested failed at 141,000 lbs. total load or about 2,200 lbs. per sq. in. None of the other five failed at the full capacity of the machine. The one that failed was a 1-3 (cement-sand-mortar). Two of the 1-2 series showed very slight cracks just under the full capacity of the machine. The moduli as determined from the 1-2 series were much less satisfactory. No. 1 was in fair agreement with the 1-3 series giving a value of  $3,492,000$ , No. 2 gave 209,000 only, and No. 3 gave 287,000 only. No explanation of this wide variation can be offered now, since no factor we could observe was varied. All specimens were bedded and capped with plaster of paris. An average of all six determinations gives  $2,250,000$ .

Table No. 32 gives certain information not tabulated in earlier chapters and Table No. 33 gives values of  $d$  and  $p$  observed and calculated from known measurements and value of  $k$ ,  $j$ , and  $n$  calculated from observed data secured from the beams. The value of  $k$  was secured by graphically locating the neutral axis and computing the ratio. The value of  $k$  for each beam of each series was determined separately

and the three averaged. The value given for  $j$  was secured by using the value of  $k$  found for that series in the formula  $j = i \frac{k}{3}$ . The value found for  $n$  was calculated

$$\text{from the formula } n = \frac{k^2}{2p(1-k)}.$$

It will be noted that the different beams of each series are quite consistent among themselves and that there is very good agreement among the different series. As expected, the S2 series gave the best results. There is no evident reason why the

TABLE No. 32

Beam No.	Days Age at Test	Max. Load	Safe Load	Wt. Lbs.	Cause of Failure
S1.A	49	14,000		1648	steel slipped
S1.B	44	7,000	5,000	1800	steel slipped
S1.C	42	11,600		1850	diagonal tension
S2 A	41	18,000		1745	tension in steel
S2 B	40	18,000	6,000	1540	tension in steel
S2 C	46	17,000		1540	tension in steel
H1-A	44	15,000		1440	horizontal shear
H1-B	44	13,000	4,000	1395	diagonal tension
H1-C	53	11,000		1370	diagonal tension
H2-A	44	13,500		1388	diagonal tension
H2-B	44	15,000	4,500	1400	tension in steel
H2-C	42	13,000		1498	diagonal tension
H3-A	56	14,000		1260	diagonal tension
H3-B	50	14,600	5,000	1270	tension in steel
H3-C	49	15,000		1240	tension in steel
H4-A	48	16,000		1420	diagonal tension
H4-B	45	14,000	5,000	1370	diagonal tension
H4-C	45	16,000		1370	diagonal tension

same amount of steel should have shown a higher yield point in these three beams than in some of the others, unless possibly there may be some connection between the yielding of the longitudinal steel and the stirrup steel. The S2-A and S2-B beams had one-fourth inch stirrups while the S2-C beams had only No. 9 wire stirrups. Diagonal tension failures, when they occurred, came quite near the yield point of the longitudinal steel, showing that the two reinforcements were very nearly balanced. The H1 series, while reasonably consistent within the series in all respects and while carrying maximum loads not greatly different from those of the other series, seems very far off in the values it gives for  $k$  and  $n$ . No explanation, other than that the design is wrong in having too much hollow above the neutral axis, is apparent since the same men did the work on all beams and the same material was used. This does not seem to be a likely explanation since the stresses near the neutral axis were low and the gross stresses were not high.

The safe loads given in Table No. 32 are calculated on the basis of one-third the average maximum load carried by the series. This seems conservative since the loads given in each case give stresses well below the basis of 16,000 for  $f_s$  and 650 for  $f_b$ . It seems likely that, with the substitution of either a harder steel having a higher yield point or of larger sections of steel in the longitudinal steel and with increased

stirrup reinforcing, beams of this size would be quite as strong as are the commonly used grades of reinforced concrete beams. The steel used in these tests was rather softer than ordinary reinforcing grades. It is significant that in no single test of the twenty-five in the second series and five in the first, was there any sign of failure in compression of either brick or mortar. The highest stress attained in compression was in the S2 series where at 18,000 lbs. total load, the stresses were calculated to be  $f_s = 42,600$  lbs. per sq. in. and  $f_b = 2,550$  lbs. per sq. in. The beam of the machine was actually balanced for some time at over 19,000 lbs. load, but of course the steel was elongating so that no reading could be taken and therefore this load was not recorded. It would, however, show a considerably higher unit compressive stress. This would seem to indicate for the particular brick, mortar and workmanship used a safe stress under these conditions of as much as 800 lbs. per sq. in. in compression. Whether this would hold good for other conditions involving different brick and mortar materials can be determined only by trial. Probably brick giving the same compressive strength, modulus of rupture and absorption and with similar control of conditions, a stress of 650 lbs. per sq. in. and a value of  $n = 15$  would be a perfectly safe basis for design. The effect of different factors involved in the strength of brickwork under these conditions is not indicated by our present knowledge of the strength of brickwork.

As in other respects, the results are in fair agreement as to the value of  $n$ , with the single exception of the H1 series. The values range from 13.3 to 21, with an average of 18, excluding the H1 series which gave the extraordinary value of 84. Averaging in this figure, the average value comes to 29. The modulus of elasticity was calculated from the beam formula for deflection previously referred to for the S2 series, the value determined thus being 2,758,000. Taking  $n$  as 13.3, the modulus would be 2,255,000. This seems like fair agreement.

The slabs tested showed remarkably good results. The flat brick slabs with only 0.15 sq. in. steel per foot width were under reinforced to develop the full strength of the brick. However, the safe live load of 25 lbs. per sq. ft. which it actually carried was very satisfactory. The indications are that by increasing the steel to 5/16 inch bars this slab could be used over longer spans successfully or on shorter spans and heavier loadings. According to Diagram 21 given on page 23 of Hool and Whitney's Concrete Designer's Manual, the brick on edge slabs were reinforced correctly on the basis of 18,000 lbs. on steel and 800 on the brick or over reinforced for the assumed stresses. Assuming  $k$  and  $n$  the same as observed in the S2 series of beams at failure, the stresses at failure were 31,700 lbs. per sq. in. in steel and 1895 in brick. In all tests, both beams and slabs, the deflections were well within the allow-

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able limit of  $\frac{1}{360}$  of the span at the design loads.

One vital question remains to be considered—that of cost. Obviously, the labor costs on such a job as this where no specialized facilities were available, will be considerably higher than on a large job. It took a bricklayer with the assistance of one helper an average of three hours to lay up a solid beam, including some time spent arranging forms and getting reinforcing ready. It is believed that with practice on this type of work enough to reduce it to routine, a bricklayer will be able to lay practically as many brick as he does on average work or to complete a beam the size of these approximately every two hours. With brick at \$17.00 per 1000 and other materials at average prices and allowing for the saving in form material and form labor, it seems likely that this construction can compete on even terms with concrete at the usual cost of \$16.00 to \$17.00 a cu. yd. With brick cheaper and concrete

TABLE No. 33

Beam No.	d in inches	p	k	j	n
S1-A	12.75	.00863	.502		
S1-B	12.75	.00863	.449		
S1-C	12.75	.00863	.391		
average			.447	.851	21
S2-A	12.75	.00863	.408		
S2-B	12.75	.00863	.392		
S2-C	12.75	.00863	.337		
average			.379	.874	13.3
H1-A	12.50	.0079	.658		
H1-B	12.50	.0079	.716		
H1-C	12.50	.0079	.628		
average			.667	.778	84
H2-A	12.50	.0088	.353		
H2-B	12.50	.0088	.440		
H2-C	12.50	.0088	.403		
average			.398	.867	15
H3-A	12.0	.00917	.379		
H3-B	12.0	.00917	.525		
H3-C	12.0	.00917	.450		
average			.451	.850	20
H4-A	14.0	.00786	.600		
H4-B	14.5	.00758	.336		
H4-C	14.5	.00758	.338		
average			.442	.853	21
average for all beams			.464	.849	29.3
averages excluding H1 series			.423	.856	18.1

materials the same or higher than the average, there is likely to be a saving in using brick. Assuming that it will be possible to control the water-cement ratio more closely with brick than with concrete and so get high early strength as well as high final strength, there will be a further saving due to the reduction in the time forms are required to be left in use. Earlier removal of forms will allow earlier finishing of the interiors also, which will reduce the total completion time as well as the time that money is tied up in a building before it can be used. Also, in buildings having a frame of reinforced concrete, substitution of brick for concrete may remove the necessity for facing with brick as a separate operation reducing both cost and time. There are many other applications not here mentioned that may appear as the method comes into use and its possibilities are recognized.

The conclusions offered herein are tentative only and as applied with certainty only to the particular conditions under which these experiments were made. The number of tests is too few to more than indicate possibilities and they need checking with a larger number of specimens and with various grades of brick.

The whole question of adhesion and shear in mortar needs thorough investigation. It seems likely that a mortar can be developed which will have desirable working qualities combined with much greater adhesion and greater resistance to shear stresses than those in common use now have. It is hoped that someone with better facilities at his command will take up this problem and work out more definite conclusions than those which follow:

1. Slabs and beams of reinforced brickwork are technically practicable under American building conditions.

2. Such slabs and beams react in a manner practically identical with the reactions of reinforced concrete, due allowance being made for properly proportioned stresses in concrete and steel.

3. The modulus of elasticity of brickwork made with cement-sand mortar and brick of the quality used in this series of tests may be assumed as 2,000,000 lbs. per sq. in. for ordinary calculations.

4. Stresses under the conditions of this experiment, of 650 lbs. per sq. in. on the brickwork in compression is safe and probably unnecessarily conservative.

5. Shearing stresses in beams are not so well resisted by brickwork as by concrete. While small beams may be safe without stirrups, no important beam should be made without at least light stirrups. Stirrups in general should be heavier than called for by standard concrete practice, just how much, is a question yet to be determined. The evidence available indicates that the difference need not be great. It seems very important that some top reinforcing be given near the ends of beams either by placing small rods in the uppermost mortar joint or by bending up one or more rod.

6. Slabs made of not more than one course of brick seem to have no difficulty in resisting shear likely to come on them if the slab is properly designed in other respects.

7. Careful and accurate control of the moisture in the brick or, more properly, the per cent of unsatisfied absorption of the brick is very important. Brick with too high absorption will injure the mortar by removing too much water. Brick completely saturated or glazed to prevent absorption are hard to lay, probably do not develop the full adhesion of the mortar and do not take full advantage of the water-cement ratio law. The necessary degree of control will have to be based on further experiment to determine the correct conditions.

8. The direct cost of brickwork as compared with concrete is likely to be about the same with some probability of saving in form work and possibly some speeding up of completion schedules and other indirect savings.





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