UNIVERSITY OF MANITOBA

AN INVESTIGATION OF CONCRETE BLOCK-FORMED BEAMS

by

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A THESIS

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ABSTRACT

A variety of masonry beams (concrete block formed) were built and tested under third point loading. A comparison is made between the experimental data and the theoretical load carrying capacity of the beams as analyzed by Ultimate Strength Theory for reinforced concrete.

A C K N O W L E D G E M E N T S

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I. INTRODUCTION

The precast concrete block has become a most popular building component today. Its use has expanded from partition walls to load carrying structural elements -- beams, columns, walls.

Numerous advantages to using reinforced masonry construction have been realized by contractors, architects, and engineers. Among them:

(1) The blocks serve a double purpose. They constitute the formwork for the concrete pour and contribute to the load carrying capacity of the structural members.

(2) The masonry units provide a most pleasing effect for both interior and exterior exposure. This eliminates the need for outside cladding, an economical factor which is particularly significant in cold weather conditions.

(3) Fire-ratings surpass those required by local building codes. Soundproofing qualities are also high.

(4) The method of construction is extremely simple -- lay blocks, install reinforcing steel, pour concrete fill. This procedure repeats itself through the structure.

At the present time only a limited amount of experimental data is available on the strength of concrete block-formed structural members and their behaviour as a composite unit under loading. Rational design methods for these members remain a subject of controversy among structural engineers.

1. OBJECT AND SCOPE OF INVESTIGATION

This thesis deals with an investigation of concrete block-formed beams subjected to symmetrical third point loading. The study was conducted at the Civil Engineering Testing Laboratory of the University of Manitoba in 1971.

The purpose was:

(1) To determine the load carrying capacity of experimental masonry beams and compare this to the theoretical capacity of the beams as analyzed by Ultimate Strength Theory under the "ACI Building Code Requirements for Reinforced Concrete"[1].

(2) To gain some further insight into the composite action of the elements in a reinforced masonry beam (precast blocks, mortar, concrete fill, reinforcing steel).

(3) To determine the effect of leaving portions of the block-formed beam unfilled. If various vertical cores (in areas of low stresses) are left hollow, does this affect the load carrying capacity and by how much? This could potentially result in a saving in concrete and labour as well as a reduction in dead load.

Reinforced masonry beams have direct practical applications as shear walls in multi-story buildings, grade beams in industrial warehouses, basement walls in single family dwellings, etc. For this reason, as near to full size models as the testing facilities would permit were chosen. Consequently, a span of twenty-four feet centre to centre with simple end supports was used. Eight inch wide concrete block and an overall depth of four feet were the other size parameters.

A total of seventeen tests were performed on nine different beams. These tests intend to throw some light on the behaviour and strength of reinforced masonry beams rather than attempt to establish rules and theories for their design. A much more extensive program would be required to accomplish the latter.

Former works in this area include a series of tests on single course masonry beams by Converse[2], Mayrose[5], and Saemann[8]. Their results indicated that reinforced masonry beams behave similarly to reinforced concrete beams. Edghill's[3] tests on beams with spans of 12'-0 and depths of 8 to 40 inches (one to five courses) also support this theory. In all of these investigations the block-formed beams were totally filled with concrete.

II. THE TEST PROGRAM

As indicated in the Introduction, all beams were made 8" wide, 4'-0 deep, and 25'-4" long. These dimensions were chosen on the basis that the experimental models would be representative of full size beams. It was the author's initial desire to use an overall depth of eight feet, thereby constituting a typical floor to ceiling height in a multi-story building. However, the testing facilities available restricted this to four feet (6 block courses).

Certain properties were characteristic of all the beams. The bottom course of each beam was constructed of lintel blocks with two reinforcing steel bars placed in the longitudinal "U-shaped" opening. The remaining five top courses were built with standard and/or H block. All courses were laid in running bond. This required the use of one half block at the ends of courses two, four and six. "Dur-a-wall" horizontal joint reinforcing was placed between every second course. These typical beam properties are shown in Figure 1.

Three different types of beams were built. These consisted of (1) beams with vertical concrete cores spaced at equal intervals along the span, (2) beams having their end thirds entirely filled with concrete and the middle portion (between the load points) left hollow, and (3) beams totally filled with concrete.

The following is a detailed description of each beam. At the end of each subsection is a diagram of each beam therein. Shaded areas in the diagrams represent portions filled with concrete.

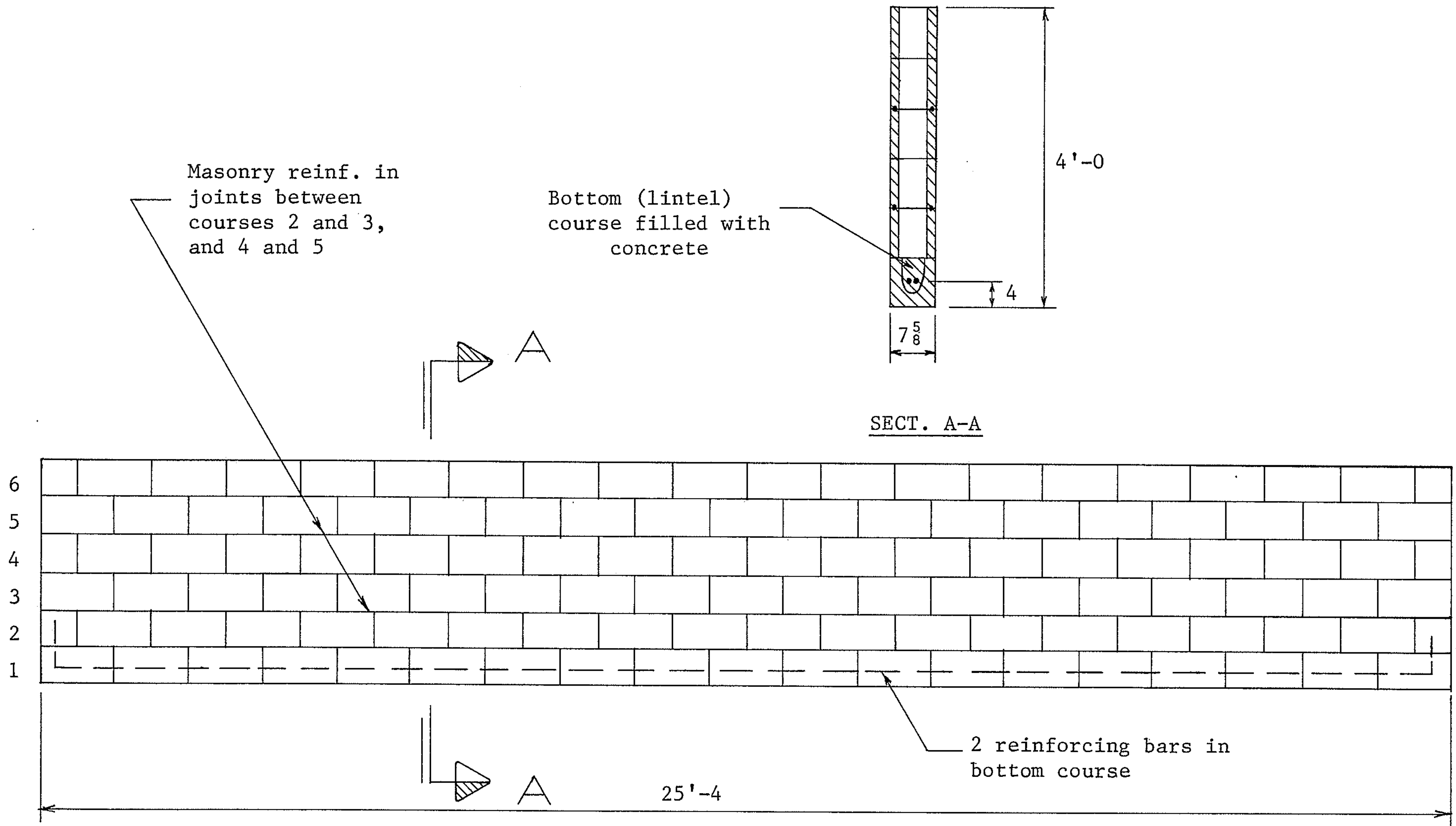

FIG. 1 TYPICAL BEAM PROPERTIES

1. CORE-FILLED BEAMS

Beam 1 (See diagram - Figure 2)

Although classified as a "core-filled beam", Beam 1 was the only beam with concrete fill in only the bottom (lintel) block course. The remaining five courses were constructed of standard block and left hollow. The bottom course also contained two #8 reinforcing bars.

Of particular importance in Beams 1, 2, 3, 4 and 5 was the construction sequence in regard to filling the beam with concrete. The bottom (lintel) course was filled first and allowed to harden. Then the remainder of the beam was built and filled.

Beam 2 (See diagram - Figure 3)

Like Beam 1, this beam had two #8 bars in the bottom course and five upper courses of standard block. However, vertical concrete cores at forty-eight inch centres were added for shear resistance. The size of each core was 4 7/8" x 5 9/16".

Beam 2A (See diagram - Figure 3)

Beam 2A was Beam 2 modified after it had been tested*. The only alteration was the addition of one intermediate concrete core between each existing one. Each new core was keyed into the lintel course by hammering out a 6" long by 4" deep shear key at the bottom of the core location. This procedure is fully explained in the Construction of Beams.

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* Throughout this thesis, an alphabetical subscript such as "A" or "B" following a beam number denotes a beam which had previously been tested and then modified or "repaired" to form the new beam.

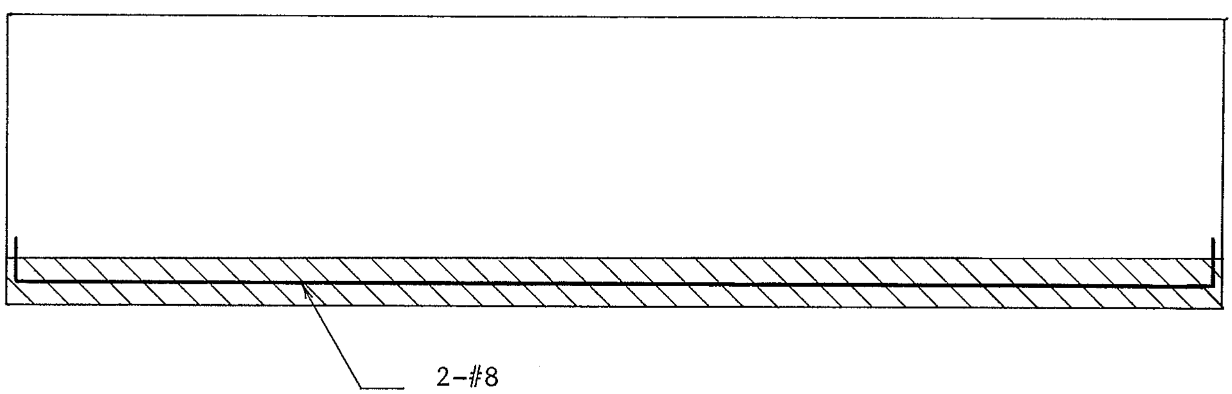
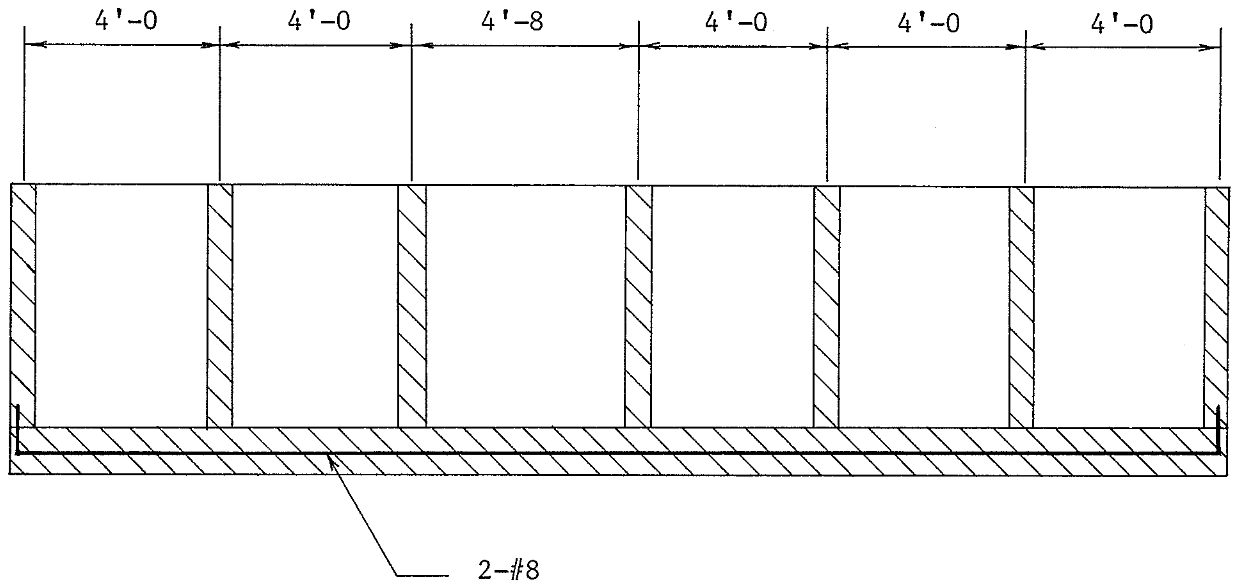
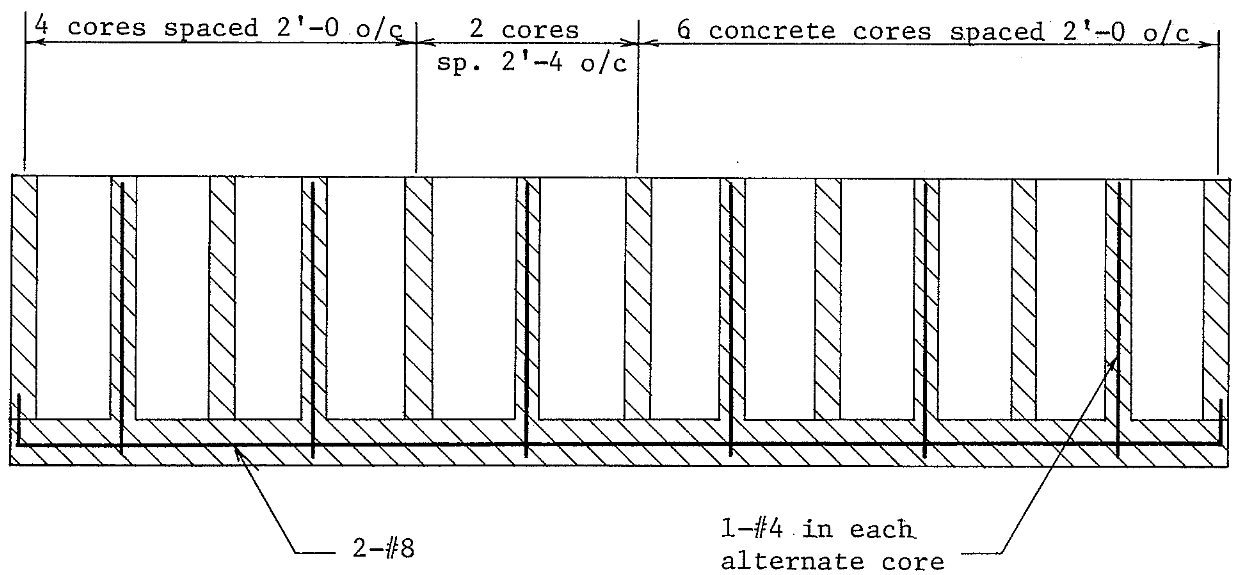


FIG. 2 DIAGRAM OF BEAM 1



BEAM 2



BEAM 2A

FIG. 3 DIAGRAM OF BEAMS 2 AND 2A

Beam 3 (See diagram - Figure 4)

Beam 3 was the same as Beam 2 with the addition of a #4 reinforcing bar in each vertical concrete core.

Beam 3A (See diagram - Figure 4)

An intermediate vertical core of concrete with one #4 bar was poured between the original concrete cores of Beam 3 to form Beam 3A.

Beam 4 (See diagram - Figure 5)

In Beam 4, the top and bottom courses were filled with concrete. The bottom course was reinforced with two #8 bars while the top course was left unreinforced. A vertical concrete core occurred every forty-eight inches.

Beam 5 (See diagram - Figure 5)

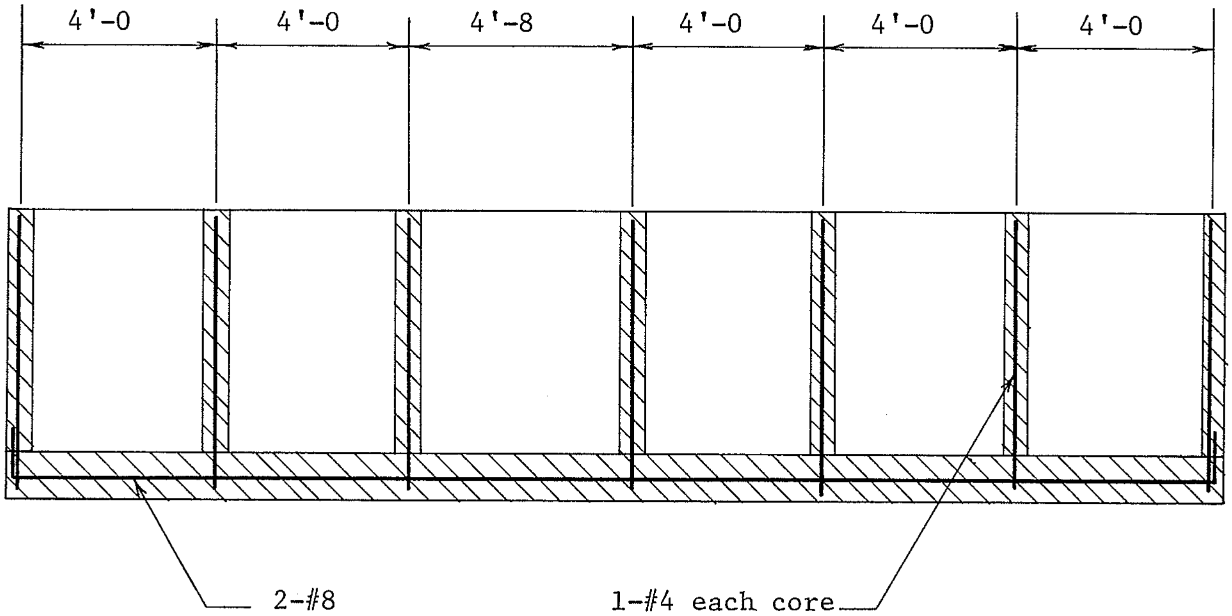
Beam 5 was the same as Beam 4 with the addition of one #8 bar in the top (horizontal) course and one #4 in each vertical filled core.

2. END-FILLED BEAMS

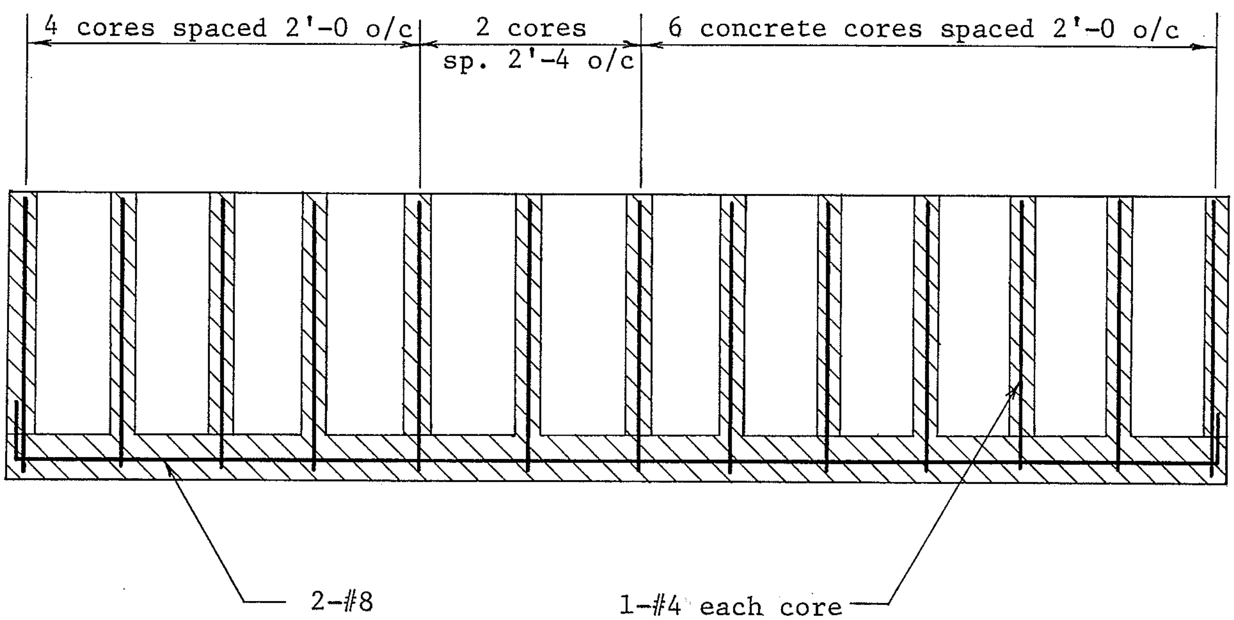
H block was used for the end third portions of the top five courses of these beams. The central third (between load points) was made of standard block. Each bottom (lintel block) course contained two #9 reinforcing bars, bonded in the end thirds only.

Beam 7 (See diagram - Figure 6)

Shear reinforcing in the totally filled end thirds of Beam 7 consisted of one vertical #4 bar every sixteen inches.

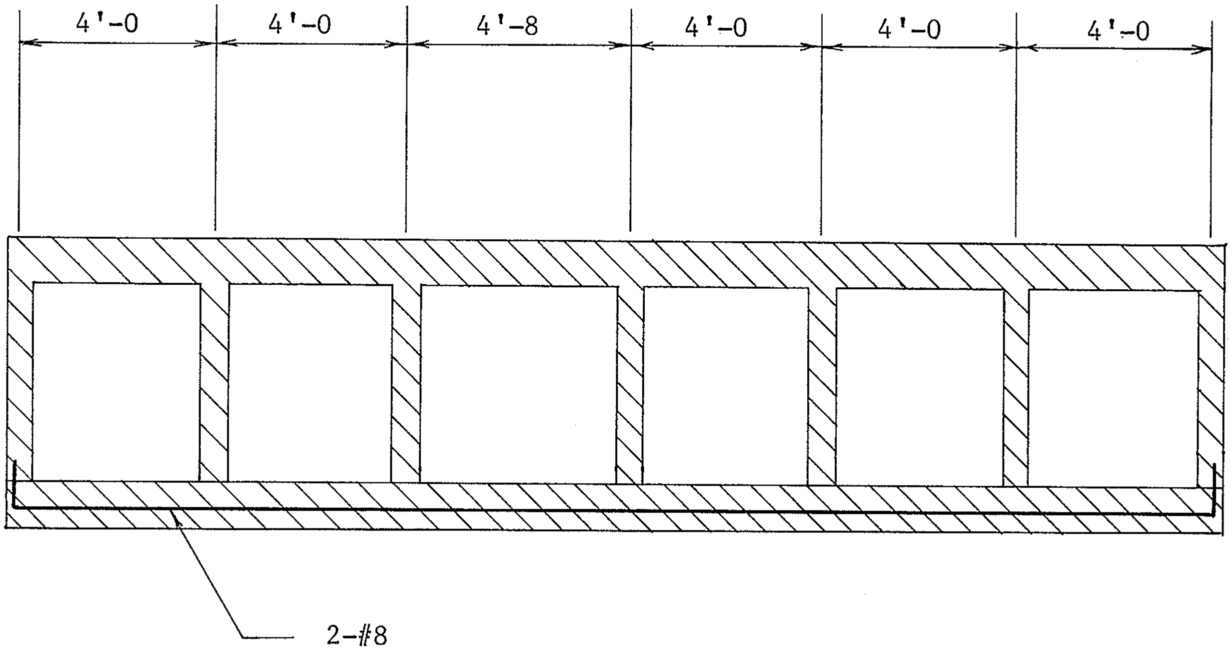


BEAM 3

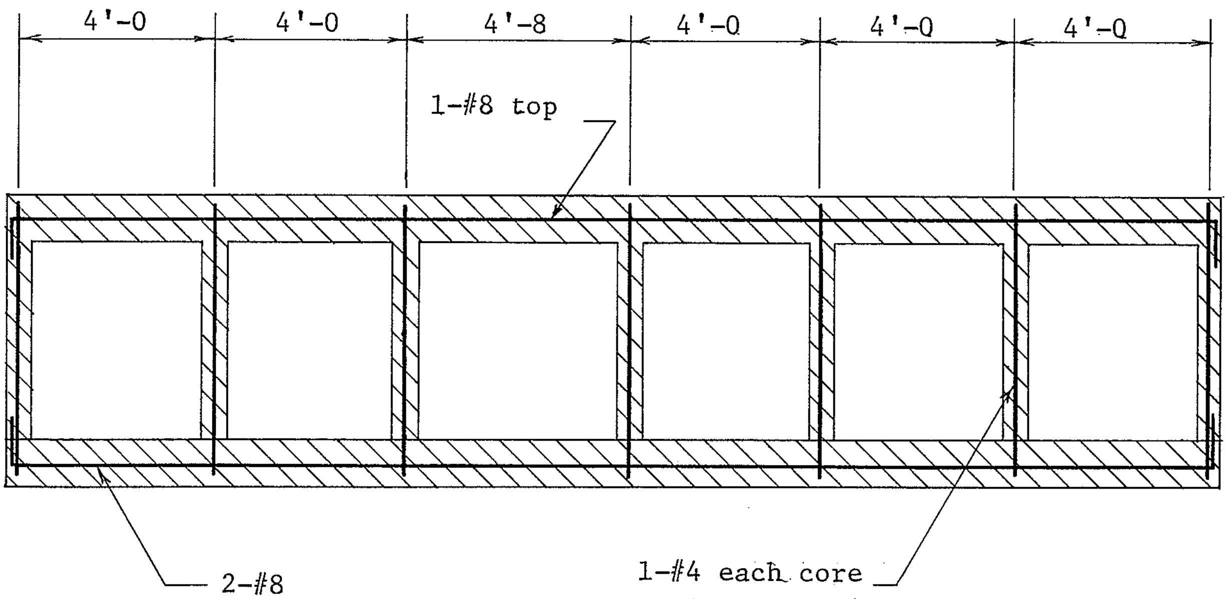


BEAM 3A

FIG. 4 DIAGRAM OF BEAMS 3 AND 3A

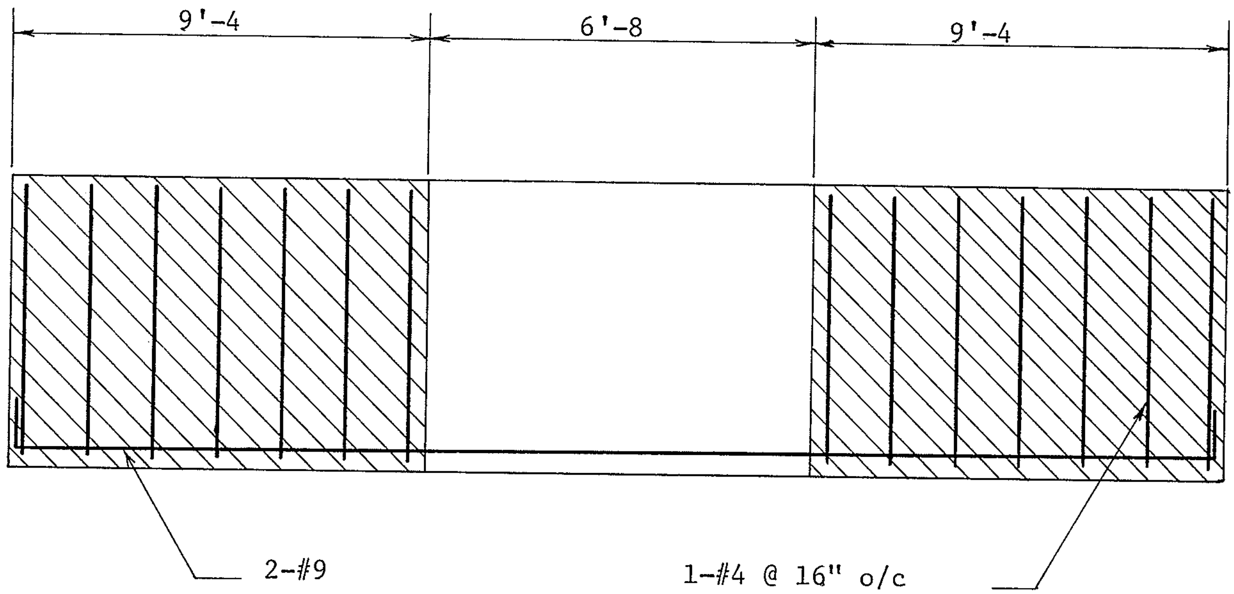


BEAM 4

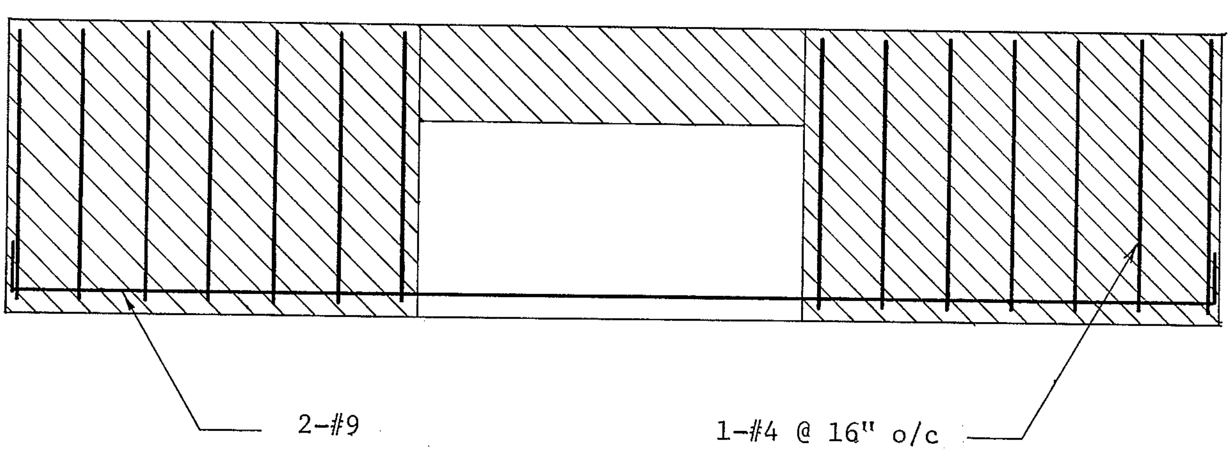


BEAM 5

FIG. 5 DIAGRAM OF BEAMS 4 AND 5



BEAM 7



BEAM 7A

FIG. 6 DIAGRAM OF BEAMS 7 AND 7A

Beam 7A (See diagram - Figure 6)

Beam 7A was the same as Beam 7 with one exception. The top two courses in the central third of the beam were filled with concrete.

Beam 8 (See diagram - Figure 7)

This beam was similar to Beam 7 except that the vertical bars in the solid end thirds were spaced at twenty-four instead of sixteen inches.

Beam 8A (See diagram - Figure 7)

After testing Beam 8, the top course of the central section was filled. This beam was now labelled Beam 8A.

Beam 9 (See diagram - Figure 8)

Beam 9 was again the same as Beam 7 with the only change being the spacing of the #4 vertical steel in the end thirds from sixteen to forty-eight inches.

Beam 9A (See diagram - Figure 8)

Beam 9A was a modification of Beam 9. This was achieved by re-building the top two central block courses of Beam 9 after it had been tested. In effect, Beams 9 and 9A were essentially the same.

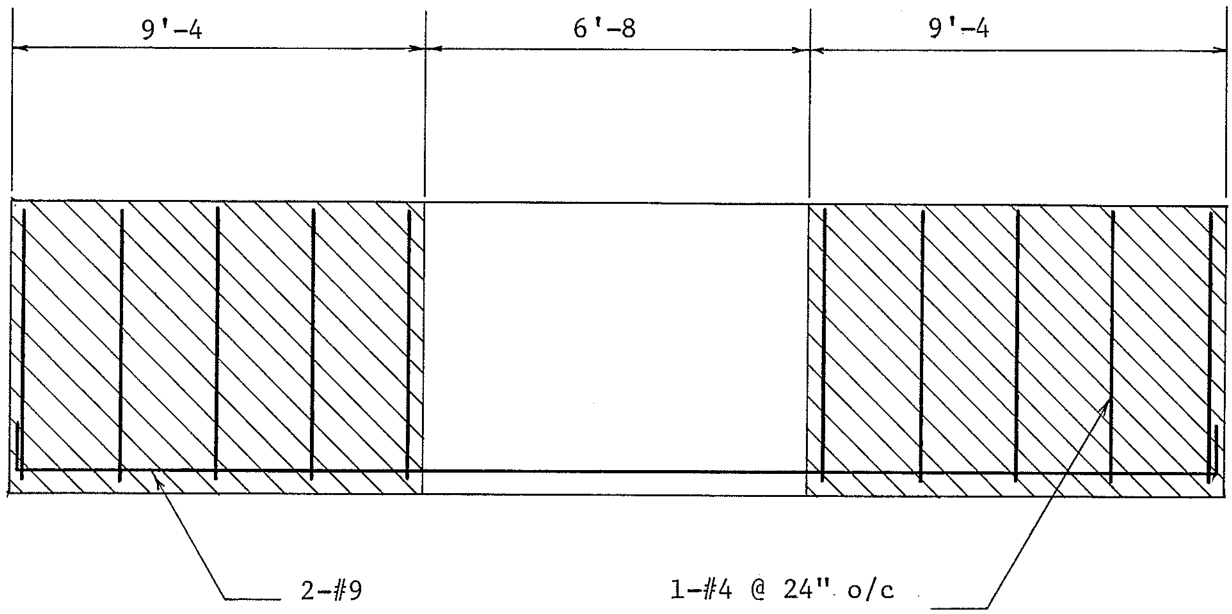
3. TOTALLY FILLED BEAMS

Beam 1A (See diagram - Figure 9)

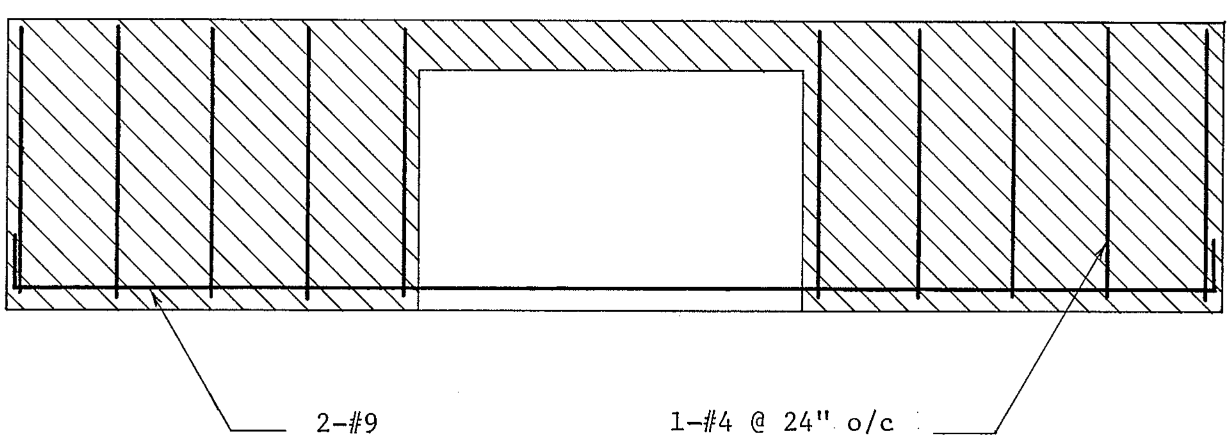
After Beam 1 was tested, it was totally filled with concrete and designated Beam 1A.

Beam 6 (See diagram - Figure 9)

Beam 6 was the only beam which was originally totally filled with



BEAM 8



BEAM 8A

FIG. 7 DIAGRAM OF BEAMS 8 AND 8A

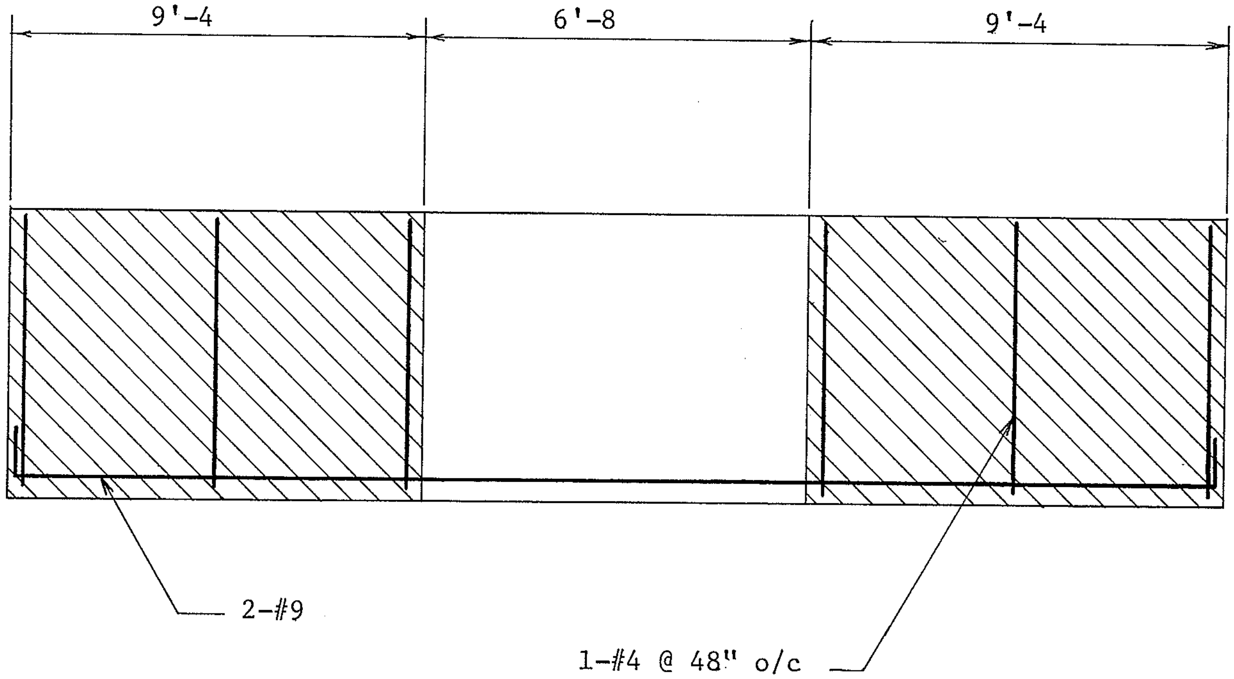
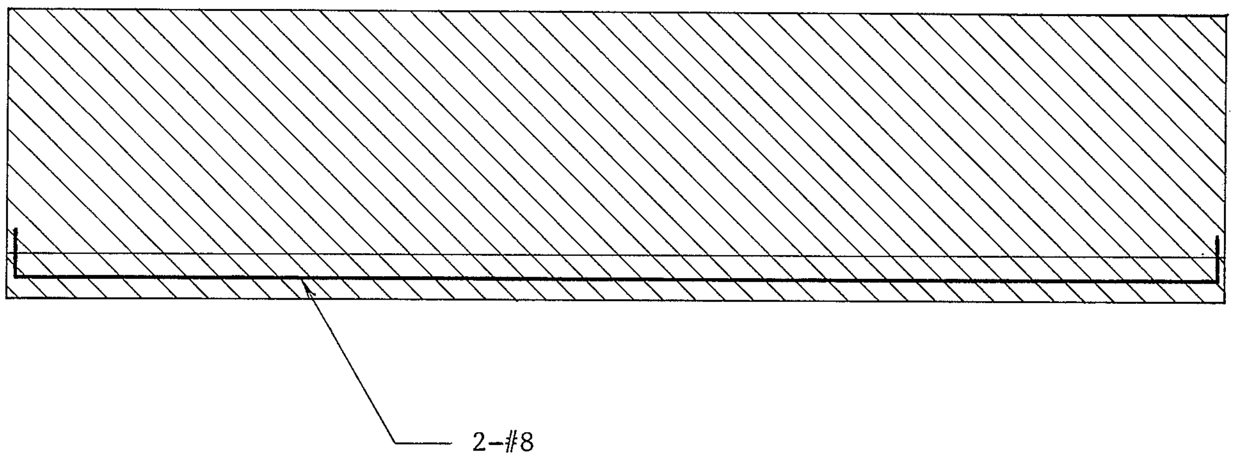
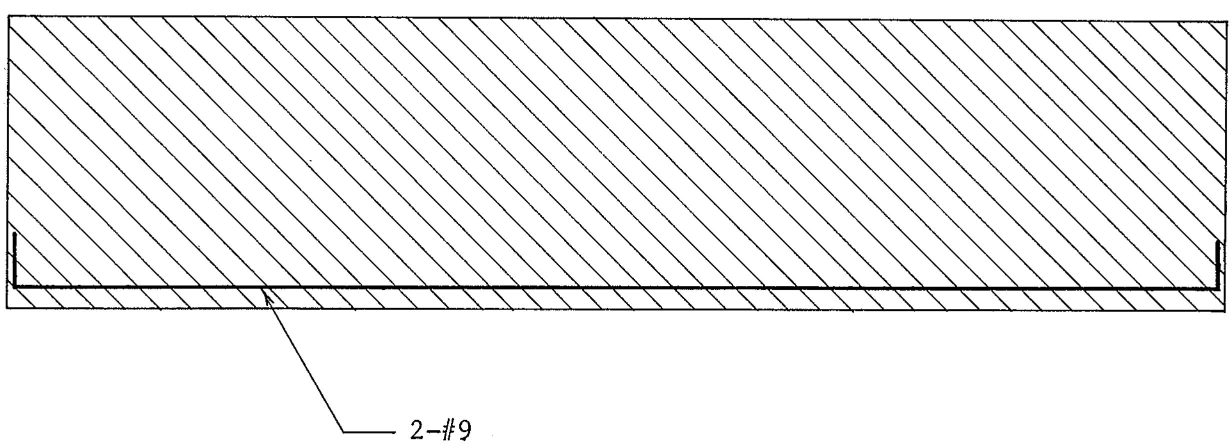


FIG. 8 DIAGRAM OF BEAM 9 (BEAM 9A SAME)



BEAM 1A



BEAM 6

FIG. 9 DIAGRAM OF BEAMS 1A AND 6

concrete. The bottom (lintel block) course contained two #9 reinforcing bars and the upper five courses were constructed of H block.

The last two beams consisted of 16'-0 portions of previously tested beams which appeared undamaged. In each of these beams, a single centre point loading was used.

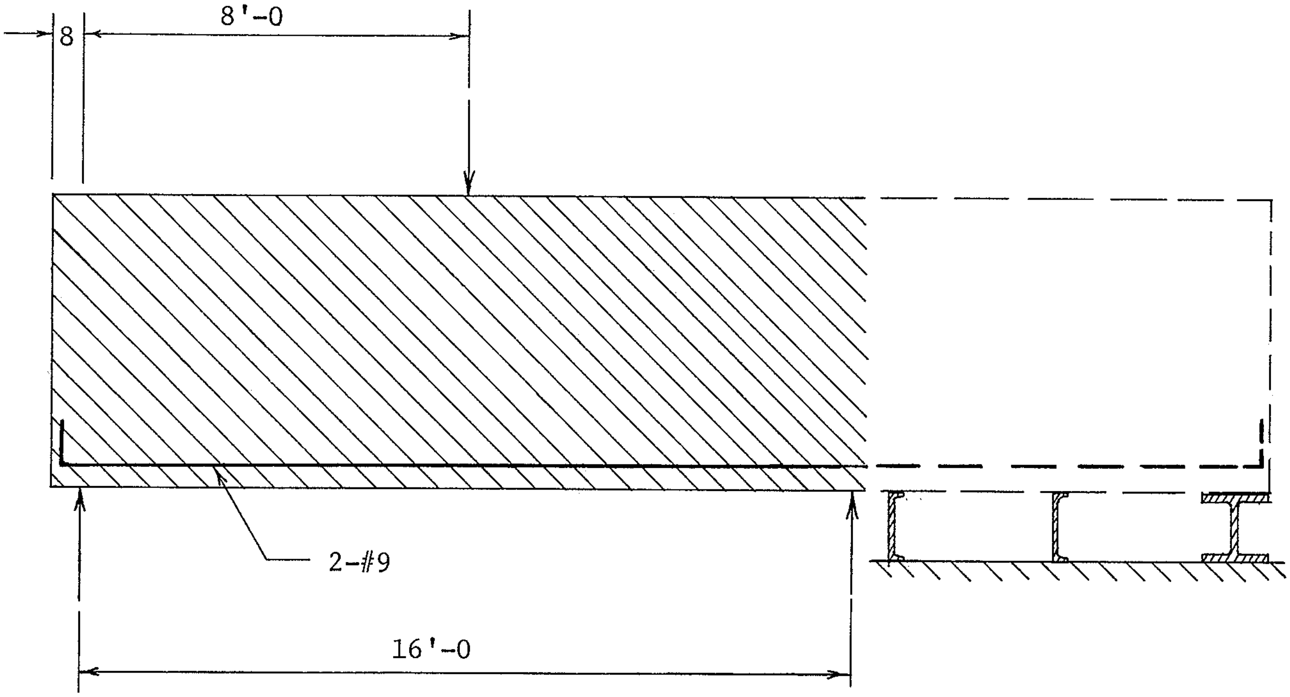
Beam 6B (See diagram - Figure 10)

The uncracked middle and right end third of Beam 6 formed Beam 6B.

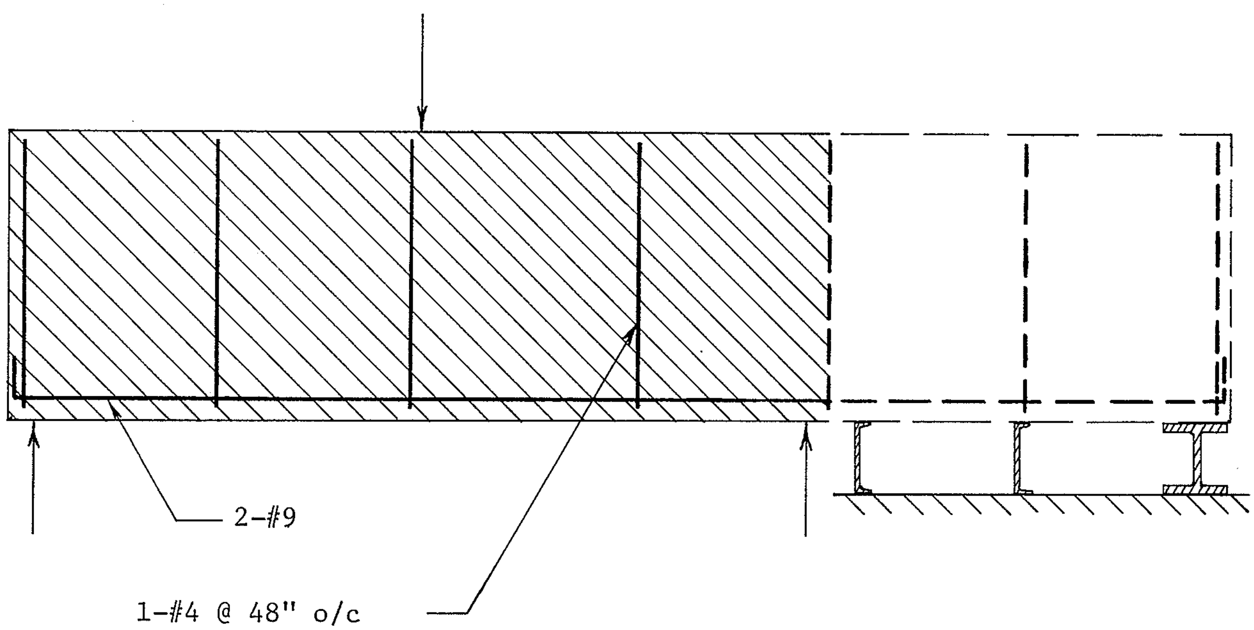
Beam 9B (See diagram - Figure 10)

This beam consisted of the middle and left end third of Beam 9A. The middle portion was filled with concrete and one #4 bar set in to provide one vertical bar every forty-eight inches along the 16'-0 span.

Table 1 is a concise summary of the test program. Part A lists the characteristics of the core-filled beams, Part B the end-filled beams, and Part C the totally filled beams. The table is repeated in the section headed Evaluation of Test Results for convenient reference.



BEAM 6B



BEAM 9B

FIG. 10 DIAGRAM OF BEAMS 6B AND 9B

T A B L E 1

BEAM SUMMARY

A. CORE-FILLED BEAMS

BEAM	BOTTOM STEEL	CORE SPACING	VERTICAL STEEL	REMARKS
B1	2 - #8	-	-	All cores empty
B2	2 - #8	48 in.	none	
B2A	2 - #8	24 in.	1-#4@48 in.	Every second core ineffective*
B3	2 - #8	48 in.	1-#4@48 in.	
B3A	2 - #8	24 in.	1-#4@24 in.	Every second core ineffective*
B4	2 - #8	48 in.	none	Top course filled
B5	2 - #8	48 in.	1-#4@48 in.	1 - #8 in top course

B. END-FILLED BEAMS

BEAM	BOTTOM STEEL	VERTICAL STEEL	CENTRE THIRD
B7	2 - #9	1 - #4 @ 16 in.	Empty
B7A	2 - #9	1 - #4 @ 16 in.	Top 2 courses filled
B8	2 - #9	1 - #4 @ 24 in.	Empty
B8A	2 - #9	1 - #4 @ 24 in.	Top course filled
B9	2 - #9	1 - #4 @ 48 in.	Empty
B9A	2 - #9	1 - #4 @ 48 in.	Full width mortar joints in top courses

T A B L E 1

(continued)

C. TOTALLY FILLED BEAMS

BEAM	BOTTOM STEEL	VERTICAL STEEL	SPAN	LOADING CONDITION
B1A	2 - #8	none	24' - 0	Third point
B6	2 - #9	none	24' - 0	Third point
B6A	2 - #9	none	16' - 0	Centre point
B9B	2 - #9	1-#4 @ 48 in.	16' - 0	Centre point

* These cores were damaged the first time the beam was tested.

III. CONSTRUCTION OF BEAMS

The beams were built in two series, Series 1 and Series 2, by the author and an experienced brick layer. The five beams of Series 1 were built directly in the loading frame to minimize handling. Each beam was tested and then several were modified and tested again. Series 2 followed the same procedure.

1. SERIES 1

Beams 1 to 5 of Series 1 were consecutively built. Construction methods were similar for each beam.

First the bottom course of lintel blocks were mortared together on a 2 x 10 timber plank supported every three feet on steel channels. Care was taken to use planks which were not badly warped. For this and each subsequent course, a horizontal guide line was run to insure that the blocks were laid in a straight line. The bottom lintel block course was then allowed to set for at least twenty-four hours before filling with concrete.

A 2-inch layer of concrete was shovelled into the block forms to insure good bond between concrete and steel. Brick fragments about 1 1/2 inches in size were set into the concrete to support the reinforcing steel. The two longitudinal steel bars were then placed in the course. The remaining void was filled with concrete and thoroughly vibrated with an electric vibrator. Where specified, vertical steel bars were set into the concrete cores as shown in Figure 11. The course was finished off by trowelling the concrete fill level with the top edge of the blocks and then allowed to set for at least twenty-four hours before further construction.

The next step was laying the remaining five courses. Double cored standard blocks were used for this purpose in all beams of Series 1. One half block was used at the extreme ends of courses two, four, and six to form a running bond construction. The end blocks of each course were laid first. After these were correctly levelled, a horizontal guide line was run from end to to end. Each interior block was then mortared and tapped into place. These procedures are shown in Figures 11 to 17.

Twenty-four hours after all blocks were mortared in place, filling of the vertical cores began. Concrete was shovelled into the required cores and vibrated twice -- when the core was half and totally full. Each core was filled level with the top block.

In Beams 4 and 5 a top course of concrete was required. This was monolithically poured with the vertical cores. For these top courses standard blocks were used only at those locations where a vertical core was to be filled. One core in the standard block was left open while the adjacent one was blocked off at the bottom with wire screening as shown in Figure 19. Lintel blocks were used for the remainder of the course as shown in Figure 18.

The top reinforcing bar in Beam 5 was laid into slots which were chipped out of the end and centre walls of each standard block shown in

FIG. 11 ALIGNMENT OF HORIZONTAL GUIDE LINE

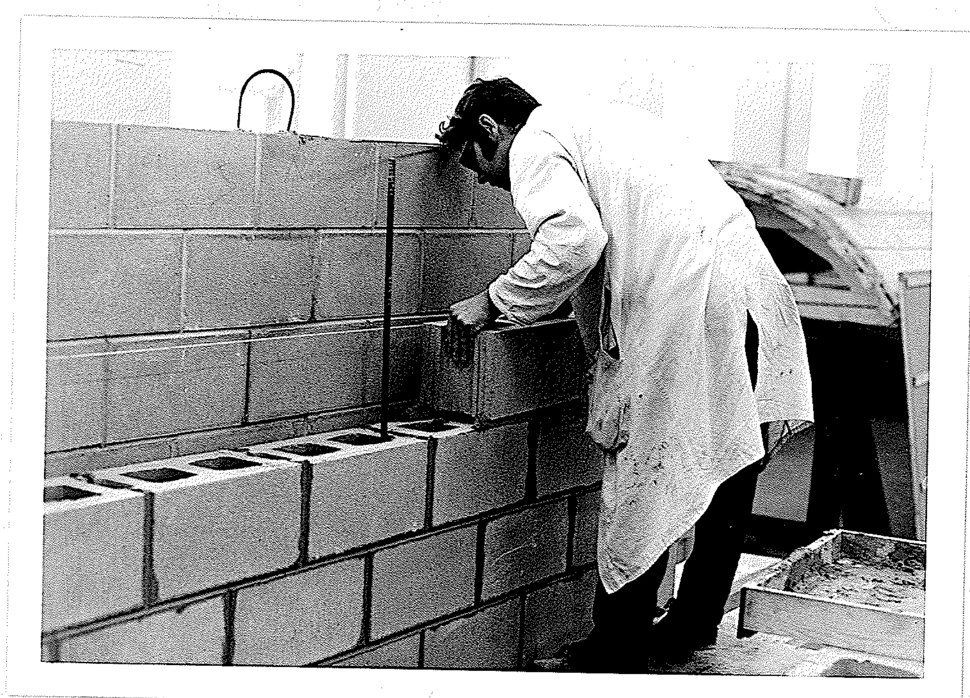


FIG. 12 METHOD OF LAYING BLOCKS TO HORIZONTAL GUIDE LINE

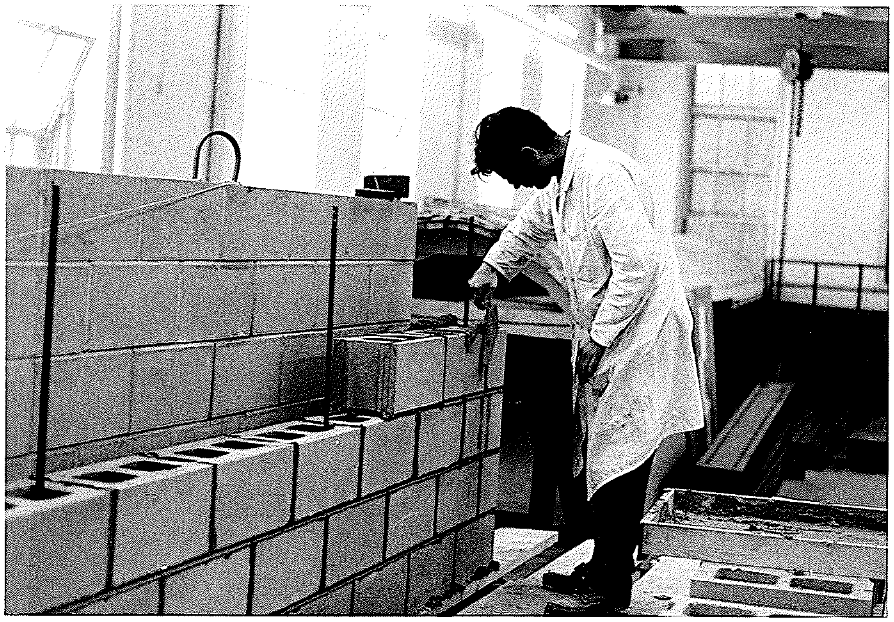


FIG. 13 SPREADING MORTAR FOR NEXT END BLOCK

FIG. 14 METHOD OF LEVELLING END BLOCKS

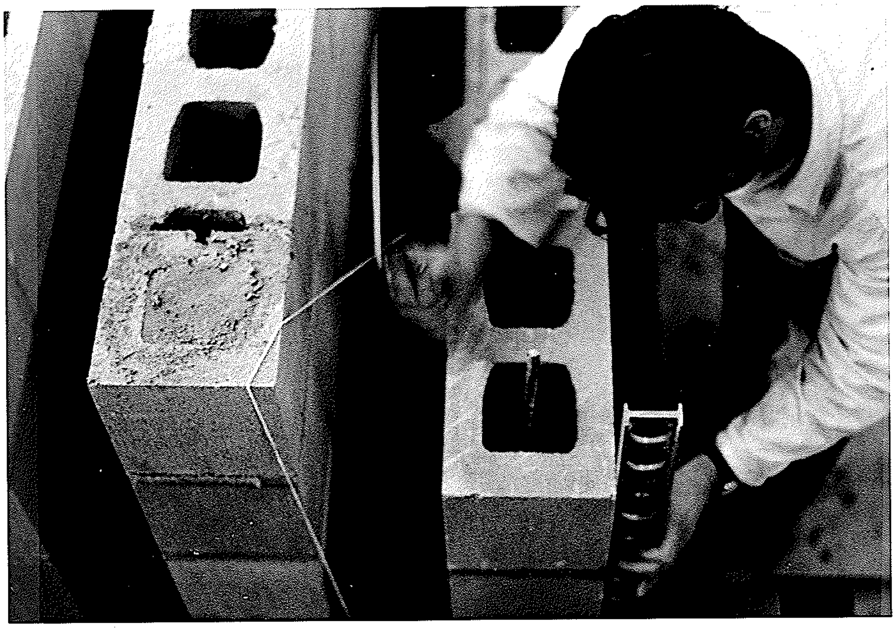


FIG. 15 TAPPING BLOCK INTO POSITION

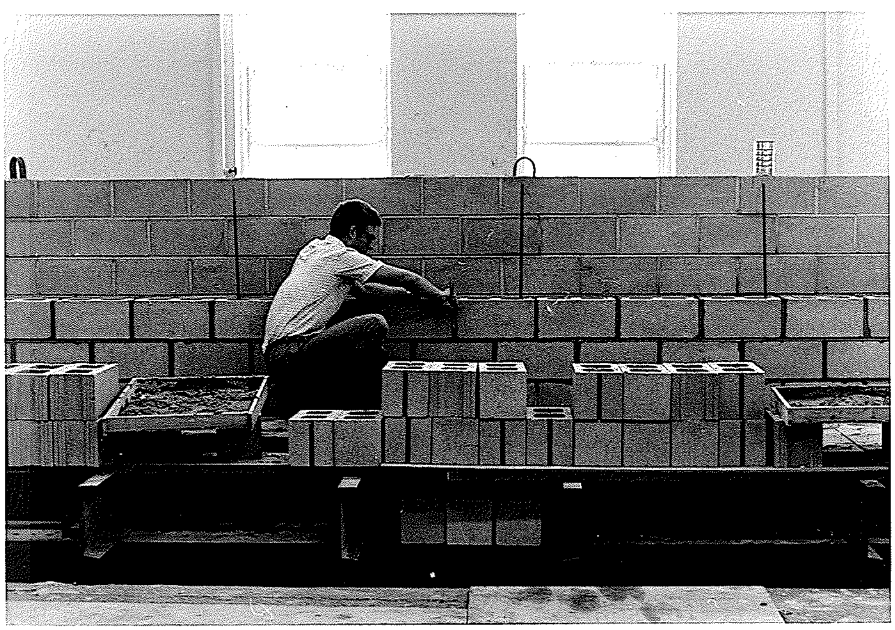


FIG. 16 METHOD OF FINISHING OFF JOINTS WITH JOINT IRON

FIG. 17 END OF BEAM SHOWING JOINT REINFORCEMENT BETWEEN COURSES FOUR AND FIVE

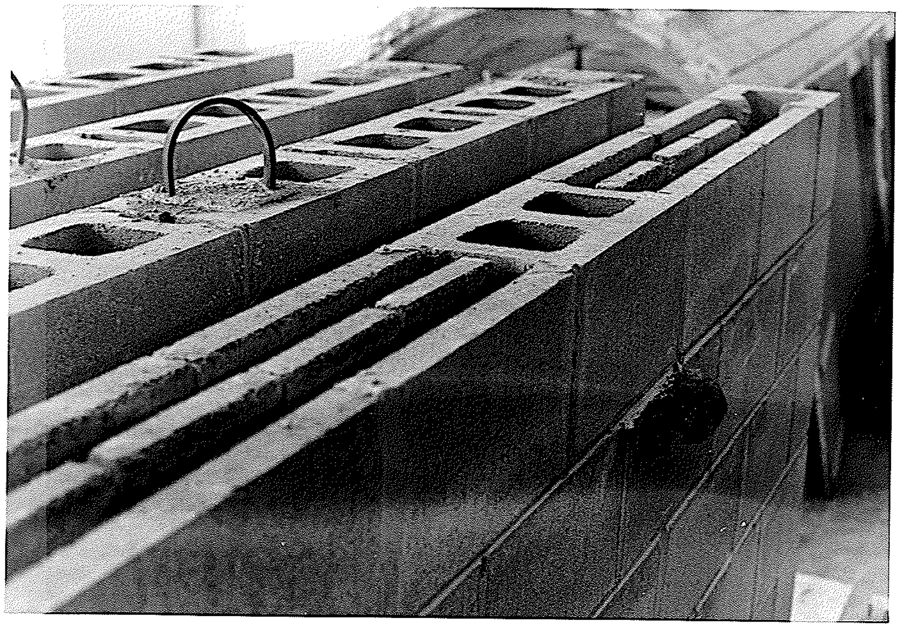


FIG. 18 TOP COURSE OF BEAM 4

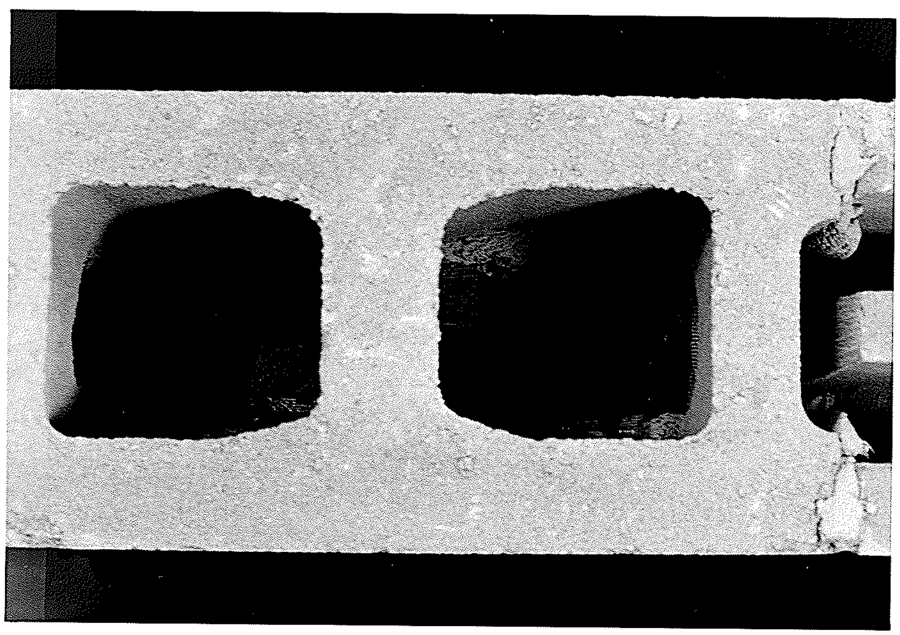


FIG. 19 LEFT CORE OF STANDARD BLOCK IN TOP COURSE

OPEN: RIGHT CORE "SCREENED OFF"

Figure 20. Methods of filling the beams with concrete are illustrated in Figures 21, 22 and 23.

The beams were then left to cure until testing.

Series 1A

This series consisted of Beams 1A, 2A and 3A. Beam 1A was formed by totally filling Beam 1 with concrete. Before this was done the centre of the beam was raised up to its initial position and supported. Cracks in the block joints were re-mortared as well as possible, and the beam was then filled.

Beams 2A and 3A called for additional vertical concrete cores centred between existing cores in Beams 2 and 3. Unlike the original cores the new ones were keyed into the bottom course of concrete. This was done by knocking out openings in the bottom course, which extended from the top of the block to the longitudinal reinforcing bars. These openings, six per beam, were made at each new core location with a portable jack hammer. The width of each opening was that of one core (approximately six inches). Plywood forms were made to re-enclose the sides of the core.

In both beams these cores were filled with concrete and one #4 bar put in.

Series 2

The second series of beams were built in a similar fashion to

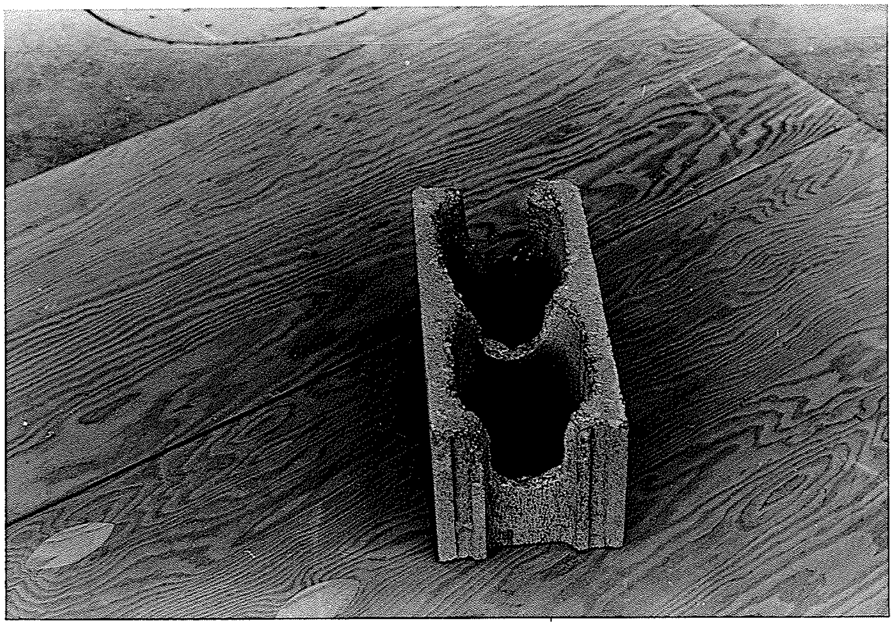


FIG. 20 TOP (STANDARD) BLOCKS USED IN BEAM 5

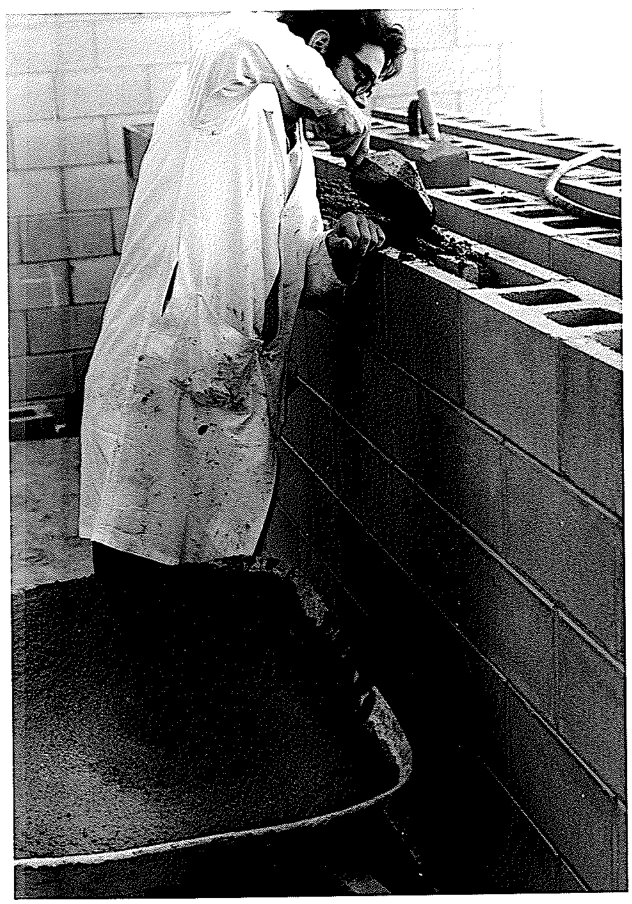


FIG. 21 FILLING VERTICAL CORES AND TOP COURSE OF BEAM 4

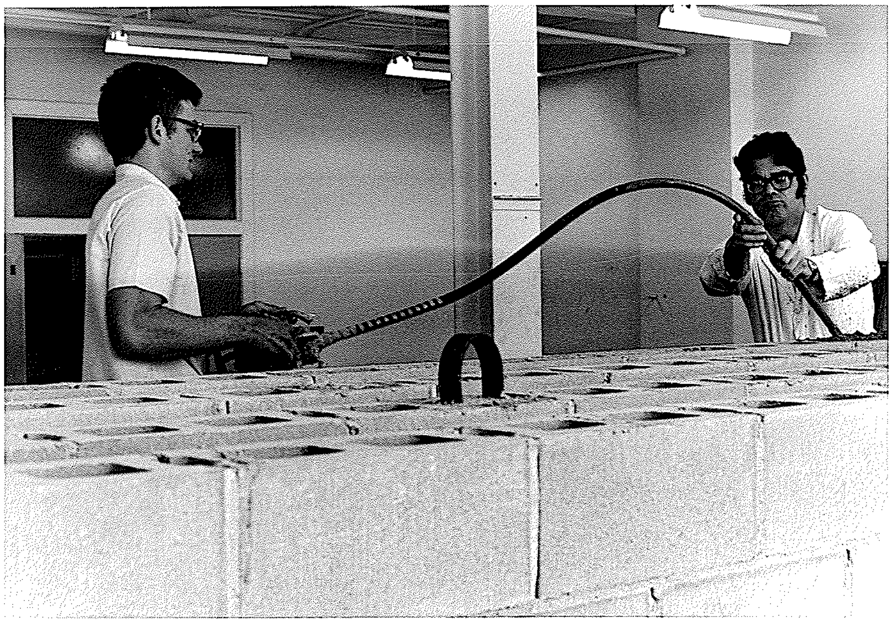


FIG. 22 VIBRATION OF CONCRETE FILL

FIG. 23 FINISHING OFF TOP COURSE OF BEAM 4

Series 1. A major difference was that the bottom course concrete was not separately poured.

The block laying procedure was the same as in Series 1 except that H blocks were used in some areas as outlined in The Test Program.

Concrete was delivered ready-mixed and shovelled into the beams. All four beams were filled on the same day with the same batch of concrete. This procedure took about four hours and as a result, the concrete put into the last third of each beam was considerably stiffer than that at the start. Although thoroughly vibrated, it was apparent that the concrete no longer flowed into the voids as well as before.

Series 2A

Beams 6B, 7A, 8A, 9A, and 9B comprised Series 2A.

The bottom support at the cracked end of Beam 6 was relocated in Beam 6B to form a span of 16'-0.

The top courses of Beams 7 and 8 were blocked off with wire screening and then filled with concrete. These beams were then designated Beam 7A and 8A.

In Beam 9, the standard blocks in the two top courses were replaced. Care was taken to make full width mortar joints.

Finally, Beam 9A was modified into Beam 9B by filling totally with concrete. One end support was moved to produce a 16'-0 span.

IV. MATERIAL PROPERTIES

1. CONCRETE BLOCKS

Three different types of precast concrete blocks were used in building the beams - lintel, H and standard blocks. Details of each type are shown in Figure 24. The nominal block dimensions are 8" x 8" x 16" while actual dimensions are 7 5/8" x 7 5/8" x 15 5/8" to allow for a 3/8" mortar joint. Three blocks of each series were tested in compression. The results of these tests are listed in Table 2. A photograph of each block type is shown in Figure 25.

2. MORTAR

Mortar batches were prepared in a wheelbarrow. Each mixture was made up of the following materials: one part masonry cement, three parts fine sand, and enough water to make the mixture properly workable. This corresponded to a N.B.C. (1970)[6] type N mortar.

Two four-inch cubes were made from each batch of mortar. The results of compression tests performed on these cubes are listed in Table 3.

A sieve analysis was carried out on the mortar sand and these results are tabulated in Table 4. In Figure 26, the sand was graded as a fine-to-medium sand.

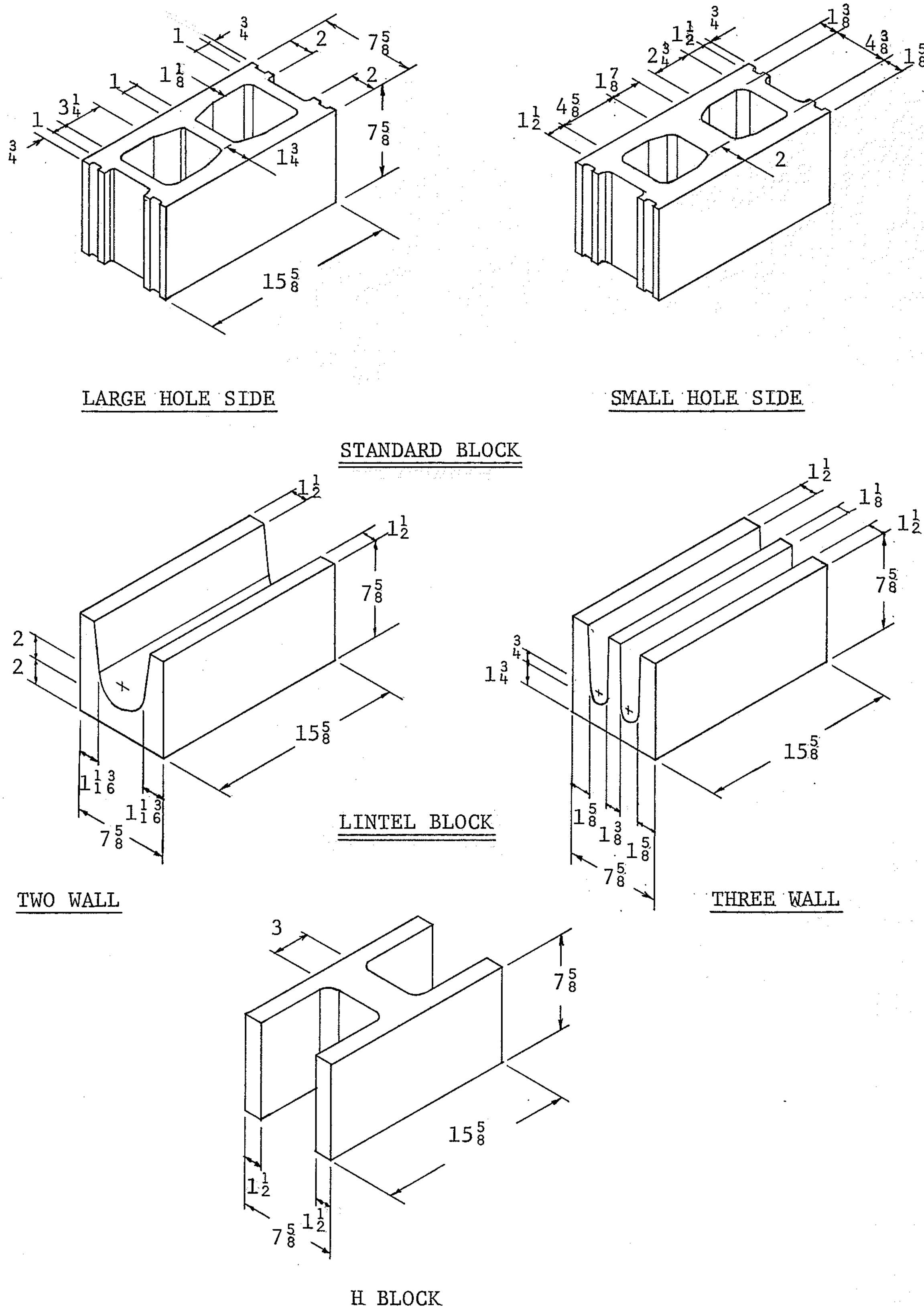


FIG. 24 CONCRETE BLOCK DETAILS

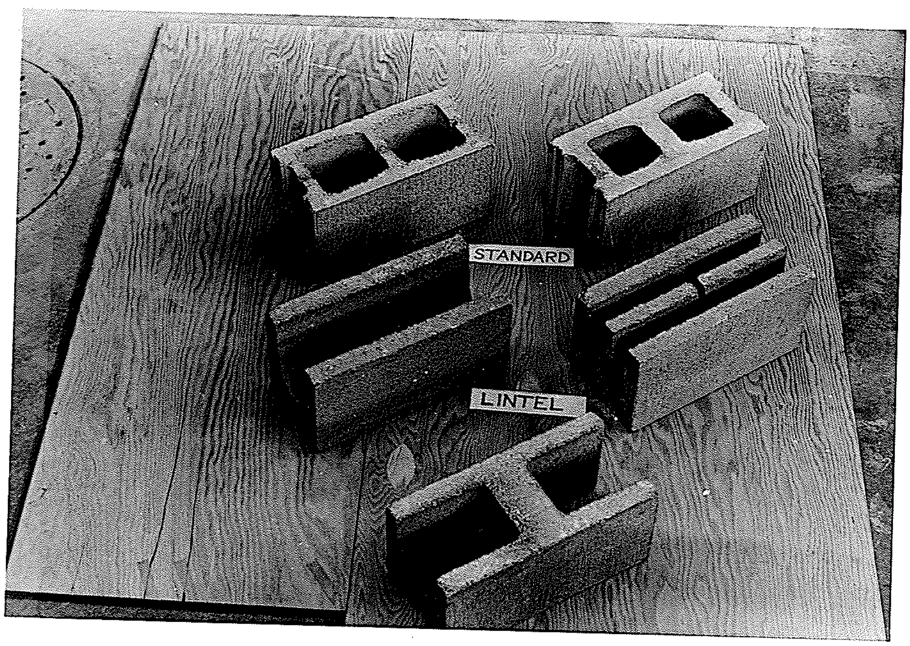


FIG. 25 PHOTOGRAPH OF BLOCK TYPES

TABLE 2

CONCRETE BLOCK COMPRESSION TEST DATA

TYPE OF BLOCK	TOTAL LOAD (KIPS) (AVERAGE OF THREE UNITS)	COMPRESSIVE STRENGTH (PSI.) GROSS AREA	NET AREA
SERIES 1			
Standard	162	1,350	2,700
Lintel	195	1,625	3,250
SERIES 2			
Standard	165	1,375	2,750
Lintel	189	1,580	3,160
H	186	1,550	3,100

TABLE 3

MORTAR CUBE TESTS

BEAM NO.	TEST NO.	TOTAL LOAD (POUNDS)	COMP. STRESS (P.S.I.)	AVG f'm (P.S.I.)	AGE (DAYS)
1	1	24,000	1,500	1,480	10
	2	23,400	1,460		
1A	1	29,600	1,850	1,960	56
	2	35,400	2,210		
	3	29,100	1,820		
2	1	23,700	1,480	1,460	53
	2	22,900	1,430		
	3	23,600	1,470		
2A	1	23,600	1,470	1,470	122
3	1	24,900	1,560	1,660	49
	2	26,700	1,670		
	3	28,300	1,770		
	4	26,200	1,640		
3A	1	23,600	1,470	1,470	115
4	1	29,900	1,870	1,780	29
	2	33,300	2,080		
	3	22,300	1,400		
5	1	39,900	2,490	1,530	10
	2	22,100	1,380		
	3	17,600	1,100		
	4	18,400	1,150		
6	1	26,800	1,680	1,720	18
	2	28,100	1,760		
6B,9B	1	32,400	2,020	2,020	36
7	1	32,400	2,020	1,860	33
	2	27,200	1,700		
7A,8A	1	17,000	1,060	1,080	7*
	2	19,500	1,220		
	3	15,500	970		

T A B L E 3

(continued)

BEAM NO.	TEST NO.	TOTAL LOAD (POUNDS)	COMP. STRESS (P.S.I.)	AVG f'm (P.S.I.)	AGE (DAYS)
8	1	29,900	1,870	1,650	29
	2	22,900	1,430		
9	1	29,100	1,820	1,560	28
	2	20,800	1,300		
9A	1	17,100	1,070	1,070	9*
	2	17,300	1,070		

* Tests were performed on mortar used to redo the top central courses of these beams.

TABLE 4

SIEVE ANALYSIS OF MORTAR SAND

SIEVE NO.	WEIGHT PASSING (GRAMS)	PER CENT PASSING
4	1,373	100.0
10	1,364	99.5
16	1,322	96.5
50	466	34.0
100	92	6.7
200	7	0.5

3. CONCRETE FILL

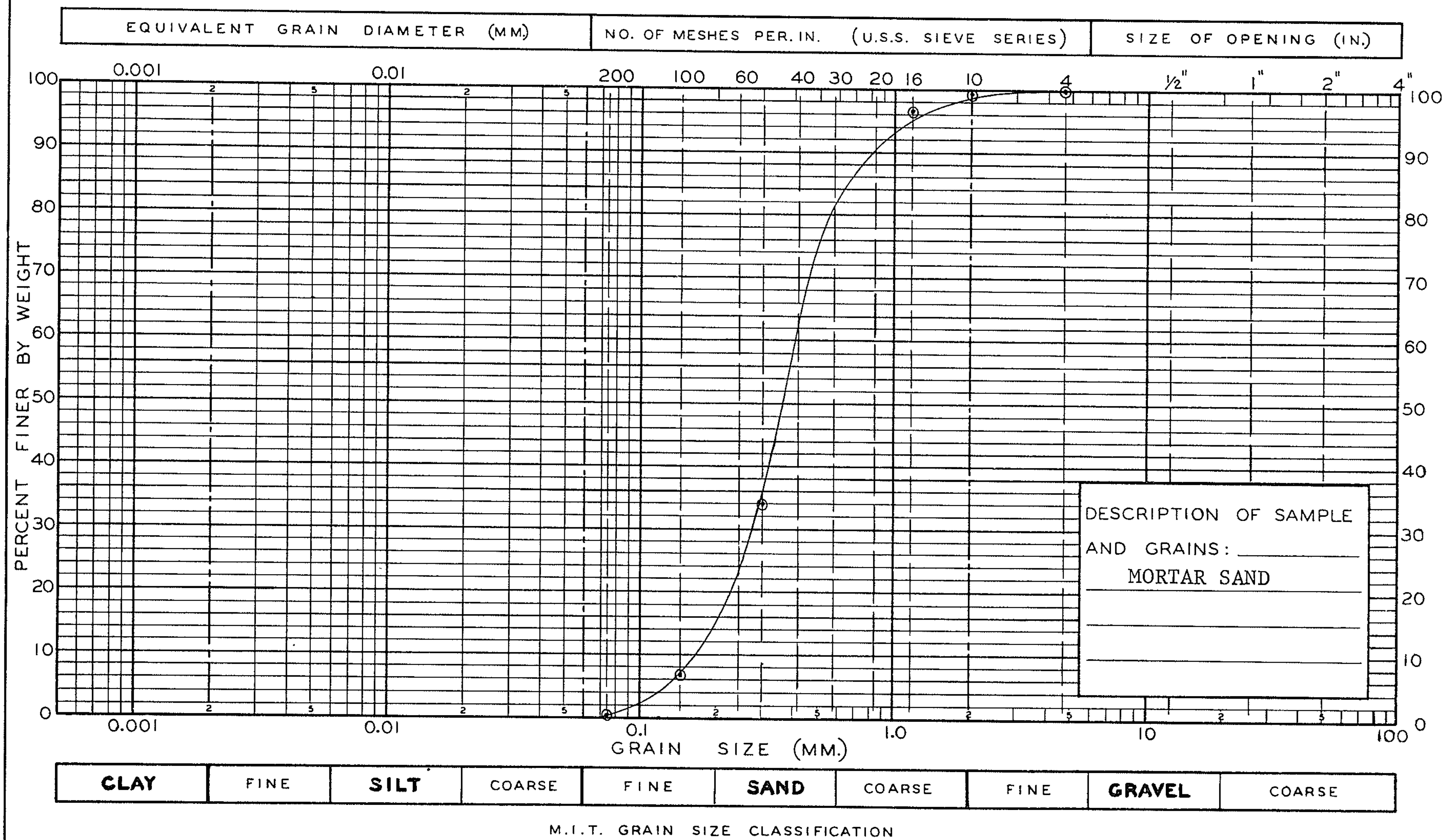
The concrete for Series 1 beams was made in a counter current rapid cement mixer at the laboratory. Each mixture contained 195 lb. coarse aggregate, 195 lb. fine aggregate, 62 lb. normal portland cement, and 47 lb. water. The slump ranged from 7 to 8 inches. The maximum size of coarse aggregate was 3/8 inches. A sieve analysis was carried out on the fine aggregate and it was graded as a medium-to-coarse sand. The results are listed in Table 5 and plotted in Figure 27.

TABLE 5

SIEVE ANALYSIS OF FINE AGGREGATE FOR CONCRETE

SIEVE NO.	WEIGHT PASSING (GRAMS)	PER CENT PASSING
4	1,508	99.9
10	1,410	93.6
16	1,173	77.7
50	189	12.5
100	44	2.9
200	17	1.0

MECHANICAL ANALYSIS OF SOILS



PROJECT: CONCRETE BLOCK-FORMED BEAMS	SAMPLE NO. 1	**SOIL MECHANICS LABORATORY**
PLOTTED: D.A.B. DATE: FEB./72	REMARKS: FINE-MEDIUM SAND	DEPARTMENT OF CIVIL ENGINEERING UNIVERSITY OF MANITOBA
CHECKED: DATE:		FORT GARRY MANITOBA

FIG. 26 SIEVE ANALYSIS OF MASONRY SAND

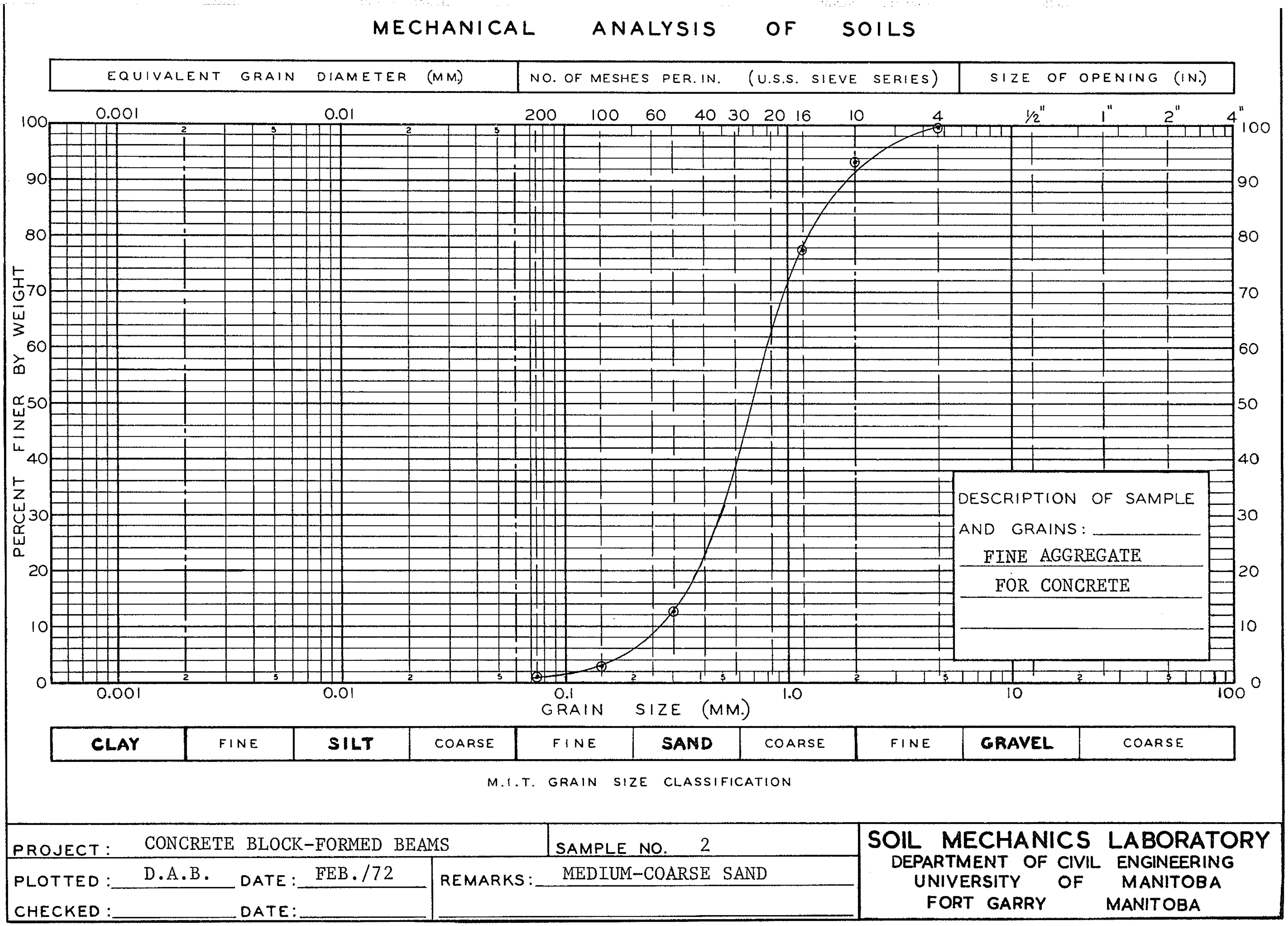


FIG. 27 SIEVE ANALYSIS OF FINE AGGREGATE FOR CONCRETE

The ready-mixed concrete used in all Series 2 beams and Beam 1A of Series 1 had a specified strength of 3,000 p.s.i., slump of 5 inches, and 1/2 inch maximum size of aggregate.

Compression tests were performed on 12" x 6" diameter sample cylinders taken from each pour. These results are listed in Table 6.

4. REINFORCING STEEL

Intermediate grade reinforcing steel was ordered from a local manufacturer. Three bar sizes were used -- #4, #8 and #9. All bars were inspected to make sure that they were free from oil, grease, scale or other foreign material prior to installation.

Three 24-inch sample coupons of each size were specified with each delivery. A tensile test was performed on each coupon to establish the yield strength of the steel to be used in the analysis of the beams. These values are listed in Table 7.

T A B L E 6

CONCRETE CYLINDER TESTS

BEAM NO.	TEST NO.	TOTAL LOAD (POUNDS)	COMP. STRESS (P.S.I.)	AVG f'c (P.S.I.)	AGE (DAYS)
1	1	63,000	2,220	2,220	10
1A	1	96,000	3,390	3,500	21
	2	102,000	3,610		
	3	98,500	3,480		
	4	101,000	3,570		
	5	97,000	3,430		
2	1	84,000	2,970	3,630	53
	2	117,000	4,140		
	3	107,000	3,780		
2A,3A	1	69,000	2,440	2,220	9
	2	71,000	2,510		
	3	45,000	1,590		
	4	64,000	2,260		
3	1	103,000	3,640	4,140	49
	2	125,000	4,420		
	3	106,000	3,740		
	4	124,000	4,390		
	5	127,000	4,490		
4	1	104,000	3,680	3,750	29
	2	99,500	3,520		
	3	114,500	4,050		
5	1	76,000	2,680	2,670	10
	2	76,000	2,680		
	3	75,500	2,670		
	4	76,000	2,680		
	5	75,000	2,650		
6,9	1	73,000	2,580	2,370	8
	2	61,000	2,160		
6B	1	100,000	3,530	3,530	35
7	1	70,500	2,490	2,490	13

TABLE 6

(continued)

BEAM NO.	TEST NO.	TOTAL LOAD (POUNDS)	COMP. STRESS (P.S.I.)	AVG f'c (P.S.I.)	AGE (DAYS)
7A,8A	1	94,000	3,320	3,320	7
8	1	78,000	2,760	2,640	10
	2	71,000	2,510		
9A	1	84,000	2,970	2,970	24
9B	1	102,000	3,610	3,360	7
	2	84,000	2,970		
	3	99,000	3,500		

TABLE 7

TENSILE STRENGTH OF REINFORCEMENT

BAR SIZE	TEST NO.	YIELD POINT LOAD (POUNDS)	YIELD POINT STRESS (P.S.I.)	YIELD POINT AVG. STRESS (P.S.I.)	ULTIMATE LOAD (POUNDS)	ULTIMATE STRESS (P.S.I.)	ULTIMATE AVG. STRESS (P.S.I.)	AVG. ELONG. IN 8 IN. (PER CENT)
SERIES 1								
#4	1	11,400	57,000	54,770	16,700	83,500	80,900	20
	2	10,700	53,500		15,700	78,500		
	3	10,750	53,800		16,150	80,700		
#8	1	39,190	49,500	49,600	60,400	76,500	76,570	24
	2	39,190	49,500		60,510	76,700		
	3	39,390	49,800		60,400	76,500		
SERIES 2								
#4	1	10,700	53,500	53,300	15,800	79,100	78,970	16
	2	10,650	53,300		15,750	78,800		
	3	10,600	53,100		15,800	79,000		
#9	1	51,000	51,000	50,950	75,800	75,800	75,070	24
	2	50,900	50,900		74,520	74,520		
	3	50,950	50,950		74,880	74,880		

V. THE TEST SET-UP AND PROCEDURE

In order to set each series of beams in their final testing position, it was first necessary to remove the 2" x 10" planks and supporting channel sections. This was accomplished by lifting each end of each beam and blocking it up on the end supports. The end supports consisted of two 10 WF 100 steel sections which were supported on two 48 WWF S-279 beams anchored on steel frames in a concrete pit.

Just prior to testing, each end of each masonry beam was again lifted to remove the wood blocking and insert the proper bearing supports. The beams were lifted by an overhead crane in one of two ways: by wrapping a chain around the beam, or by hooking a chain to lifting hooks provided in the beam. In either case, only one end of the beam was raised at a time. The lifting point was about four feet from the end of the beam.

The bearing supports consisted of steel rollers sandwiched between 1-inch steel plates. One 1-inch roller was used at one end and five 3/8-inch rollers at the other. 1/8-inch plywood cushions were placed on top and bottom of the steel plates to distribute the end reactions uniformly.

Loads were applied at the third points with hydraulic jacks. The jacks used had capacities of 400, 200, and 20 kips. At each loading point, a 1-inch steel plate was set level into a thin plaster capping on top of the beam. The jacks were then accurately positioned on the plates. Directly above each jack, a load cell was bolted to a steel frame. The load cells used had capacities of 200 and 20 kips. Each frame consisted of two 12 WF 53 columns and four 15 [40.0 cross ties. A diagram of the test set-up is shown in Figure 28.

As load was applied the jacks pushed against the load cells which were connected to separate strain indicators. Each load cell was pre-calibrated. For example, an increment of 50 micro-inches on the strain indicator corresponded to an applied load of 100 pounds. Thus, load increments were attained by presetting the strain indicators to certain readings and then pumping the hydraulic jacks until these readings were balanced to zero. Photographs of a beam in testing position, an hydraulic jack and load cell, and a "Budd" strain indicator are shown in Figures 29, 30 and 31, respectively.

At each load increment, vertical deflections were measured at the load points and midspan, using "Mercer" dial gauges set either under or on top of the beam. These gauges read to the nearest thousandth of an inch and had maximum travels of 1 inch (at the load points) and 2 inches (at midspan).

Loads were always applied symmetrically at the two load points. As cracks formed in the beams their locations and corresponding load were marked with ink to increase their visibility.

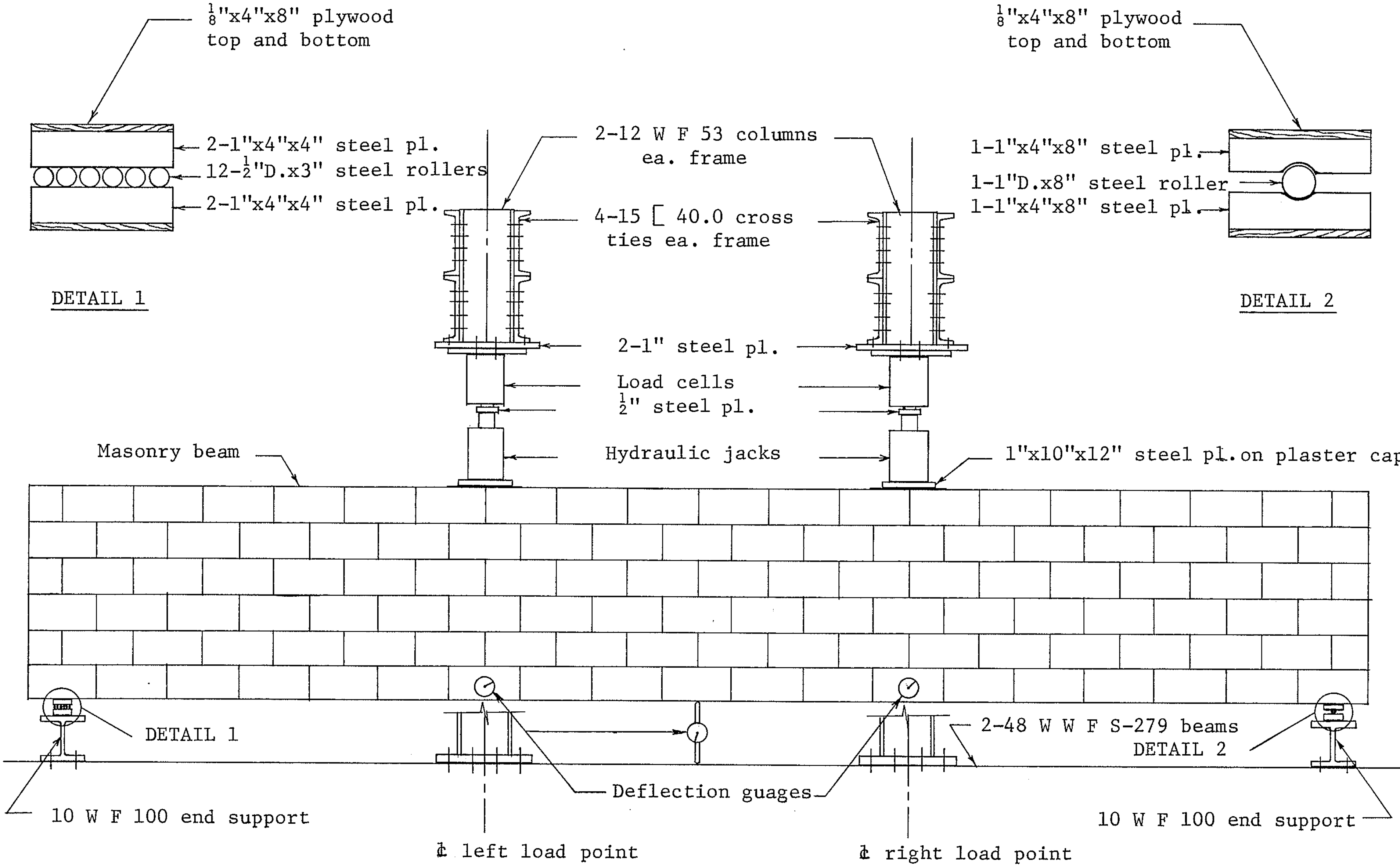


FIG. 28 THE TEST SET UP (SCALE: ⅜" = 1'-0)

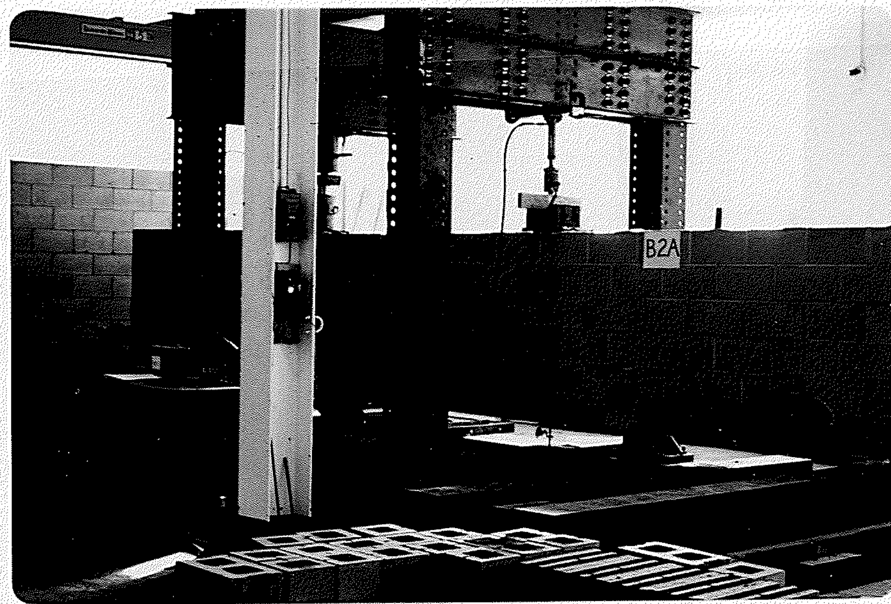


FIG. 29 MASONRY BEAM IN TESTING POSITION

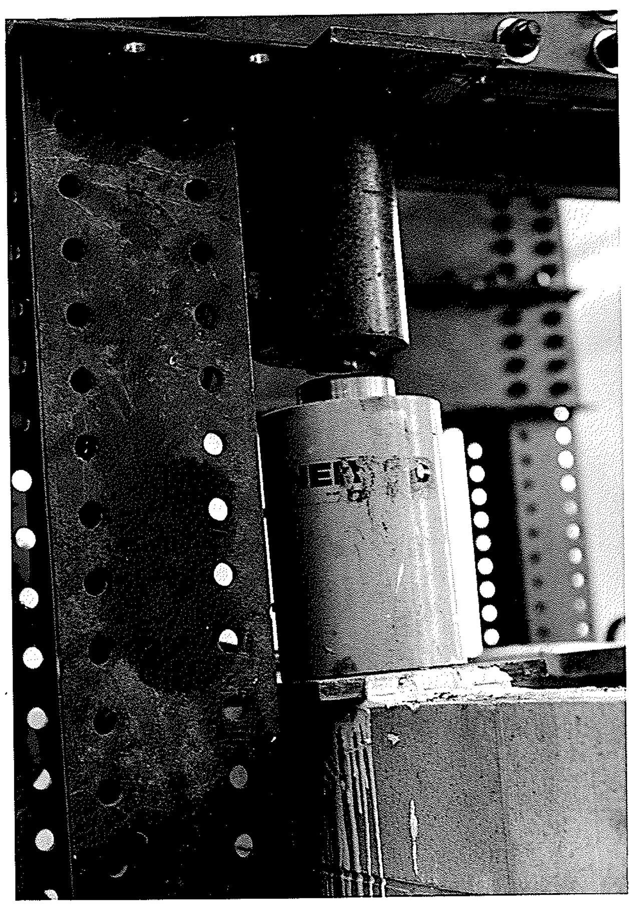


FIG. 30 HYDRAULIC JACK AND LOAD CELL

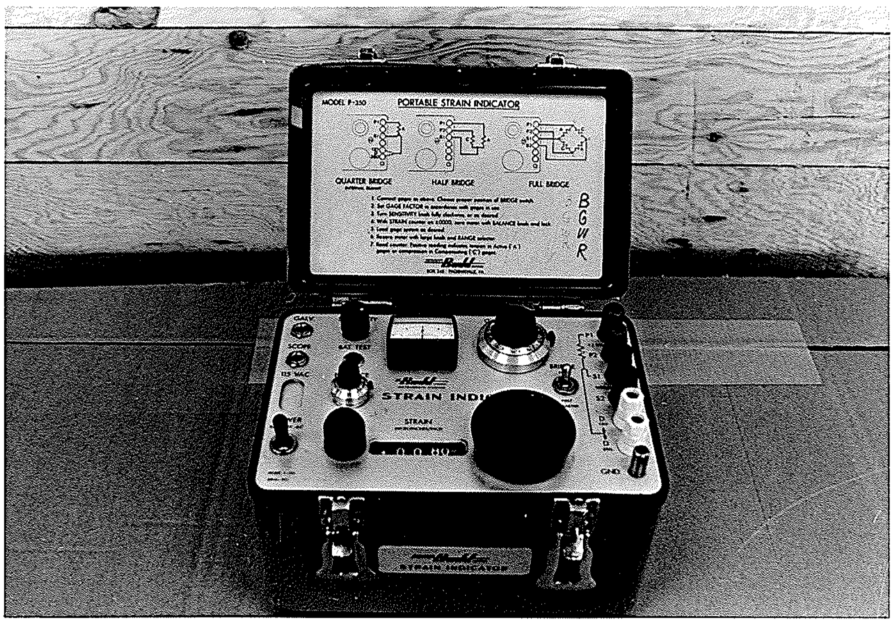


FIG. 31 "BUDD" STRAIN INDICATOR

VI. TEST RESULTS

The behaviour of all the beams under load followed a general pattern. At first under low loads only slight deflections were recorded. Deflections at the two load points were virtually the same while that at midspan was slightly higher. Normally at about 30% of the ultimate load small cracks were first noticed in the mortar joints along the bottom of the beam. As the load was further increased, these cracks began to extend upward into the beam.

From this point on, various types of failure took place. However, for the great majority of beams tested, this consisted of diagonal shear cracks which started near one of the end supports and extended up to the load point. These cracks were partially step-like following the mortar joints but also cut directly through the concrete blocks at some points. With the formation of these cracks, higher deflections were measured with each load increment.

At some load increments, a considerably large crack opened. This was accompanied by a decrease in load of up to 50%. As more load was applied, deflections became very large. Since the beam could not withstand further load but only continued to deflect, the test was then terminated.

The complete set of test results are listed in Tables 8 to 23 at the end of this section. The test results for each type of beam, core-filled, end-filled or totally filled are outlined below.

1. CORE-FILLED BEAMS

The first beam of this series, Beam 1, which had no cores filled, failed under its own dead load. The beam failed in longitudinal shear with the upper block courses having slid outward. This caused cracking in the mortar joints, especially in the end sections of the beam. Although the failure occurred before the placement of the deflection gauges, midspan deflection was estimated at 3 inches. A photograph of the "warped" beam is shown in Figure 32.

The addition of individual concrete cores in Beams 2 to 5 increased the load capacity. The cores provided a greater resistance to longitudinal shear stresses, preventing a sliding out of the end blocks. When the shear resistance of the concrete was surpassed, diagonal cracks developed in the beam.

The strongest beam of this series was Beam 3A which withstood an applied load of 11.7^K. Both repaired beams (Beams 2A and 3A) carried at least as much load as their originals (Beams 2 and 3). The top course concrete in Beams 4 and 5 appeared to offer little additional load carrying capacity, since shear stresses governed.

Photographs of these beam failures are shown in Figures 32 to 41. Load deflection curves are plotted in Figures 42 to 47.

2. END-FILLED BEAMS

In the Series 2 beams, the ends were entirely filled to improve the shear resistance. This resulted in an increase of ultimate loads

FIG. 32 BEAM 1

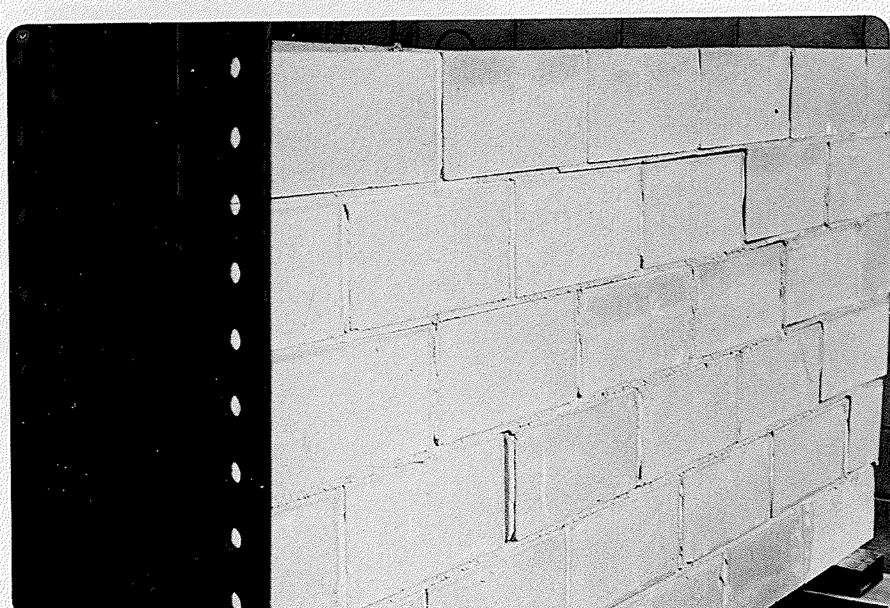


FIG. 33 HORIZONTAL SHEAR FAILURE IN END PORTION OF

BEAM 1

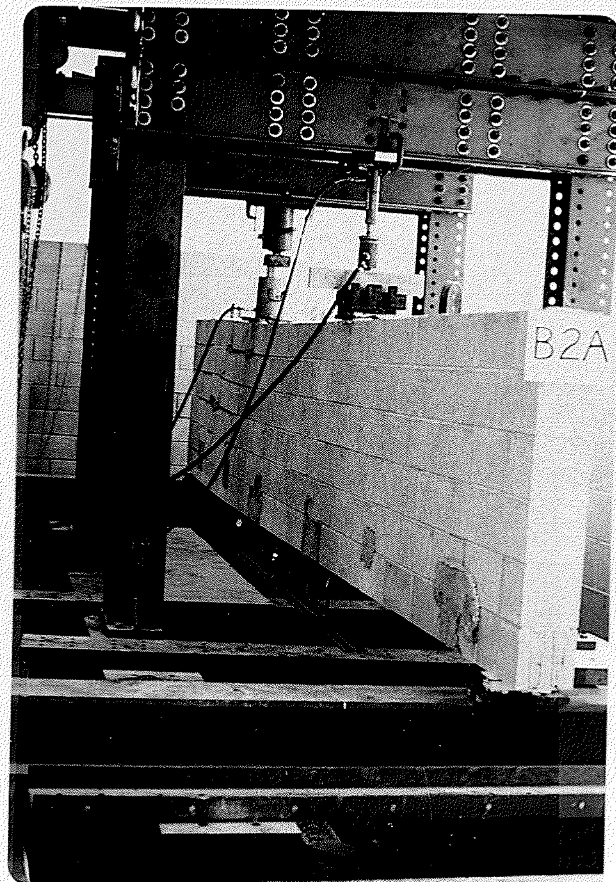


FIG. 34 BEAM 2A

FIG. 35 SHEAR FAILURE IN BEAM 2A

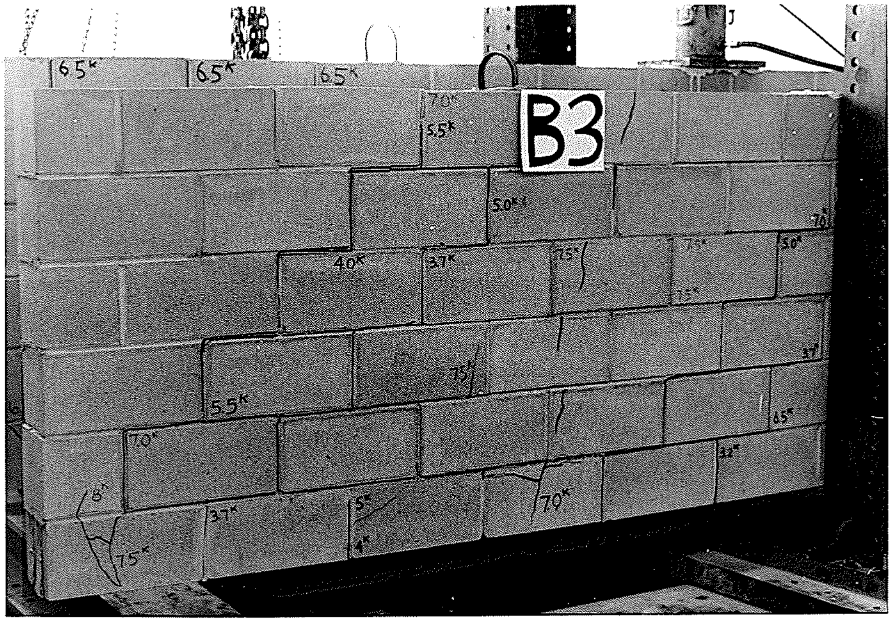


FIG. 36 SHEAR FAILURE IN BEAM 3

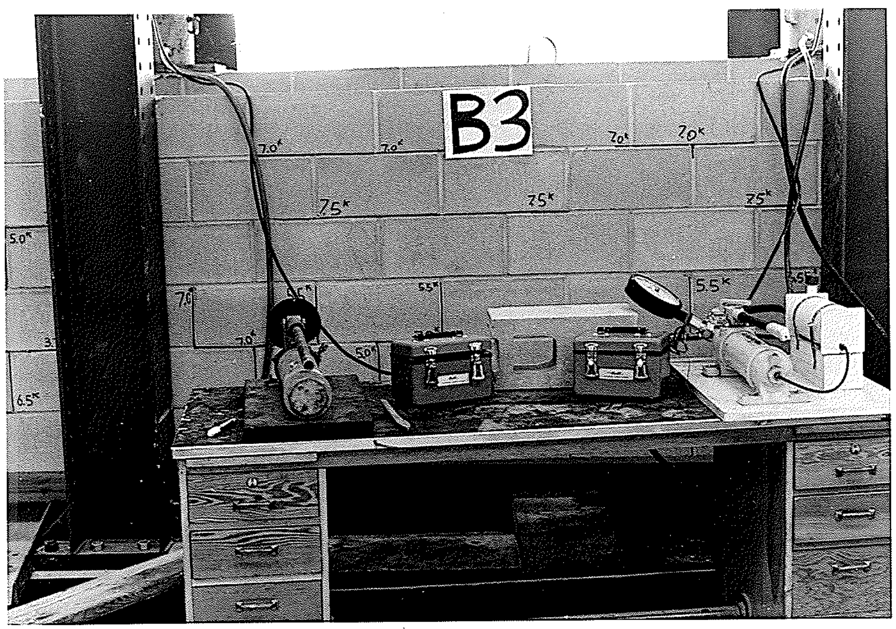


FIG. 37 CENTRE OF BEAM 3

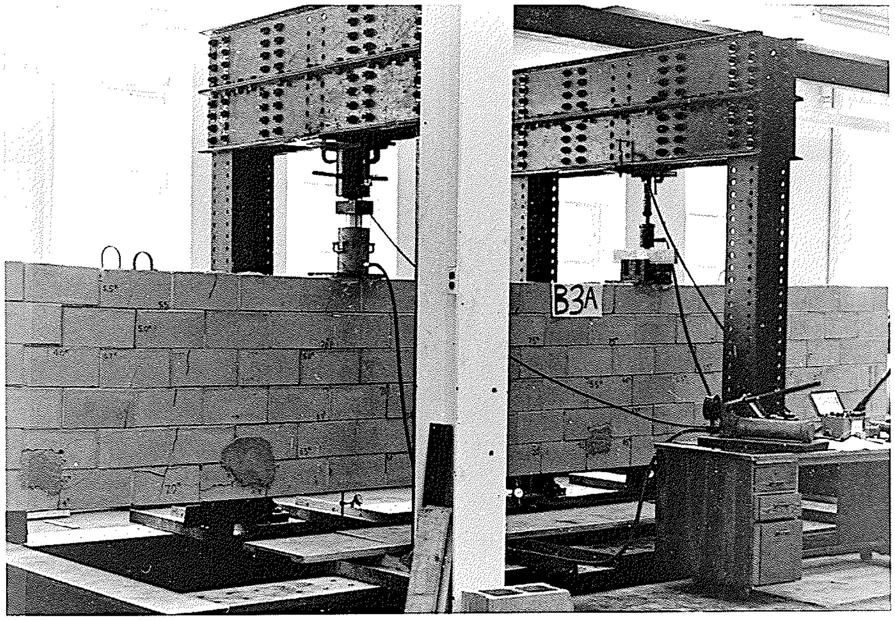


FIG. 38 BEAM 3A

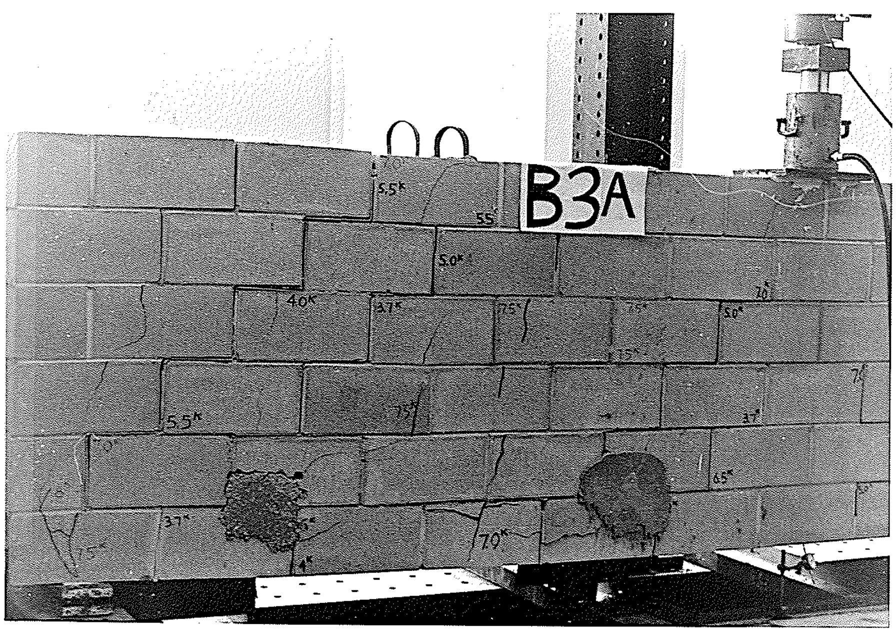


FIG. 39 SHEAR FAILURE IN BEAM 3A

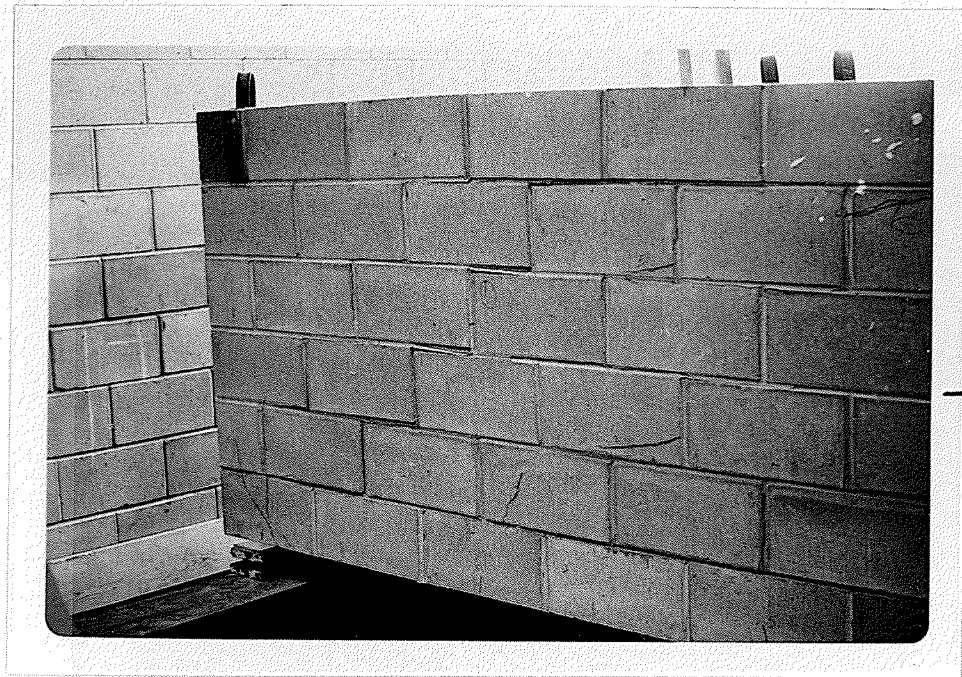


FIG. 40 SHEAR FAILURE IN BEAM 4

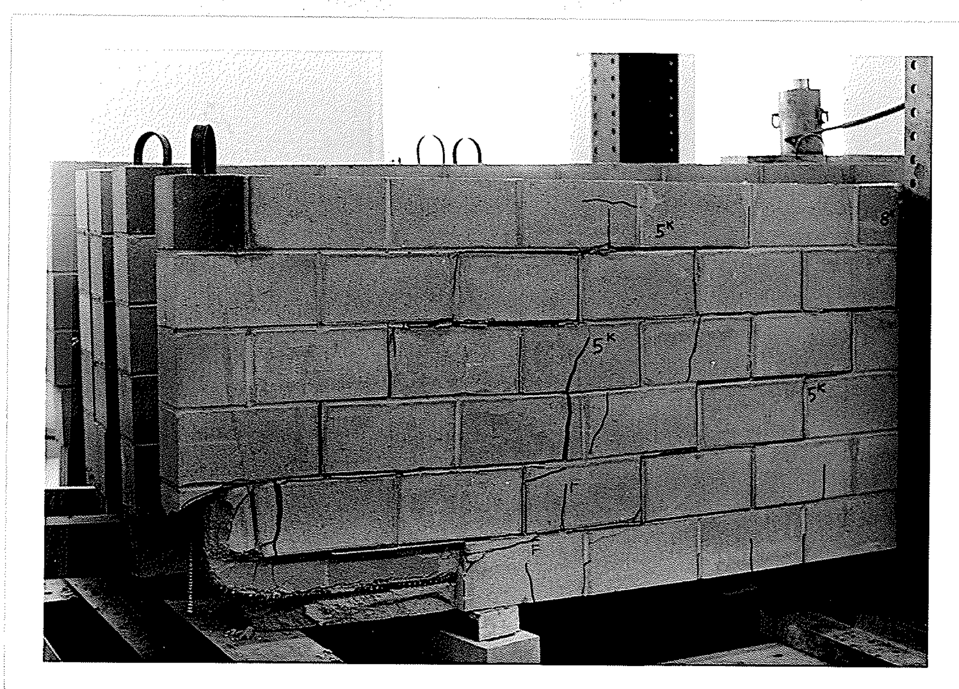


FIG. 41 SHEAR FAILURE IN BEAM 5

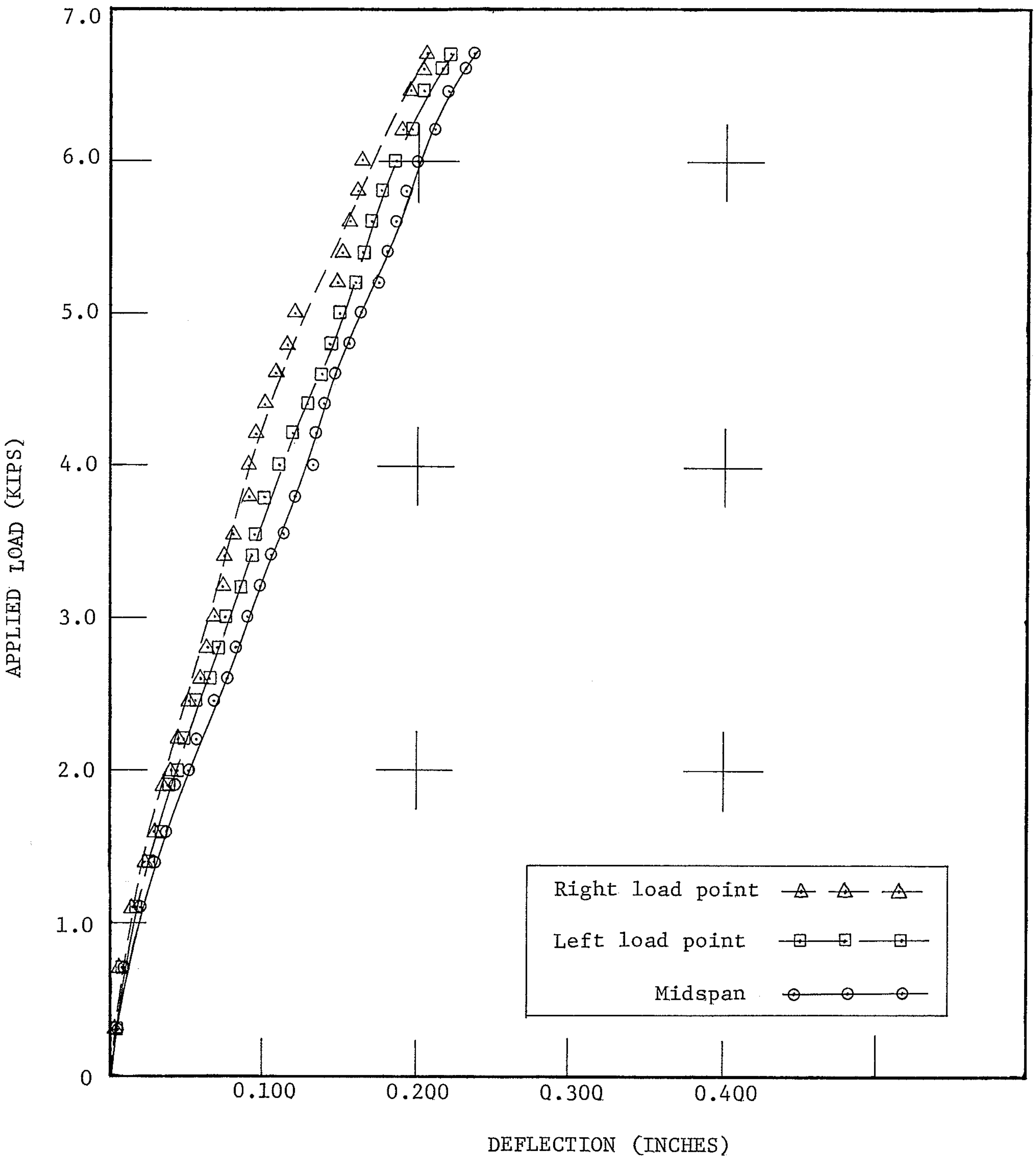


FIG. 42 LOAD-DEFLECTION CURVES FOR BEAM 2

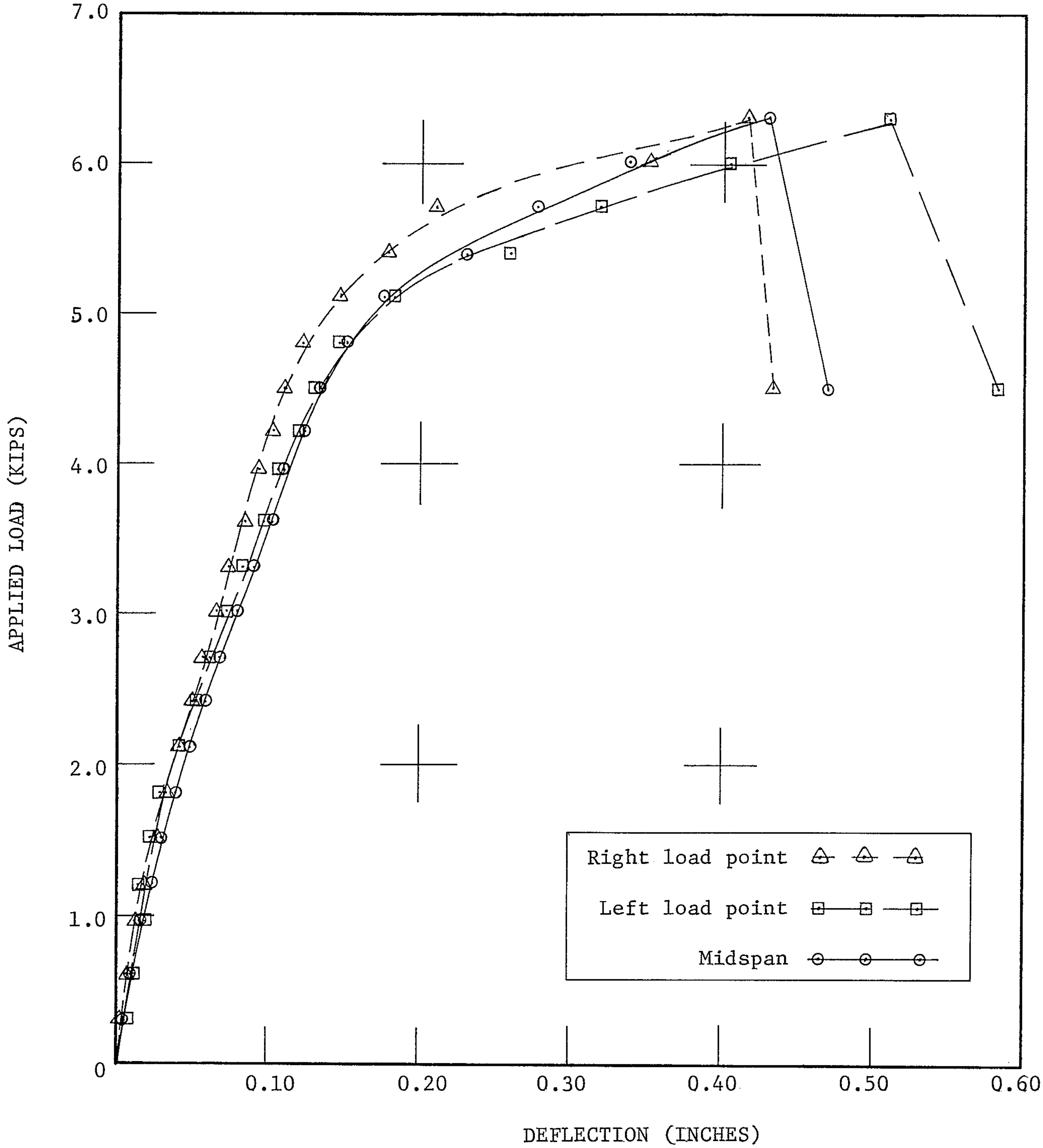


FIG. 43 LOAD-DEFLECTION CURVES FOR BEAM 2A

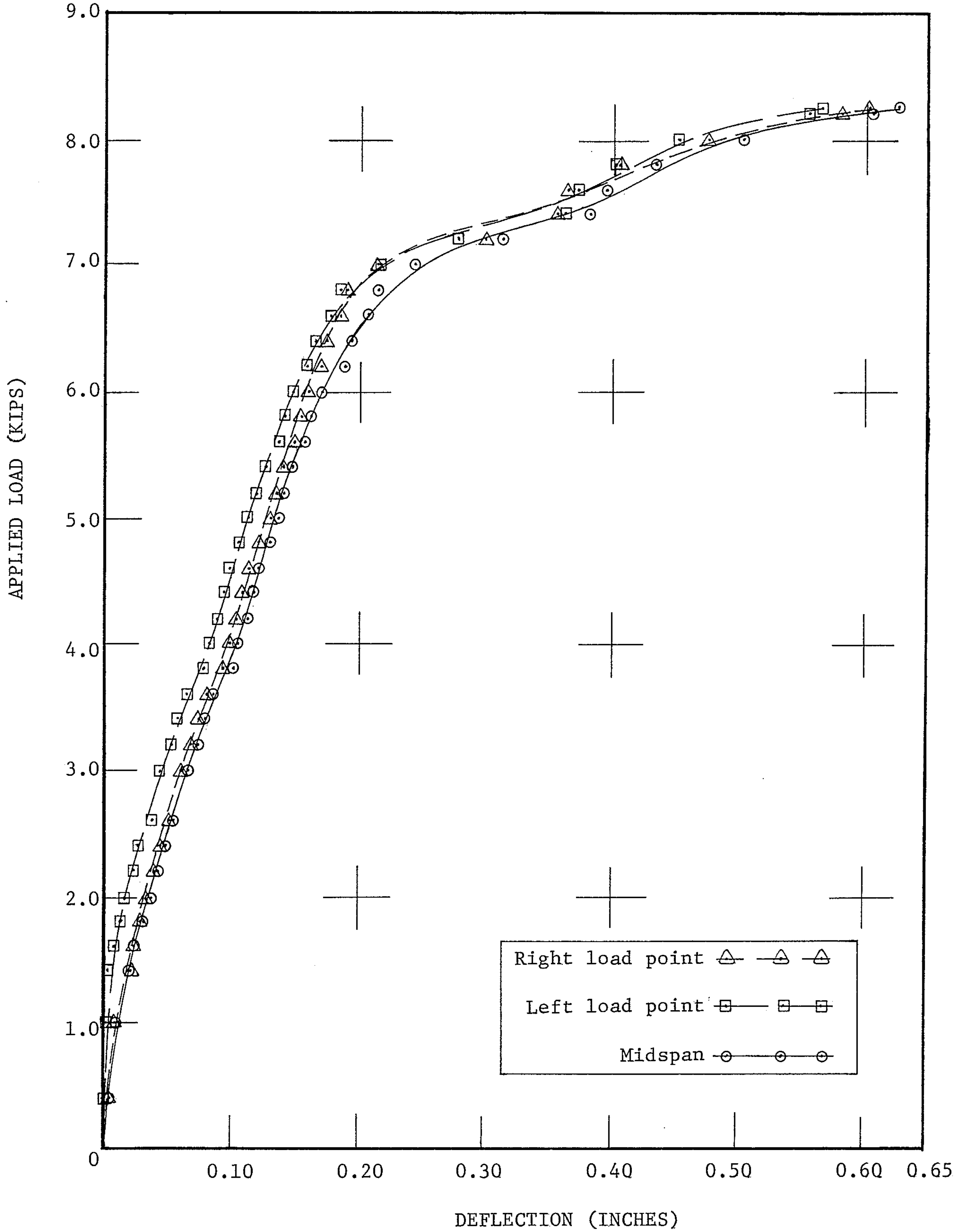


FIG. 44 LOAD-DEFLECTION CURVES FOR BEAM 3

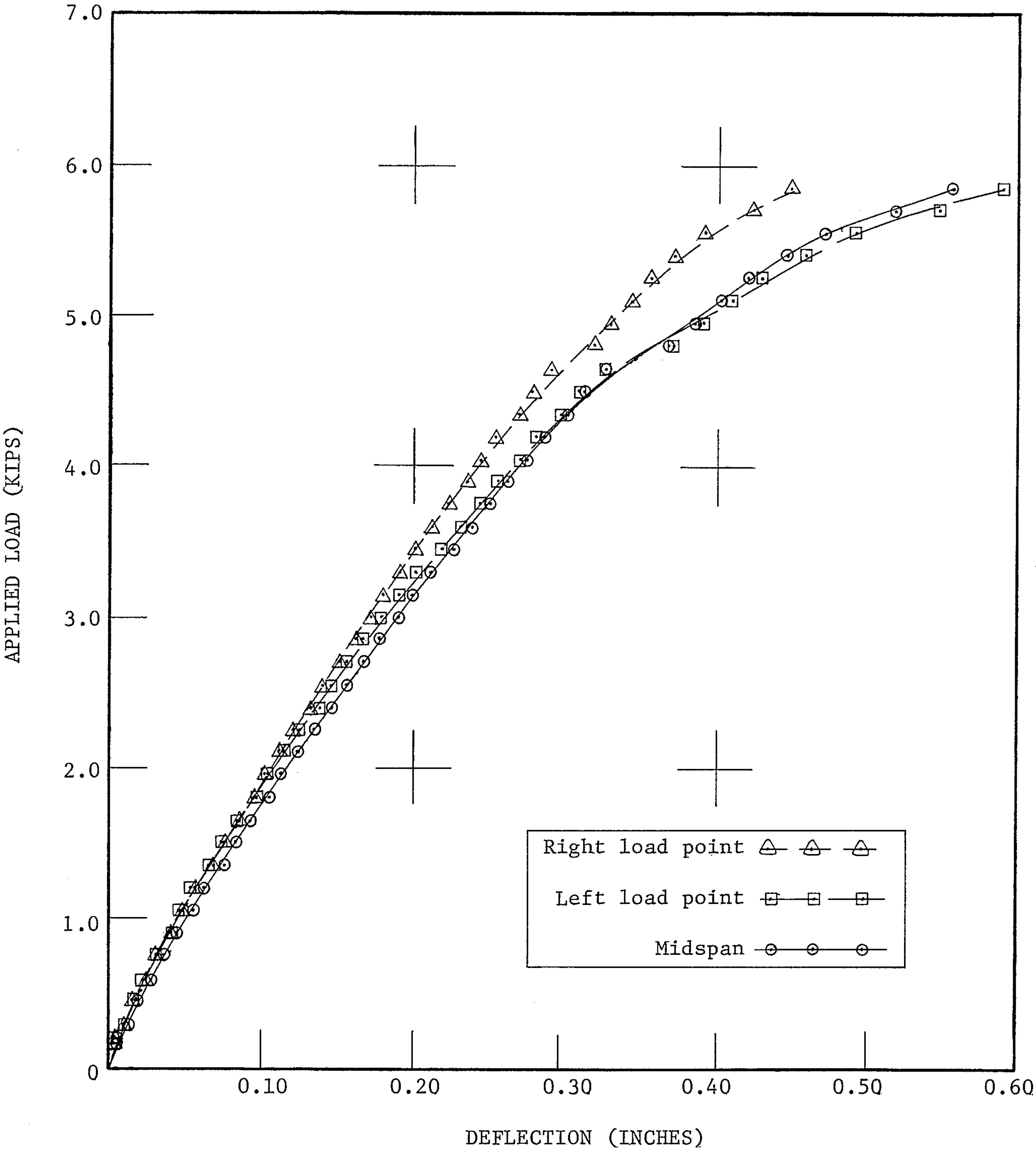


FIG. 45 LOAD-DEFLECTION CURVES FOR BEAM 3A

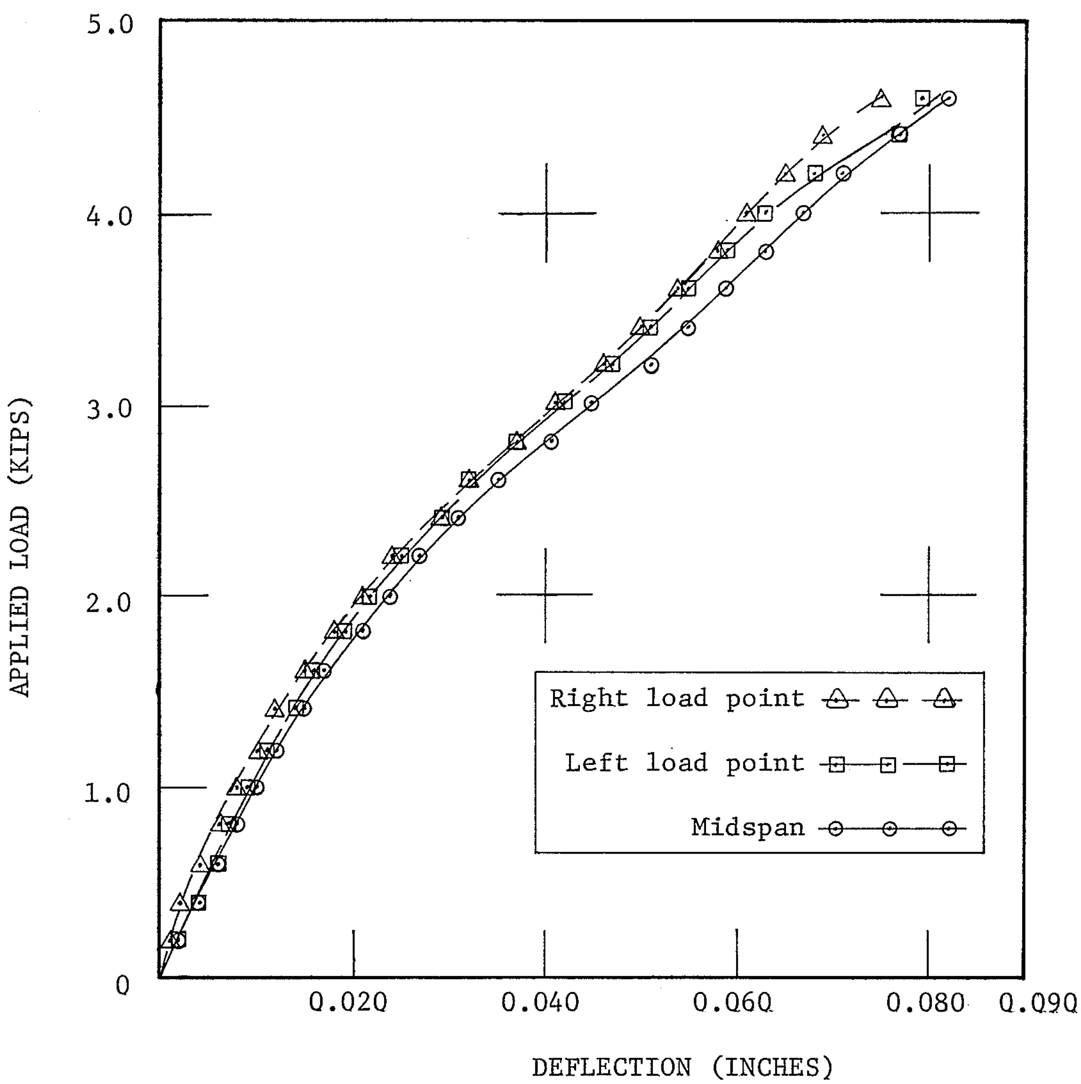


FIG. 46 LOAD-DEFLECTION CURVES FOR BEAM 4

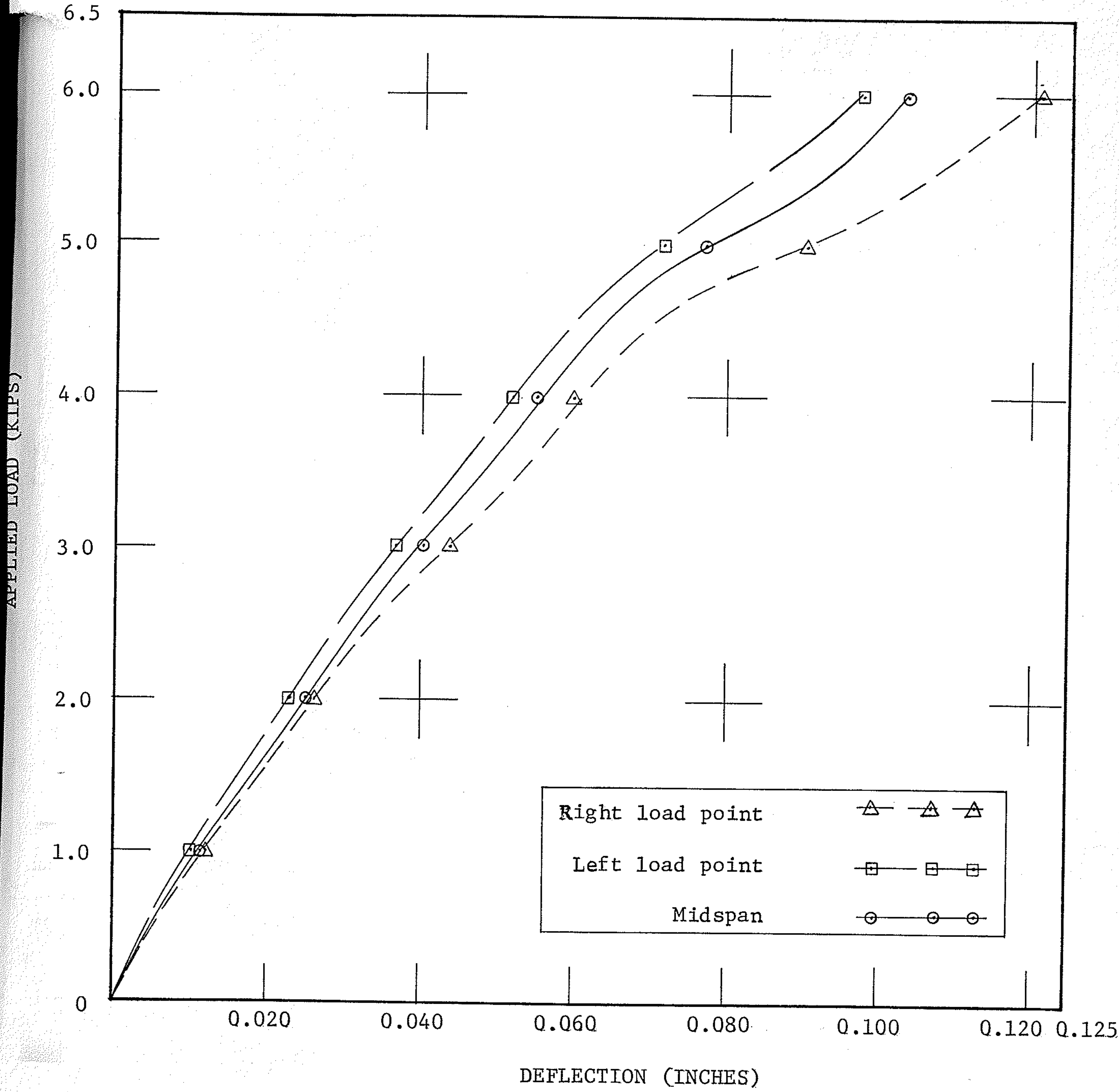


FIG. 47 LOAD-DEFLECTION CURVES FOR BEAM 5

up to 39 kips. In Beams 7, 8 and 9, the mode of failure was crushing of the blocks and mortar in the top two courses between load points. Midspan deflections were slightly higher than in the core-filled beams. Higher tension in the longitudinal steel caused the blocks in the centre of the bottom course to break apart at the mortar joints and drop out. Failure portions of these three beams are shown in Figures 48 to 51. Figure 52 shows the low slump concrete used in the right end of the Series 2 beams.

In Beams 7A, 8A and 9A, the top two courses were rebuilt stronger than before. Consequently, the ultimate load was higher. Both Beam 7A and 8A withstood compressive forces in the top layers. The mode of failure changed to shear cracks in the end zone that was filled with the low slump concrete. Failures of these beams are shown in Figures 53 to 57.

Near ultimate load some crushing in the central top blocks was noticed in Beam 9A. However, when more load was applied, a shear crack developed and later widened, again in the end zone which was filled last. Meanwhile, the top block courses remained intact. This beam carried 24 kips at each load point compared to 21 kips by its counterpart Beam 9.

Load-deflection curves for the end-filled beams are shown in Figures 58 to 63.

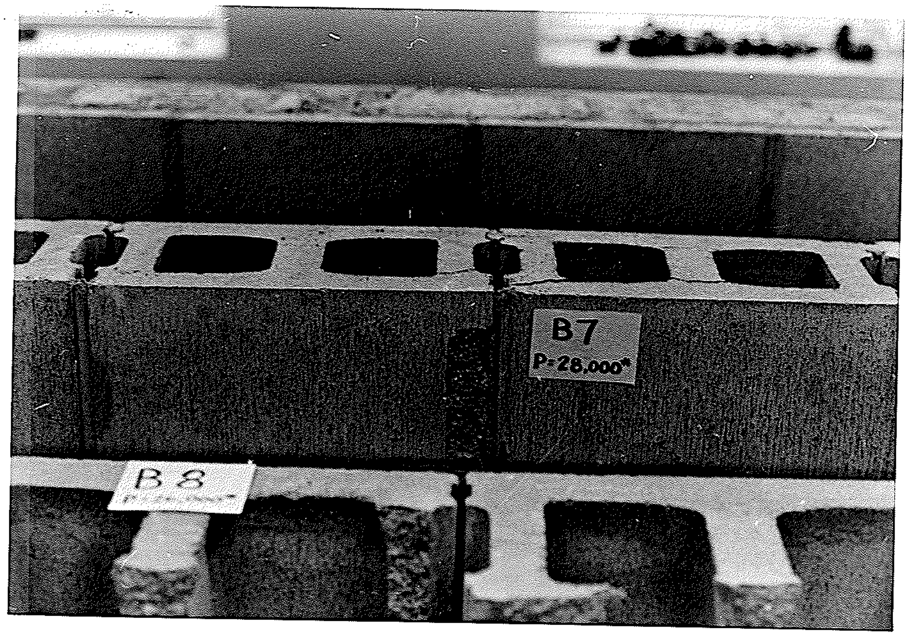


FIG. 48 BEAM 7 - FLEXURAL FAILURE BY CRUSHING OF BLOCKS

FIG. 49 BEAM 8 - INITIATION OF CRACKS IN TOP OF BLOCKS

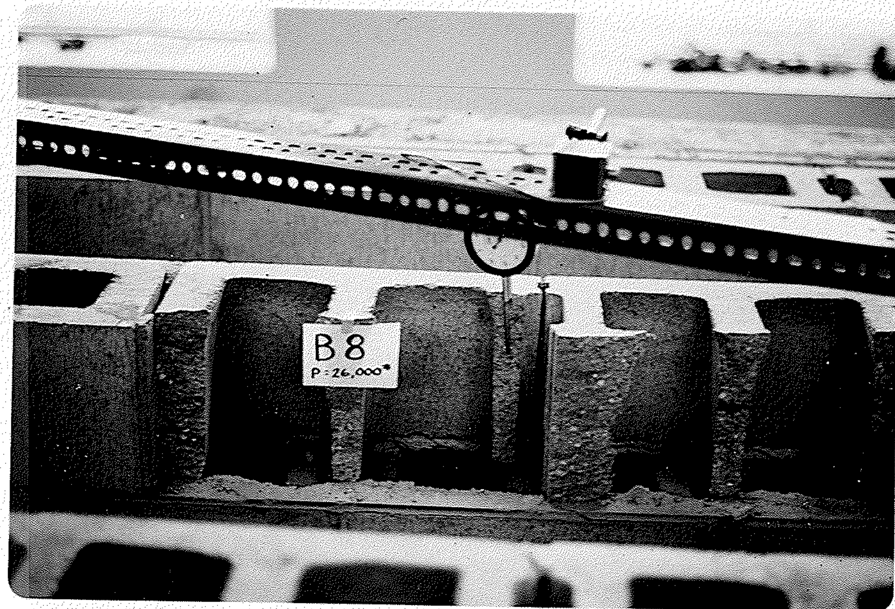


FIG. 50 BEAM 8 - FLEXURAL FAILURE BY CRUSHING OF BLOCKS

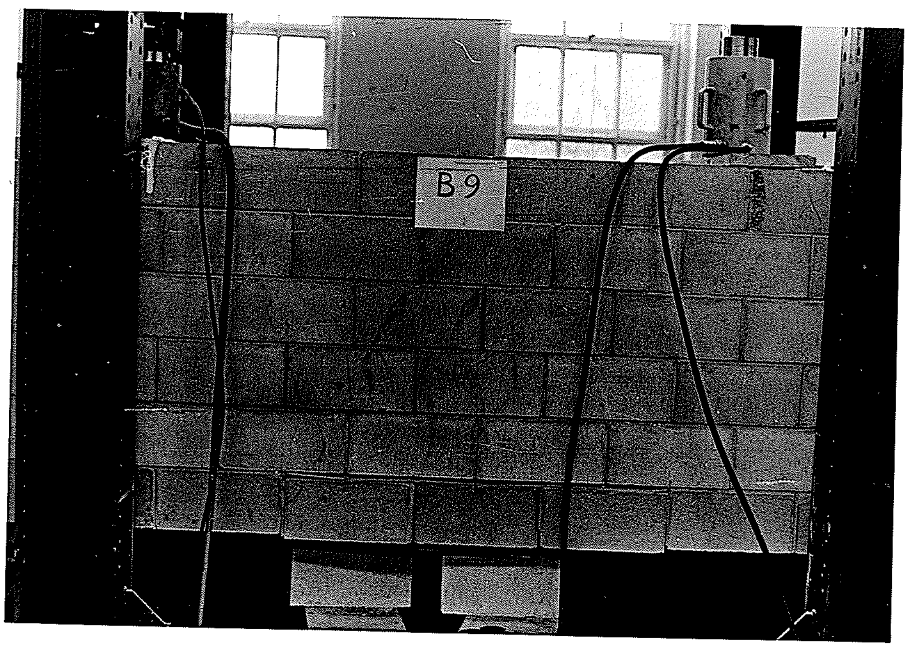


FIG. 51 CENTRE OF BEAM 9. TOP BLOCK AND MORTAR JOINT CRUSHED LEADING TO FLEXURAL FAILURE. BOTTOM BLOCKS FELL OUT DUE TO HIGH TENSILE FORCE

FIG. 52 POOR QUALITY CONCRETE IN RIGHT END OF SERIES 2 BEAMS

FIG. 53 BEAM 7A

FIG. 54 BEAM 8A

FIG. 55 SHEAR FAILURE IN BEAM 8A

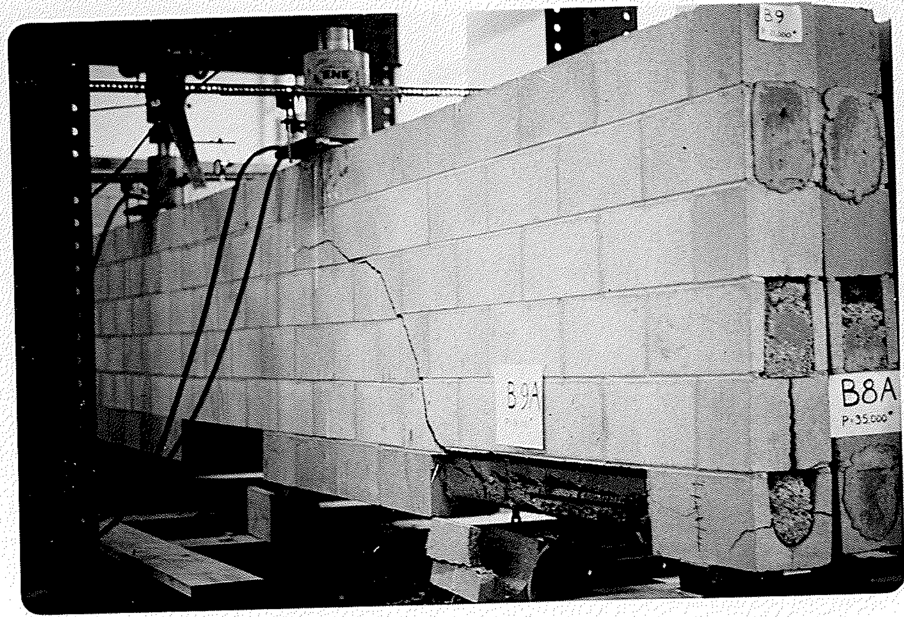


FIG. 56 SHEAR FAILURE IN BEAM 9A

FIG. 57 CLOSEUP OF SHEAR CRACK IN BEAM 9A.

NOTE LARGE VOID AREA ALONG REINFORCING BAR.

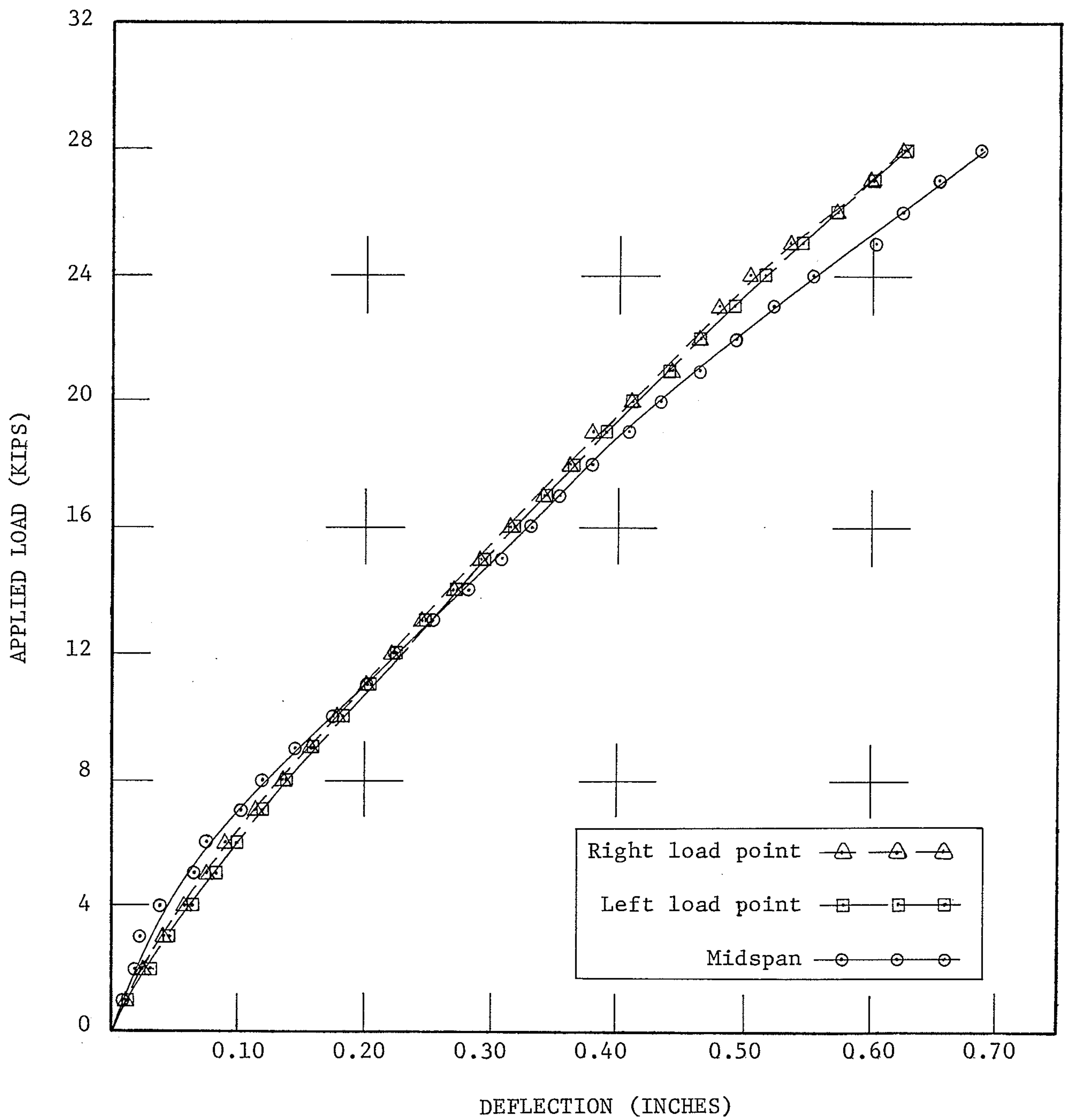


FIG. 58 LOAD-DEFLECTION CURVES FOR BEAM 7

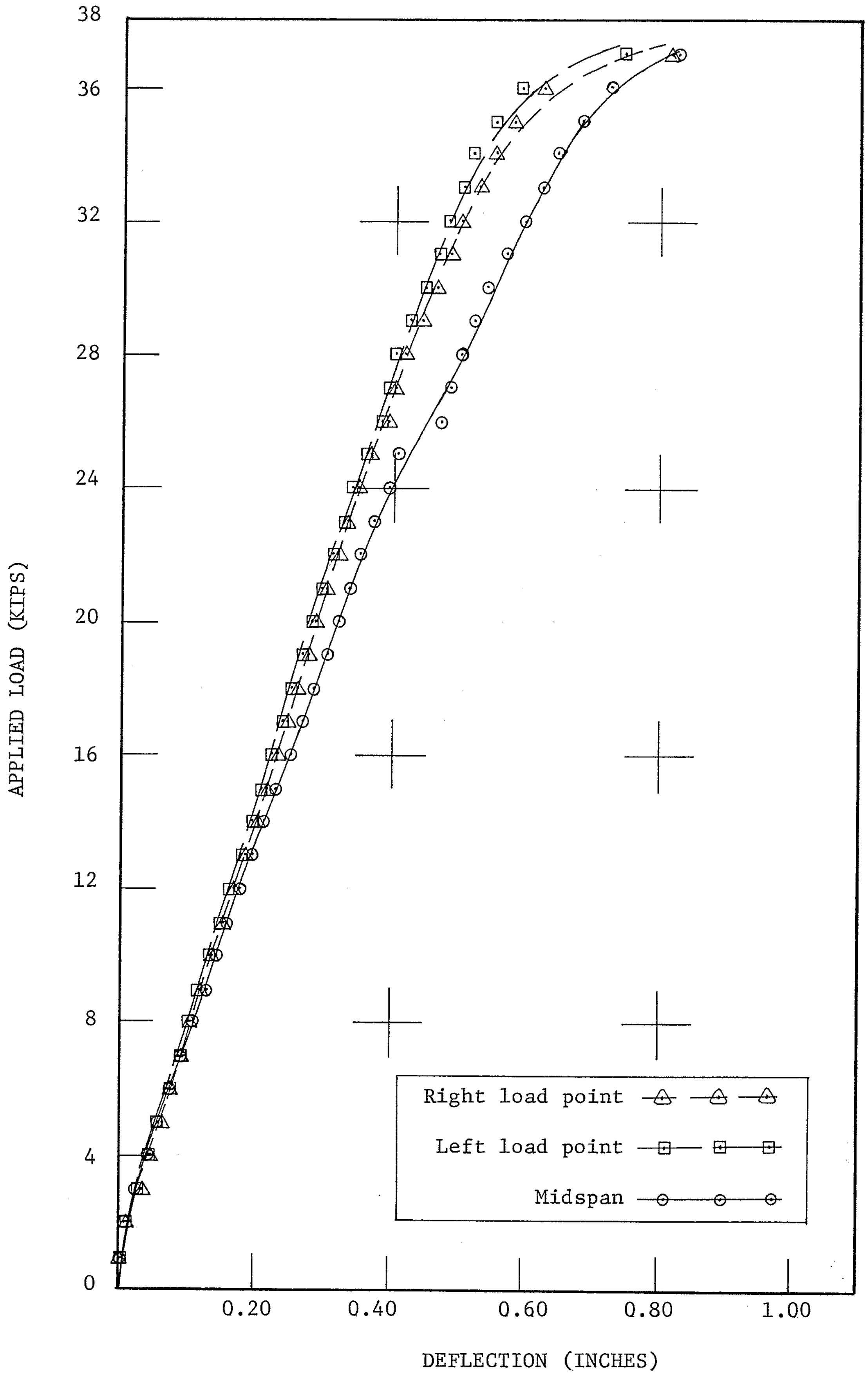


FIG. 59 LOAD-DEFLECTION CURVES FOR BEAM 7A

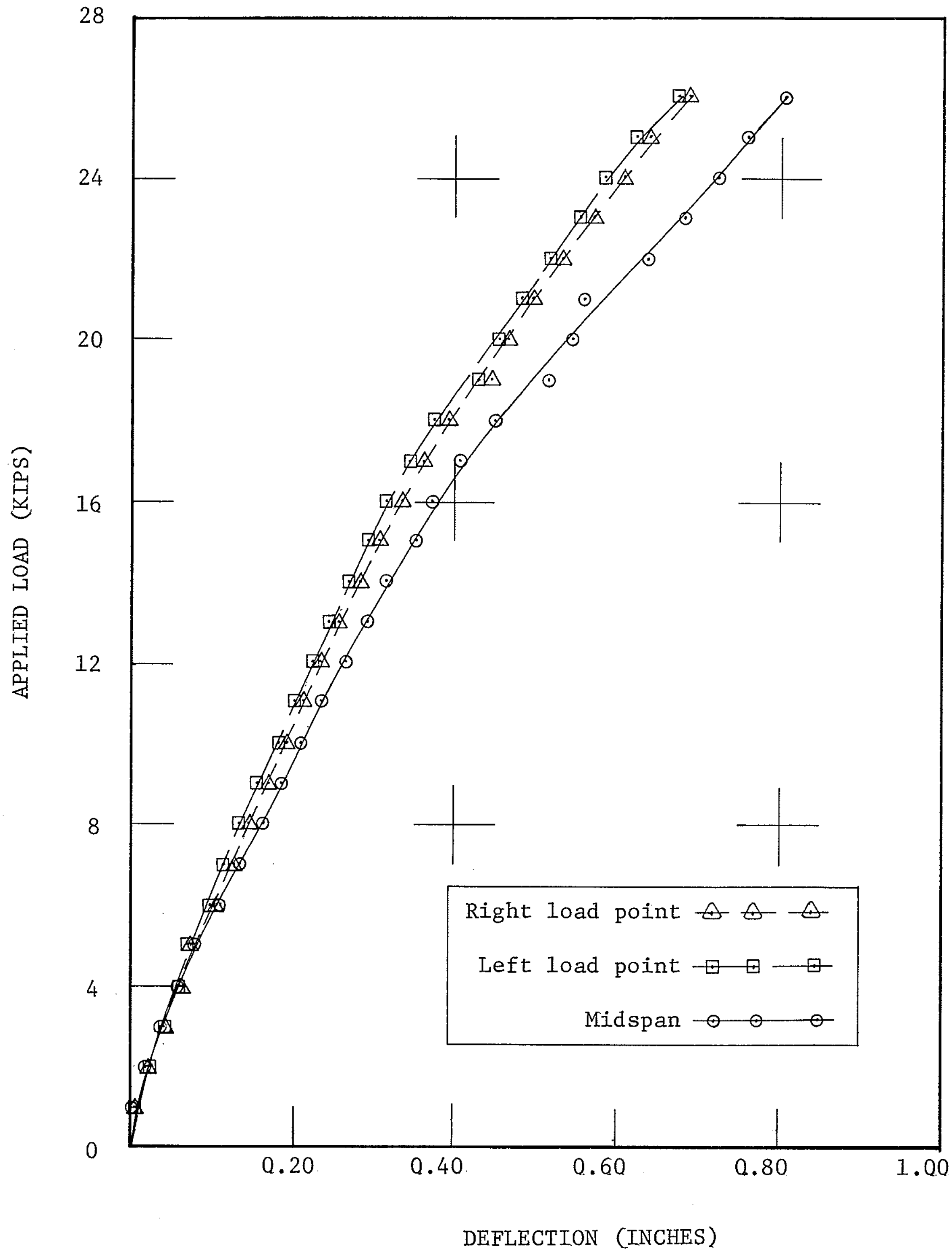


FIG. 60 LOAD-DEFLECTION CURVES FOR BEAM 8

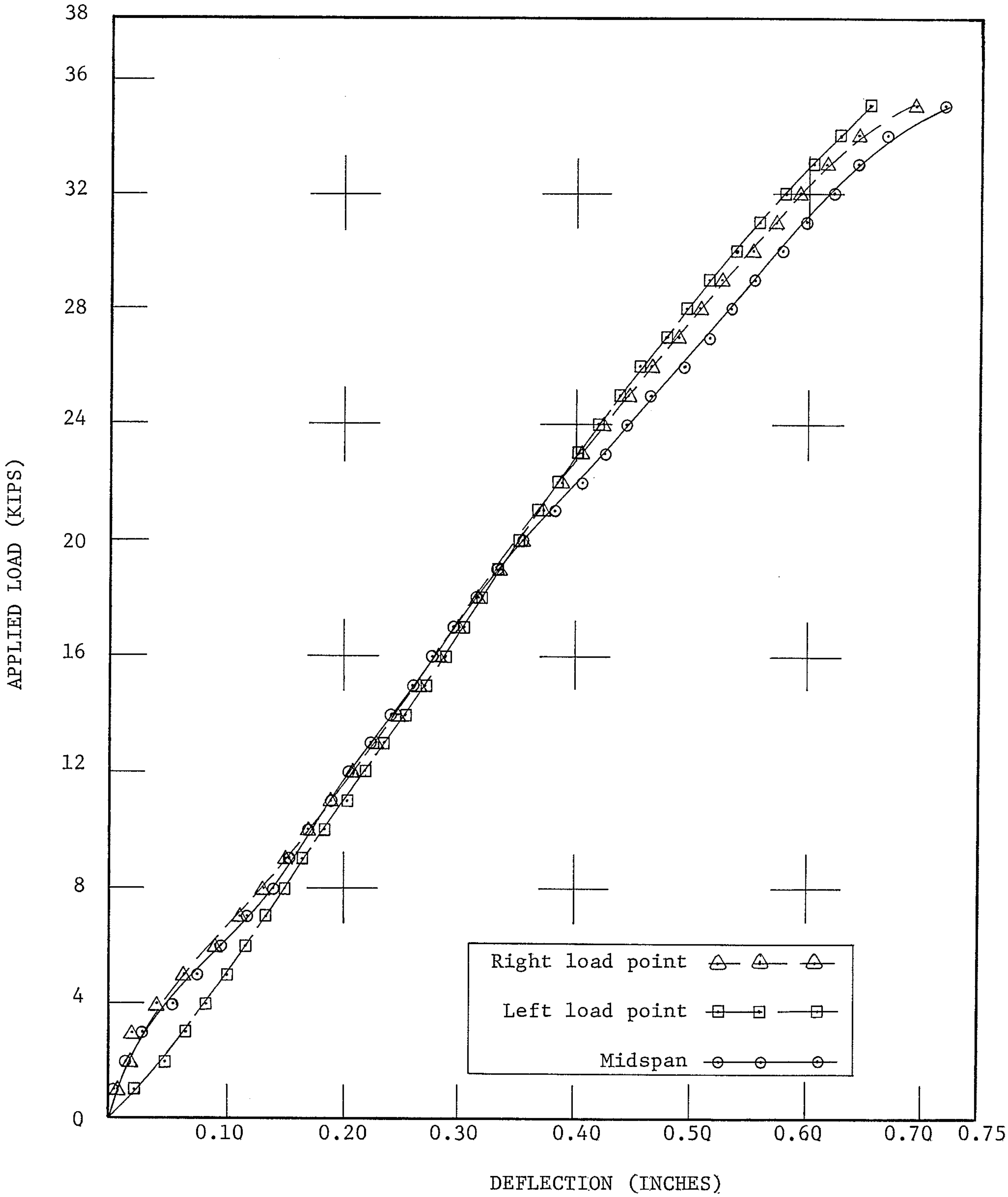


FIG. 61 LOAD-DEFLECTION CURVES FOR BEAM 8A

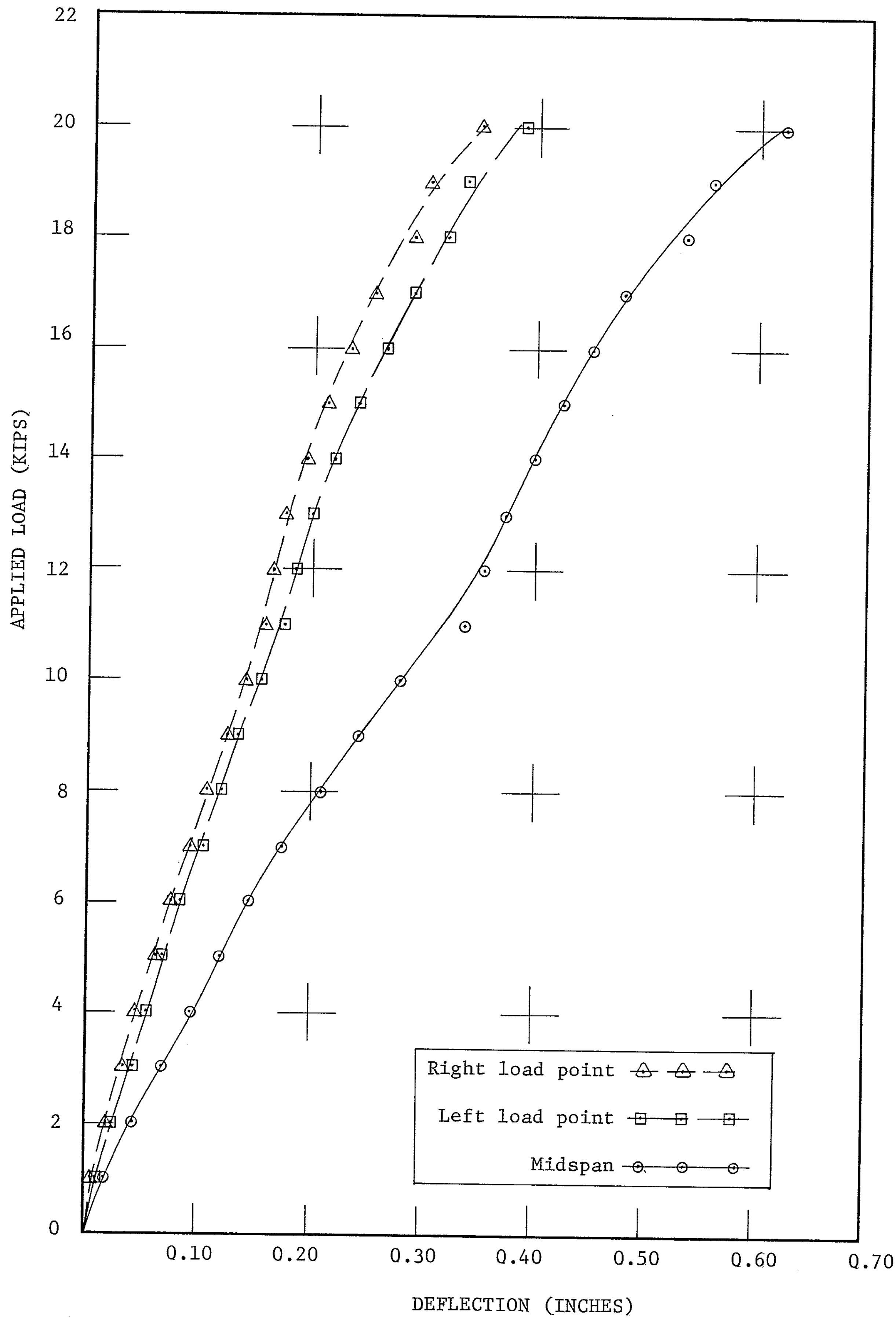


FIG. 62 LOAD-DEFLECTION CURVES FOR BEAM 9

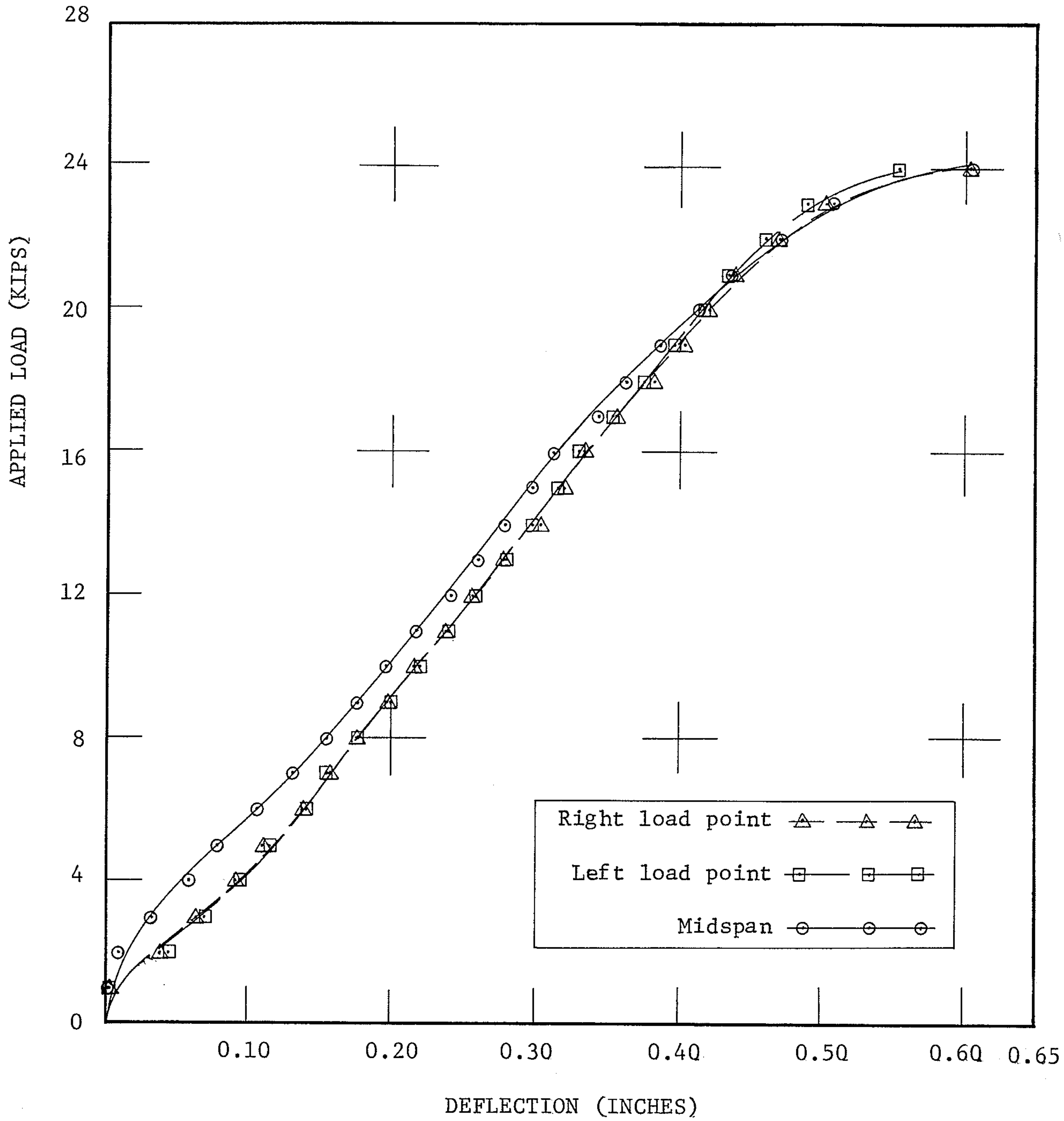


FIG. 63 LOAD-DEFLECTION CURVES FOR BEAM 9A

3. TOTALLY FILLED BEAMS

The two 24'-0 span solid beams, Beam 1A and Beam 6, both failed in shear. Beam 1A carried about the same amount of load as the core-filled beams of Series 1, and Beam 6 less than many of the end-filled Series 2 beams. However, Beam 6 withstood a substantially greater load than Beam 1A. Although Beam 1A was previously tested as Beam 1, the only substantial difference was that the bottom course in Beam 1A was poured separately.

The two 16'-0 span beams resisted the highest loads as was expected. Both developed shear and flexure cracks but the ultimate failure was a large diagonal shear crack from load point to support. Beam 9B with shear reinforcing every 48 inches outlasted Beam 6B to a load of 85 kips.

Photographs of these totally filled beams at failure are shown in Figures 64 to 70, while load-deflection curves are drawn in Figures 71 to 74.

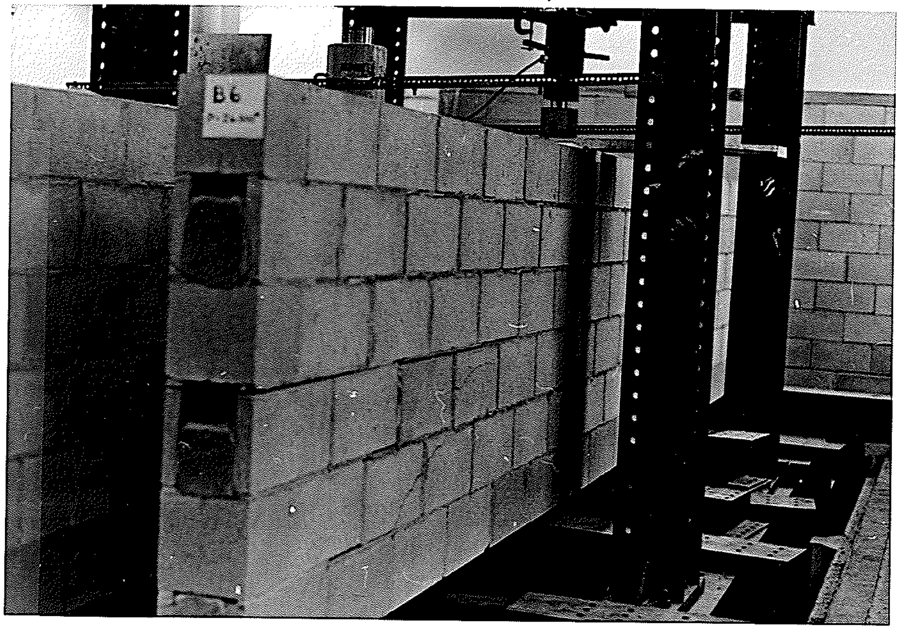


FIG. 64 BEAM 6

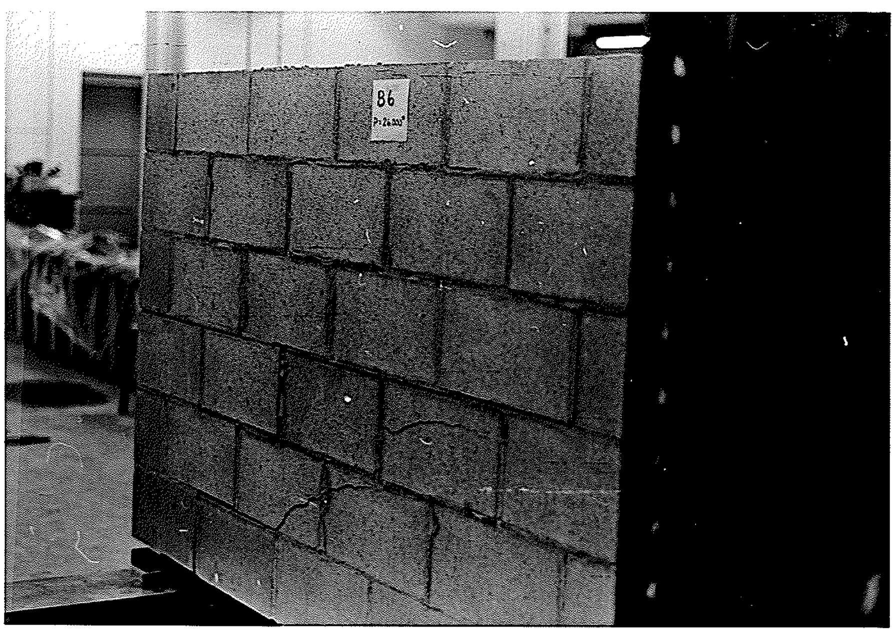


FIG. 65 SHEAR FAILURE IN BEAM 6

FIG. 66 FLEXURAL SHEAR FAILURE IN BEAM 6B

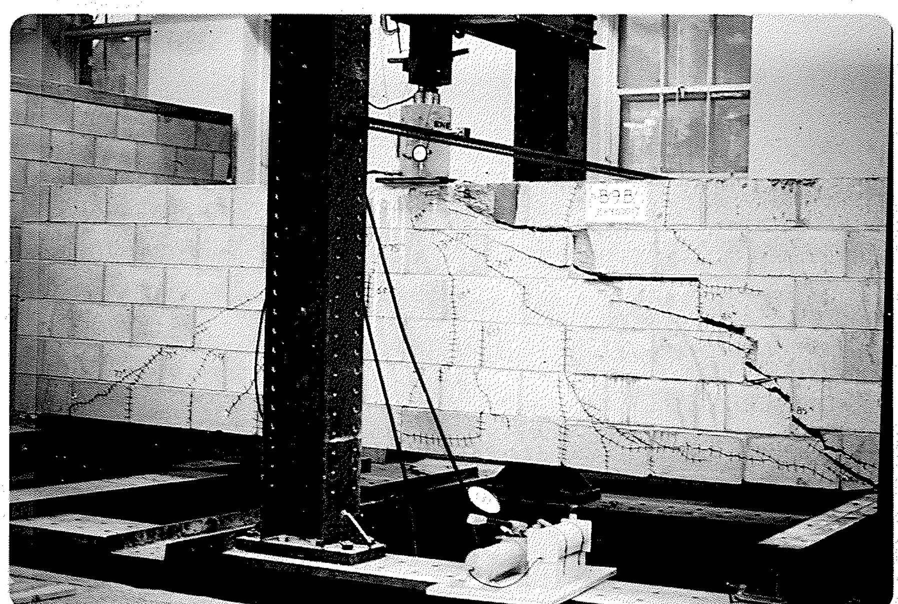


FIG. 67 FLEXURAL SHEAR FAILURE IN BEAM 9B

FIG. 68 CLOSEUP OF SHEAR CRACK AND CRUSHED CONCRETE IN TOP OF BEAM 9B

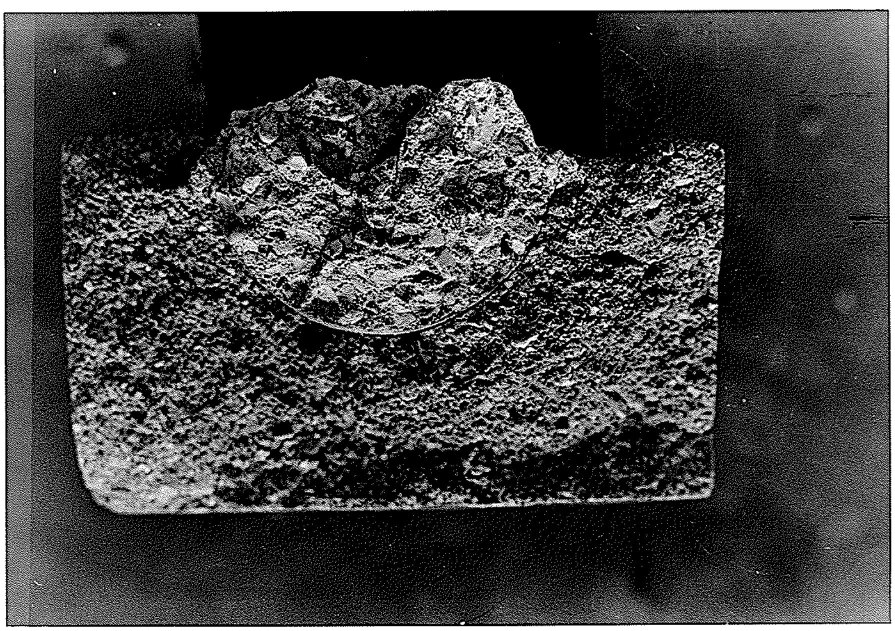


FIG. 69 CORED OUT PORTION FROM BOTTOM COURSE OF LEFT END OF BEAM 9B SHOWING GOOD BOND BETWEEN CONCRETE AND BLOCK. COMPARE WITH RIGHT END OF FIG. 52.

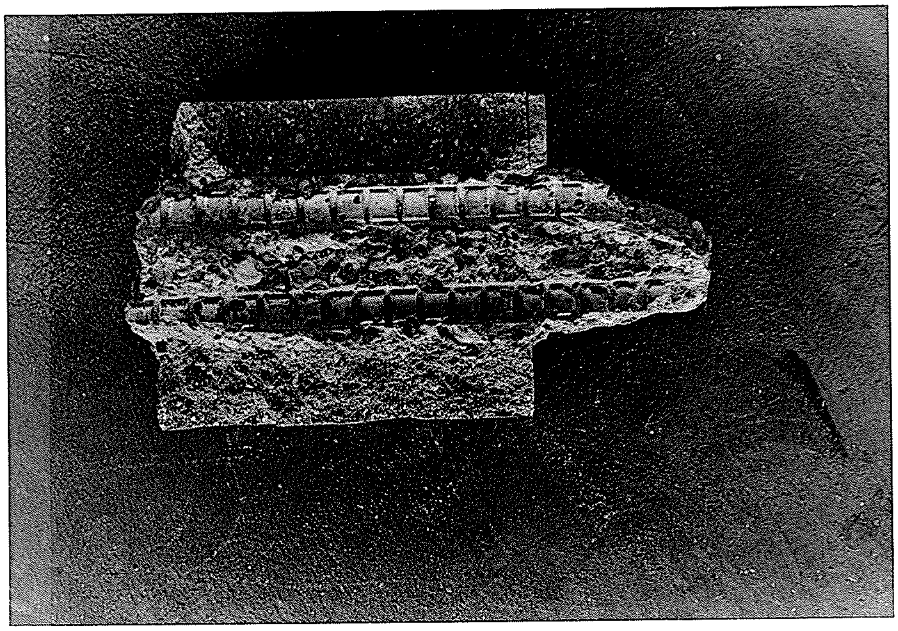


FIG. 70 TOP VIEW OF SAME PIECE SHOWING GOOD BOND BETWEEN CONCRETE AND REINFORCING STEEL. COMPARE WITH RIGHT END IN FIG. 57.

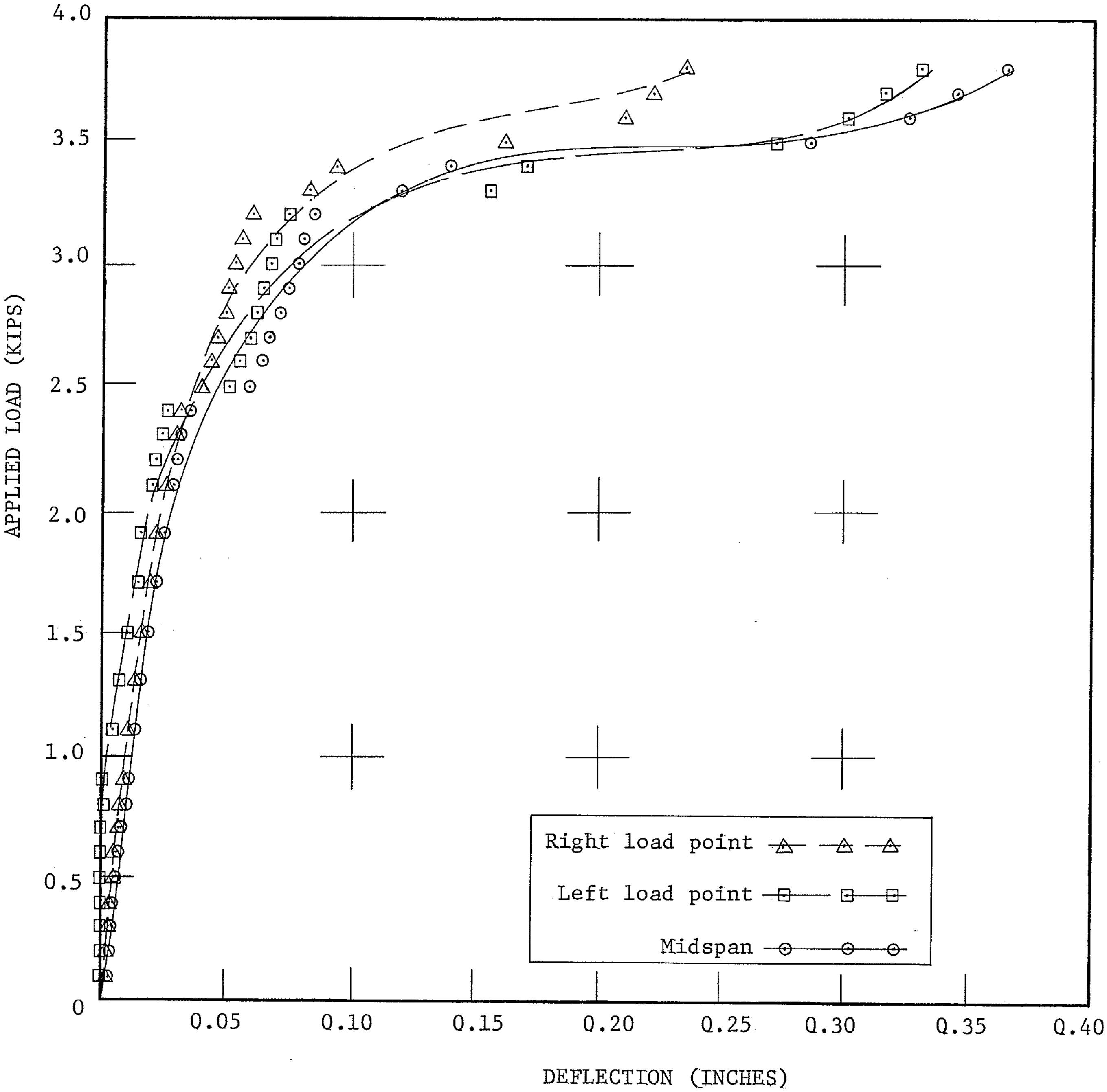


FIG. 71 LOAD-DEFLECTION CURVES FOR BEAM 1A

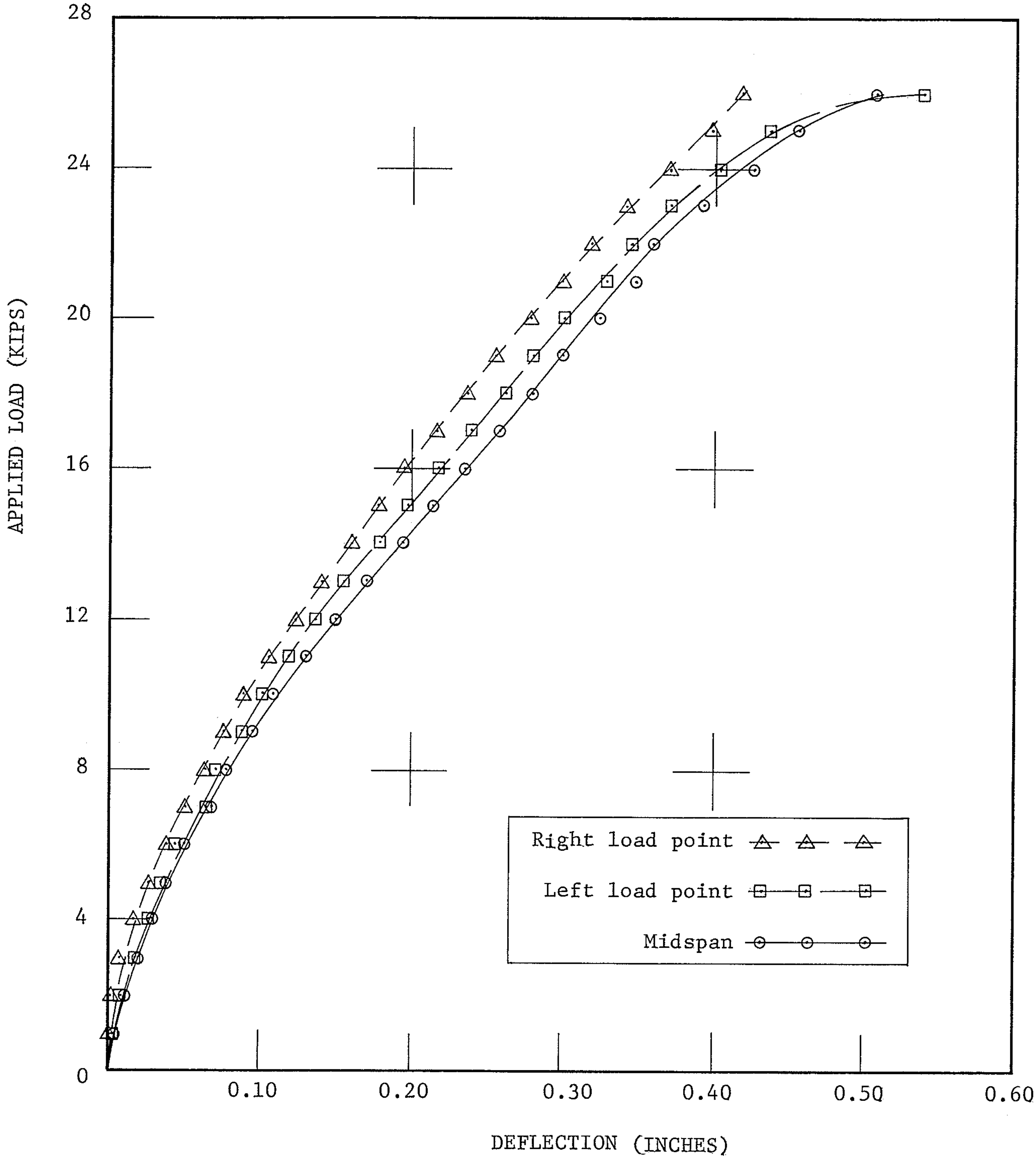


FIG. 72 LOAD-DEFLECTION CURVES FOR BEAM 6

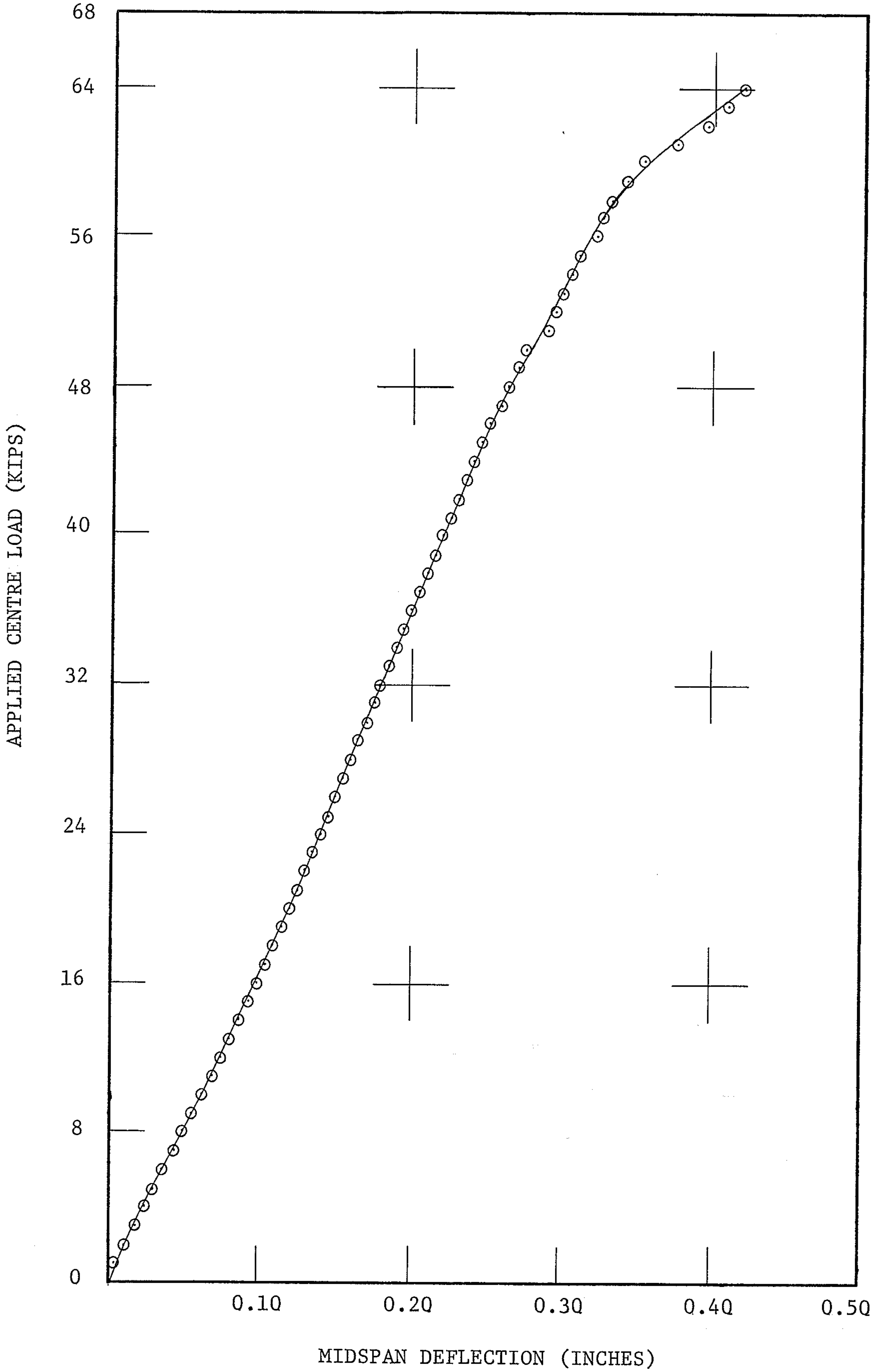


FIG. 73 LOAD-DEFLECTION CURVE FOR BEAM 6B

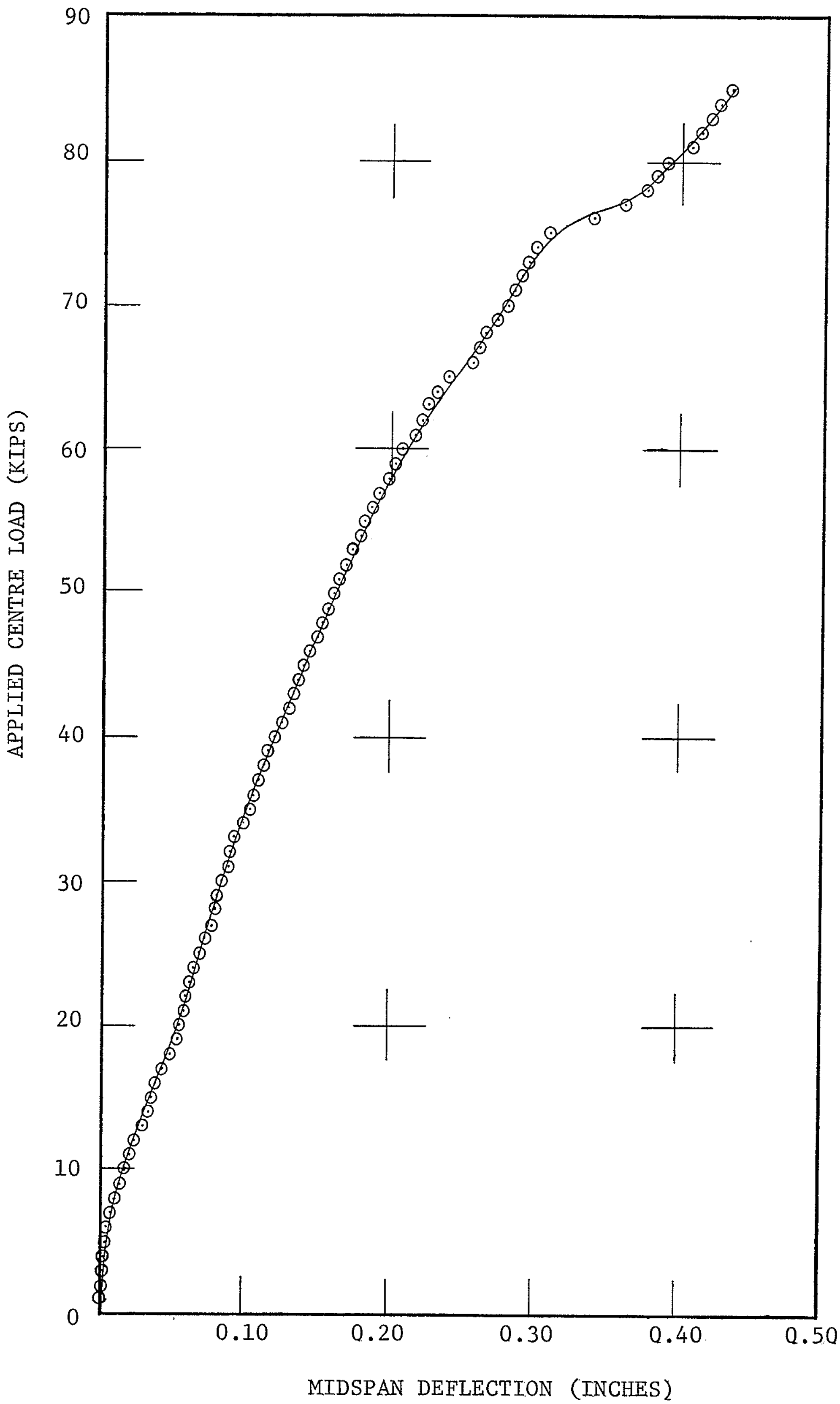


FIG. 74 LOAD-DEFLECTION CURVE FOR BEAM 9B

T A B L E 8

BEAM 2 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
0.3	0.003	0.005	0.002
0.7	0.008	0.011	0.007
1.1	0.018	0.021	0.016
1.4	0.027	0.031	0.024
1.6	0.034	0.038	0.030
1.8	0.038	0.043	0.035
2.0	0.045	0.053	0.041
2.2	0.049	0.059	0.046
2.4	0.057	0.068	0.053
2.6	0.065	0.078	0.060
2.8	0.071	0.083	0.064
3.0	0.076	0.091	0.069
3.2	0.085	0.098	0.074
3.4	0.093	0.106	0.075
3.6	0.095	0.114	0.081
3.8	0.102	0.121	0.086
4.0	0.111	0.128	0.091
4.2	0.121	0.134	0.096
4.4	0.129	0.140	0.101
4.6	0.138	0.147	0.108
4.8	0.144	0.156	0.116
5.0	0.150	0.164	0.121
5.2	0.160	0.176	0.148
5.4	0.165	0.181	0.152
5.6	0.170	0.187	0.157
5.8	0.177	0.193	0.162
6.0	0.186	0.200	0.168
6.2	0.197	0.212	0.190
6.4	0.204	0.221	0.196
6.6	0.216	0.232	0.203
6.7	0.221	0.237	0.207

TABLE 9

BEAM 2A TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
0.3	0.008	0.005	0
0.6	0.013	0.010	0.007
0.9	0.019	0.016	0.012
1.2	0.017	0.025	0.018
1.5	0.023	0.031	0.026
1.8	0.032	0.040	0.033
2.1	0.042	0.050	0.042
2.4	0.052	0.060	0.051
2.7	0.062	0.069	0.058
3.0	0.072	0.080	0.066
3.3	0.083	0.090	0.075
3.6	0.098	0.103	0.085
3.9	0.109	0.112	0.094
4.2	0.121	0.124	0.103
4.5	0.132	0.135	0.111
4.8	0.147	0.148	0.122
5.1	0.179	0.176	0.140
5.4	0.259	0.230	0.177
5.7	0.317	0.275	0.208
6.0	0.405	0.339	0.350
6.3 (failure)	0.519	0.430	0.415
4.5k sustained for 20 hours	0.581	0.469	0.432

T A B L E   1 0

BEAM 3 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
0.4	0	0.005	0.005
1.0	0.001	0.010	0.011
1.4	0.004	0.020	0.021
1.6	0.009	0.025	0.025
1.8	0.013	0.031	0.030
2.0	0.017	0.037	0.034
2.2	0.023	0.043	0.040
2.4	0.027	0.048	0.045
2.6	0.034	0.056	0.052
3.0	0.045	0.067	0.062
3.2	0.053	0.075	0.070
3.4	0.059	0.080	0.075
3.6	0.065	0.086	0.082
3.8	0.079	0.102	0.094
4.0	0.083	0.106	0.098
4.2	0.089	0.113	0.104
4.4	0.094	0.118	0.109
4.6	0.098	0.122	0.114
4.8	0.105	0.130	0.122
5.0	0.113	0.137	0.131
5.2	0.120	0.142	0.136
5.4	0.127	0.149	0.142
5.6	0.136	0.158	0.150
5.8	0.142	0.164	0.156
6.0	0.148	0.172	0.161
6.2	0.160	0.188	0.171
6.4	0.166	0.195	0.176
6.6	0.178	0.208	0.186
6.8	0.185	0.216	0.192
7.0	0.217	0.245	0.216
7.2	0.279	0.315	0.302
7.4	0.364	0.384	0.358
7.6	0.374	0.395	0.366
7.8	0.401	0.435	0.403
8.0	0.453	0.503	0.476
8.2	0.556	0.606	0.582
8.25	0.565	0.626	0.602

TABLE 11

BEAM 3A TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
0.3	0.003	0.005	0.003
0.6	0.010	0.013	0.010
0.9	0.016	0.019	0.018
1.2	0.023	0.028	0.025
1.5	0.032	0.037	0.032
1.8	0.041	0.045	0.040
2.1	0.046	0.056	0.048
2.4	0.054	0.063	0.056
2.7	0.066	0.076	0.069
3.0	0.074	0.083	0.075
3.3	0.084	0.093	0.085
3.6	0.096	0.105	0.095
3.9	0.104	0.113	0.103
4.2	0.115	0.124	0.112
4.5	0.126	0.135	0.122
4.8	0.138	0.147	0.133
5.1	0.146	0.156	0.141
5.4	0.157	0.167	0.151
5.7	0.166	0.177	0.161
6.0	0.178	0.190	0.172
6.3	0.190	0.200	0.180
6.6	0.201	0.211	0.190
6.9	0.219	0.226	0.201
7.2	0.231	0.238	0.212
7.5	0.243	0.251	0.222
7.8	0.260	0.267	0.235
8.1	0.270	0.274	0.243
8.4	0.281	0.286	0.254
8.7	0.297	0.301	0.268
9.0	0.308	0.312	0.277
9.3	0.326	0.326	0.290
9.6	0.371	0.368	0.318
9.9	0.391	0.385	0.330
10.2	0.410	0.402	0.343
10.5	0.429	0.420	0.356
10.8	0.458	0.446	0.372
11.1	0.492	0.472	0.392
11.4	0.547	0.518	0.424
11.7	0.589	0.555	0.448

T A B L E 1 2

BEAM 4 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
0.2	0.002	0.002	0.001
0.4	0.004	0.004	0.002
0.6	0.006	0.006	0.004
0.8	0.007	0.008	0.006
1.0	0.009	0.010	0.008
1.2	0.011	0.012	0.010
1.4	0.014	0.015	0.012
1.6	0.016	0.017	0.015
1.8	0.019	0.021	0.018
2.0	0.022	0.024	0.021
2.2	0.025	0.027	0.024
2.4	0.029	0.031	0.029
2.6	0.032	0.035	0.032
2.8	0.037	0.041	0.037
3.0	0.042	0.045	0.041
3.2	0.047	0.051	0.046
3.4	0.051	0.055	0.050
3.6	0.055	0.059	0.054
3.8	0.059	0.063	0.058
4.0	0.063	0.067	0.061
4.2	0.068	0.071	0.065
4.4	0.077	0.077	0.069
4.6	0.079	0.082	0.075

T A B L E 1 3

BEAM 5 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES)		
	LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1.0	0.010	0.012	0.013
2.0	0.023	0.025	0.026
3.0	0.037	0.041	0.044
4.0	0.052	0.055	0.060
5.0	0.072	0.077	0.091
6.0	0.098	0.104	0.121

TABLE 14

BEAM 7 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.014	0.009	0.012
2	0.031	0.021	0.025
3	0.047	0.023	0.042
4	0.065	0.039	0.059
5	0.084	0.067	0.077
6	0.100	0.077	0.091
7	0.121	0.103	0.117
8	0.139	0.121	0.137
9	0.160	0.147	0.159
10	0.183	0.177	0.180
11	0.205	0.202	0.202
12	0.225	0.224	0.223
13	0.248	0.254	0.246
14	0.272	0.282	0.270
15	0.295	0.309	0.292
16	0.318	0.331	0.316
17	0.342	0.355	0.343
18	0.365	0.380	0.363
19	0.390	0.409	0.380
20	0.414	0.435	0.414
21	0.441	0.463	0.442
22	0.466	0.493	0.466
23	0.491	0.522	0.480
24	0.517	0.553	0.505
25	0.546	0.602	0.536
26	0.573	0.624	0.573
27	0.601	0.654	0.598
28	0.628	0.687	0.627

TABLE 15

BEAM 7A TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.007	0.006	0.010
2	0.019	0.016	0.023
3	0.032	0.028	0.035
4	0.045	0.043	0.049
5	0.059	0.059	0.064
6	0.073	0.075	0.079
7	0.089	0.092	0.095
8	0.104	0.110	0.106
9	0.118	0.126	0.122
10	0.133	0.144	0.137
11	0.149	0.161	0.154
12	0.165	0.179	0.171
13	0.180	0.196	0.186
14	0.195	0.214	0.202
15	0.209	0.231	0.218
16	0.224	0.253	0.233
17	0.240	0.271	0.249
18	0.254	0.288	0.263
19	0.269	0.306	0.279
20	0.284	0.322	0.290
21	0.299	0.340	0.306
22	0.314	0.356	0.321
23	0.330	0.374	0.337
24	0.345	0.393	0.353
25	0.361	0.410	0.369
26	0.384	0.475	0.394
27	0.397	0.489	0.407
28	0.408	0.503	0.421
29	0.428	0.522	0.449
30	0.448	0.542	0.467
31	0.470	0.570	0.488
32	0.486	0.598	0.506
33	0.508	0.622	0.530
34	0.521	0.645	0.556
35	0.557	0.681	0.582
36	0.595	0.726	0.625
37	0.815	0.827	0.747

TABLE 16

BEAM 8 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.009	0.001	0.007
2	0.023	0.020	0.022
3	0.042	0.039	0.042
4	0.062	0.060	0.064
5	0.080	0.088	0.083
6	0.098	0.111	0.108
7	0.116	0.137	0.128
8	0.136	0.162	0.146
9	0.157	0.186	0.168
10	0.180	0.211	0.191
11	0.201	0.235	0.212
12	0.224	0.265	0.234
13	0.245	0.290	0.257
14	0.269	0.315	0.281
15	0.292	0.351	0.306
16	0.315	0.372	0.334
17	0.342	0.403	0.359
18	0.373	0.448	0.391
19	0.428	0.514	0.442
20	0.452	0.541	0.465
21	0.480	0.579	0.493
22	0.516	0.637	0.533
23	0.552	0.683	0.568
24	0.583	0.722	0.608
25	0.614	0.761	0.638
26	0.673	0.805	0.689

TABLE 17

BEAM 8A TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.023	0.005	0.008
2	0.048	0.015	0.018
3	0.066	0.028	0.020
4	0.084	0.056	0.041
5	0.101	0.077	0.066
6	0.118	0.097	0.091
7	0.135	0.119	0.112
8	0.151	0.141	0.132
9	0.167	0.154	0.153
10	0.186	0.172	0.172
11	0.204	0.191	0.191
12	0.220	0.207	0.210
13	0.237	0.225	0.229
14	0.255	0.242	0.248
15	0.271	0.261	0.267
16	0.289	0.278	0.283
17	0.304	0.296	0.301
18	0.320	0.316	0.318
19	0.335	0.333	0.336
20	0.351	0.357	0.355
21	0.369	0.382	0.372
22	0.385	0.406	0.388
23	0.403	0.426	0.405
24	0.420	0.444	0.424
25	0.438	0.466	0.446
26	0.456	0.494	0.465
27	0.478	0.517	0.489
28	0.496	0.534	0.507
29	0.516	0.554	0.527
30	0.539	0.578	0.552
31	0.559	0.598	0.573
32	0.581	0.622	0.593
33	0.604	0.644	0.616
34	0.628	0.668	0.642
35	0.653	0.719	0.693

T A B L E 1 8

BEAM 9 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.013	0.019	0.008
2	0.026	0.045	0.020
3	0.042	0.069	0.034
4	0.056	0.094	0.046
5	0.070	0.120	0.063
6	0.086	0.147	0.078
7	0.104	0.177	0.093
8	0.121	0.210	0.108
9	0.137	0.243	0.126
10	0.157	0.280	0.142
11	0.174	0.339	0.159
12	0.185	0.355	0.166
13	0.200	0.374	0.176
14	0.220	0.399	0.194
15	0.240	0.424	0.212
16	0.265	0.450	0.233
17	0.290	0.478	0.255
18	0.318	0.533	0.288
19	0.336	0.558	0.302
20	0.359	0.585	0.322
21	0.388	0.622	0.349

TABLE 19

BEAM 9A TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.003	0.001	0.002
2	0.046	0.012	0.040
3	0.071	0.034	0.065
4	0.095	0.060	0.093
5	0.116	0.080	0.112
6	0.137	0.106	0.136
7	0.157	0.132	0.158
8	0.178	0.156	0.178
9	0.199	0.177	0.198
10	0.220	0.197	0.217
11	0.240	0.217	0.238
12	0.260	0.241	0.259
13	0.279	0.261	0.278
14	0.298	0.278	0.304
15	0.317	0.299	0.320
16	0.336	0.313	0.339
17	0.355	0.344	0.357
18	0.376	0.364	0.383
19	0.397	0.387	0.403
20	0.418	0.414	0.423
21	0.438	0.440	0.446
22	0.461	0.471	0.470
23	0.490	0.508	0.503
24	0.549	0.607	0.605

TABLE 20

BEAM 1A TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
0.1	0	0.003	0.002
0.2	0	0.004	0.002
0.3	0	0.004	0.003
0.4	0	0.005	0.003
0.5	0	0.006	0.005
0.6	0	0.007	0.006
0.7	0	0.008	0.007
0.8	0.001	0.010	0.008
0.9	0.001	0.011	0.009
1.1	0.004	0.013	0.011
1.3	0.007	0.016	0.013
1.5	0.010	0.018	0.016
1.7	0.014	0.022	0.019
1.9	0.016	0.025	0.022
2.1	0.020	0.028	0.026
2.2	0.022	0.030	0.028
2.3	0.024	0.032	0.030
2.4	0.026	0.035	0.032
2.5	0.051	0.059	0.039
2.6	0.055	0.064	0.043
2.7	0.058	0.067	0.046
2.8	0.062	0.072	0.048
2.9	0.064	0.074	0.050
3.0	0.067	0.078	0.053
3.1	0.070	0.081	0.056
3.2	0.074	0.085	0.059
3.3	0.156	0.119	0.082
3.4	0.171	0.139	0.093
3.5	0.172	0.286	0.187
3.6	0.301	0.326	0.210
3.7	0.316	0.345	0.222
3.8	0.331	0.365	0.235

TABLE 21

BEAM 6 TEST RESULTS

APPLIED LOAD AT EACH THIRD POINT (KIPS)	DEFLECTION (INCHES) LEFT LOAD POINT	CENTRE	RIGHT LOAD POINT
0	0	0	0
1	0.004	0.005	0.001
2	0.010	0.012	0.001
3	0.018	0.020	0.008
4	0.026	0.029	0.017
5	0.035	0.039	0.027
6	0.045	0.051	0.039
7	0.059	0.065	0.052
8	0.072	0.079	0.064
9	0.089	0.096	0.076
10	0.103	0.111	0.090
11	0.120	0.131	0.107
12	0.137	0.151	0.124
13	0.156	0.171	0.141
14	0.180	0.194	0.161
15	0.198	0.215	0.178
16	0.218	0.236	0.196
17	0.240	0.258	0.217
18	0.263	0.281	0.237
19	0.282	0.301	0.256
20	0.301	0.325	0.278
21	0.328	0.348	0.300
22	0.346	0.369	0.318
23	0.371	0.393	0.341
24	0.404	0.426	0.370
25	0.436	0.456	0.397
26	0.537	0.507	0.418

TABLE 22

BEAM 6B TEST RESULTS

APPLIED LOAD (KIPS)	DEFLECTION (INCHES)	APPLIED LOAD (KIPS)	DEFLECTION (INCHES)
0	0	33	0.185
1	0.005	34	0.190
2	0.012	35	0.194
3	0.019	36	0.199
4	0.025	37	0.205
5	0.031	38	0.210
6	0.037	39	0.215
7	0.044	40	0.221
8	0.050	41	0.226
9	0.056	42	0.231
10	0.063	43	0.236
11	0.069	44	0.241
12	0.075	45	0.246
13	0.081	46	0.252
14	0.087	47	0.258
15	0.093	48	0.263
16	0.099	49	0.269
17	0.104	50	0.275
18	0.109	51	0.289
19	0.115	52	0.294
20	0.120	53	0.299
21	0.125	54	0.304
22	0.130	55	0.310
23	0.135	56	0.322
24	0.140	57	0.326
25	0.145	58	0.332
26	0.150	59	0.342
27	0.155	60	0.353
28	0.159	61	0.375
29	0.164	62	0.395
30	0.170	63	0.408
31	0.176	64	0.420
32	0.180		

TABLE 23

BEAM 9B TEST RESULTS

APPLIED LOAD (KIPS)	DEFLECTION (INCHES)	APPLIED LOAD (KIPS)	DEFLECTION (INCHES)
0	0	43	0.133
1	0	44	0.136
2	0.001	45	0.140
3	0.002	46	0.145
4	0.003	47	0.149
5	0.004	48	0.153
6	0.006	49	0.157
7	0.009	50	0.162
8	0.012	51	0.165
9	0.015	52	0.169
10	0.018	53	0.173
11	0.022	54	0.178
12	0.025	55	0.182
13	0.032	56	0.187
14	0.035	57	0.192
15	0.037	58	0.198
16	0.039	59	0.203
17	0.044	60	0.208
18	0.049	61	0.217
19	0.054	62	0.222
20	0.056	63	0.225
21	0.058	64	0.232
22	0.059	65	0.240
23	0.063	66	0.257
24	0.066	67	0.261
25	0.069	68	0.265
26	0.073	69	0.273
27	0.078	70	0.281
28	0.080	71	0.285
29	0.082	72	0.290
30	0.085	73	0.295
31	0.088	74	0.301
32	0.090	75	0.309
33	0.093	76	0.339
34	0.099	77	0.360
35	0.104	78	0.376
36	0.107	79	0.382
37	0.110	80	0.389
38	0.114	81	0.407
39	0.117	82	0.413
40	0.121	83	0.419
41	0.126	84	0.425
42	0.130	85	0.433

VII. DESIGN AND ANALYSIS

The beams were designed and analyzed in accordance with the "ACI Building Code for Reinforced Concrete"[1]. A diagram for each type of loading encountered is drawn below. Figure 75 applies to the core-filled and totally filled beams; Figure 76 to the end-filled beams; and Figure 77 to the two 16'-0 span concentrically loaded beams.

1. PERCENTAGE OF LONGITUDINAL REINFORCEMENT

Percentages of longitudinal reinforcement were kept within the following limits:

$$\frac{200}{fy} \leq p \leq 0.75 \; \frac{0.85\ K_1 f'_c}{fy} \; \frac{87{,}000}{87{,}000 + fy} \quad \cdots\cdots \qquad (1)$$

where p = percentage of longitudinal steel $(\frac{A_s}{bd})$.

fy = yield point of longitudinal steel, (p.s.i.).

K_1 = constant, (0.85).

f'_c = ultimate compressive strength of concrete fill, (p.s.i.).

A_s = cross-sectional area of longitudinal steel, (in.^{2}).

b = effective width of beam, (in.).

d = depth of compression face to centroid of longitudinal steel (in.).

2. DEFLECTIONS

Allowable deflections at midspan for short time loads are governed by the following formula:

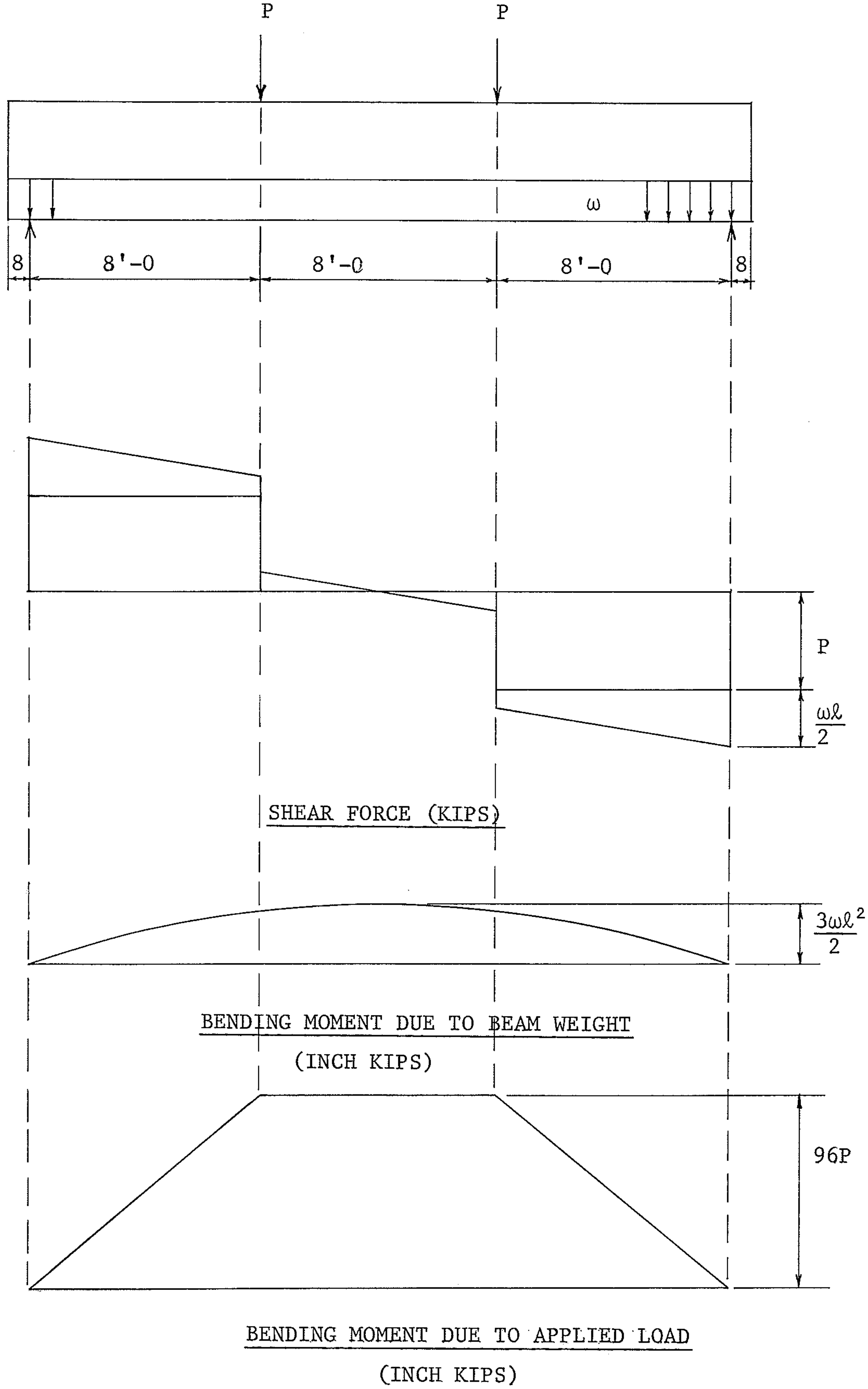


FIG. 75 LOADING DIAGRAM FOR CORE-FILLED AND TOTALLY FILLED BEAMS

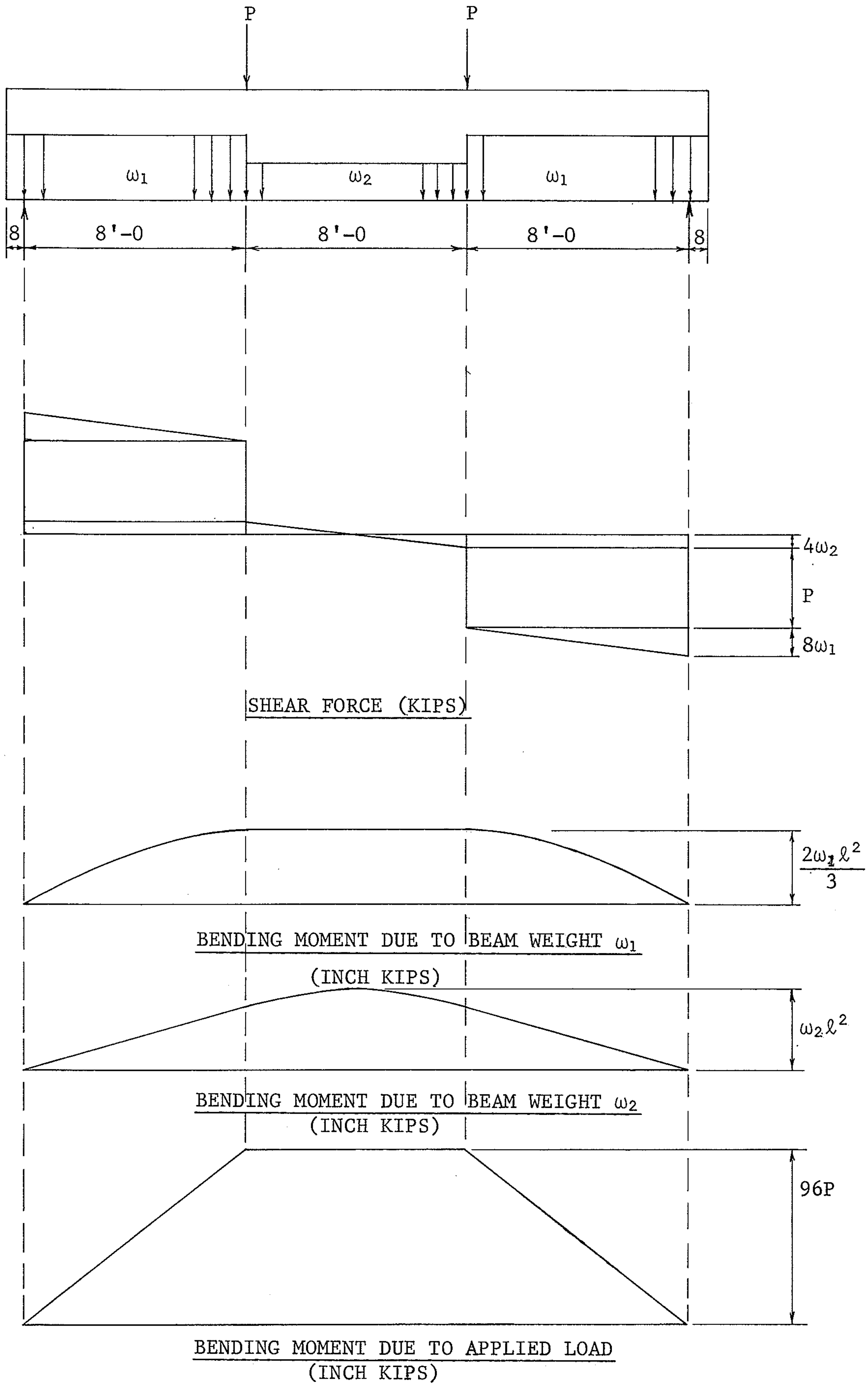
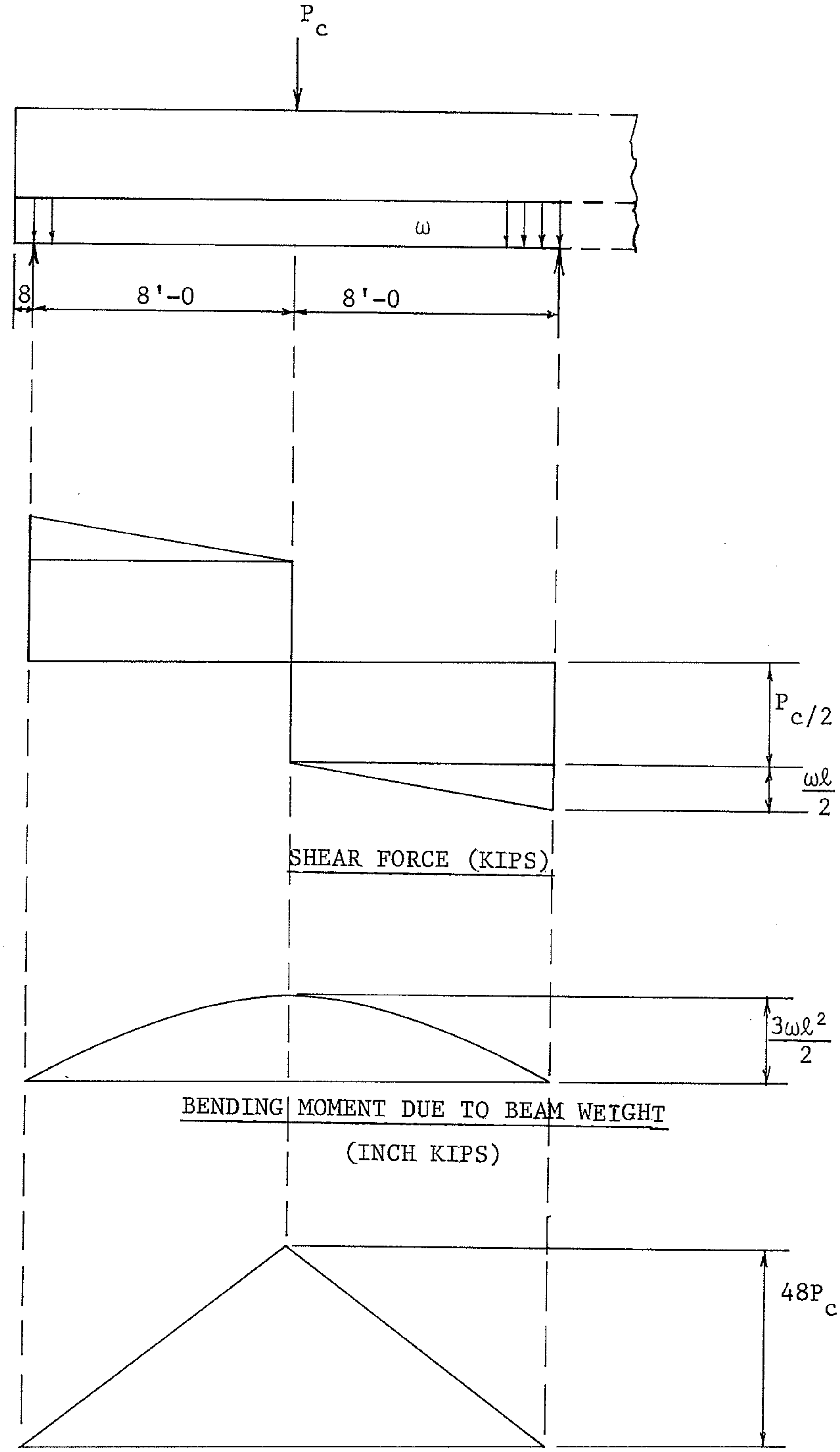


FIG. 76 LOADING DIAGRAM FOR END-FILLED BEAMS



BENDING MOMENT DUE TO APPLIED LOAD

(INCH KIPS)

FIG. 77 LOADING DIAGRAM FOR CONCENTRICALLY LOADED 16'-0 SPAN BEAMS

$$\Delta(\max) = \frac{\ell}{360} \quad \ldots. \tag{2}$$

for $\ell = 24 \times 12 = 288$ in.

$\Delta(\max) = 0.805$ in.

and for $\ell = 16 \times 12 = 192$ in.

$\Delta(\max) = 0.534$ in.

An analysis of each type of beam follows.

3. CORE-FILLED BEAMS

(a) Flexural Computations

From the external loading diagram (Figure 75),

$$M_{test} = 96P + \frac{3\,w\,\ell^2}{2} \quad \ldots. \tag{3}$$

where M_{test} = maximum external bending moment resisted by beam, (in.k.).

P = maximum third point load applied, (k.).

w = self weight of beam, (k/ft.).

ℓ = length of beam centre to centre of simple supports, (ft.).

The ultimate flexural capacity is determined by

$$M_u = \frac{A_s f_y \left(d - \frac{a}{2}\right)}{1000} \quad \ldots. \tag{4}$$

where M_u = ultimate moment, (in.k.).

$$a = \frac{A_s f_y}{0.85\, f'_m\, b} \quad \ldots. \tag{5}$$

= depth of compression zone.

f'_m = ultimate compressive strength of mortar (as determined from 4" cube tests), (p.s.i.).

b = effective width of beam at critical section, (in.).

Values of M_{test} and M_u are listed in Table 24.

(b) <u>Shear</u>

The assumption made here is that the vertical concrete cores carry virtually all the shear forces. This was verified by the performance of Beam 1. Using the concept of horizontal shear flow,

$$vb = \frac{V_{test}Q}{1000\ I} \quad \ldots. \qquad (6)$$

where vb = horizontal shear flow at the critical section (i.e. the neutral axis), (lb./in.).

$V_{test} = P + \frac{w\ell}{2}$

= maximum shearing force in the beam (at ultimate load, (kips).

$Q = A'\bar{y}$, (second moment of area about neutral axis), (in.^{3}).

A' = area of concrete above neutral axis, (in.^{2}).

$\bar{y}$ = distance between centroid of A' and neutral axis, (in.)

I = moment of inertia of core section about neutral axis, (in.^{4}).

The horizontal shear stress acting across the core was then found by dividing the horizontal shear force by the core area

$$v_H = \frac{V_H}{A_{core}} \quad \ldots. \qquad (7)$$

where A_{core} = area of concrete plus the transformed area of vertical steel in the core.

Finally, the ratio $\frac{v_H}{f'_c}$ was computed for each beam and tabulated in Table 24.

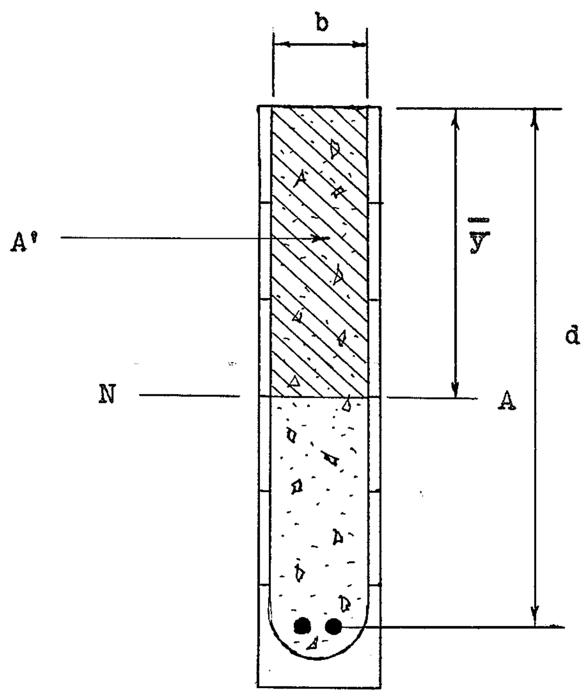


FIG. 78 CROSS SECTION OF BEAM THROUGH CONCRETE CORE

The horizontal shear force at this section is then obtained by multiplying by the span of beam carried by each core.

$$V_H = vb \times S \quad .... \tag{8}$$

where V_H = horizontal shear force, (lbs.).

S = core spacing = 48 in.

Note that in the two beams in which intermediate cores were poured (Beams 2A and 3A) the original cores were already broken and therefore considered ineffective.

4. END-FILLED BEAMS

(a) Flexural Computations

From the external loading diagram, (Figure 76),

$$M_{test} = 96P + \frac{2w_1 \ell^2}{3} + w_2 \;{}^{2} \quad \ldots . \qquad (9)$$

where w_1 = self weight of beam in end third regions = .382 kips/ft.

w_2 = self weight of beam in central third, (kips/ft.).

The ultimate flexural capacity is again determined by formula (4)

$$M_u = A_s f_y \; (d - \frac{a}{2}) \quad \ldots . \qquad (4)$$

Values of M_{test} and M_u are compared in Table 24.

(b) Shear

The shear stress carried by the concrete at ultimate load is determined by the ACI[1] formula for shear and diagonal tension:

$$v_{cu} = \left\{ 1.9 \sqrt{f'_c} + 2,500 \frac{p \, V_{test} d}{M_{test}} \leq 3.5 \sqrt{f'_c} \right\} \quad \ldots . \qquad (10)$$

where v_{cu} = shear stress carried by concrete at ultimate load at the critical section, (p.s.i.).

$V_{test} = P + 8w_1 + 4w_2$, (kips).

The shear force carried by the concrete at ultimate load, V_{cu}, is then found by formula (11).

$$V_{cu} = \frac{v_{cu} \, b'd}{1000} \quad \ldots . \qquad (11)$$

where b' = width of poured-in-place concrete = 4.88 in.

V_{cu} = shear force carried by concrete, (kips).

The shear carried by the vertical shear reinforcement is determined by

$$V'_u = \frac{A_v fyd}{1000S} \quad .... \tag{12}$$

where A_v = area of web reinforcement within a distance S, (in.^{2}).

fy = yield strength of shear reinforcement, (p.s.i.).

S = spacing of vertical shear reinforcement, (in.).

The total shear capacity of the beam is then the sum of the shear carried by the concrete and reinforcement.

$$V_u = V_{cu} + V'_u \quad .... \tag{13}$$

where V_u = total ultimate shear capacity, (kips.).

5. TOTALLY FILLED BEAMS

(a) Flexural Computations

For the 24'-0 span beams, formula (3) is again used to determine the maximum bending moment

$$M_{test} = 96P + \frac{12w\ell^2}{8} \quad .... \tag{3}$$

For the 16'-0 span concentrically loaded beams, the loading diagram of Figure 77 is used to obtain the following formula:

$$M_{test} = 48P_c + \frac{3w\ell^2}{2} \quad .... \tag{14}$$

where P_c = centre point load, (kips).

For both spans, the ultimate flexural capacity is again determined by formula (4),

$$M_u = A_s f_y \left(d - \frac{a}{2}\right) \quad \cdots\cdots \qquad (4)$$

where

$$a = \frac{A_s f_y}{0.85 \ f'_c \ b} \quad \cdots\cdots \qquad (15)$$

Values of M_u and M_{test} are again compared in Table 24.

(b) Shear

The total ultimate shear capacity of these beams is determined by the same method as used for the end-filled beams except that V_{test} is to be taken as

$$V_{test} = P + \frac{w\ell}{2}$$

for the 24'-0 span beams and

$$V_{test} = \frac{P_c}{2} + \frac{w\ell}{2}$$

for the 16'-0 span beams.

VIII. EVALUATION OF TEST RESULTS

A complete summary of the test results is listed in Table 24. In this table, a comparison is made between the theoretical ultimate capacities of the beams and the actual test results. Table 1 entitled Beam Summary outlines the physical properties of each beam and is repeated in this section for convenient reference in conjunction with Table 24.

1. COMPARISON OF EXPERIMENTAL STRENGTH TO ULTIMATE STRENGTH THEORY

Shear was the governing mode of failure for each core-filled beam. In each case, the maximum bending moment reached, M_{test}, was well below the ultimate moment M_u. In Table 24, the value v_H represents the horizontal shear stress in the concrete cores at failure. A percentile ratio of $\frac{v_H}{f'_c}$ is then listed for each core-filled beam. These percentages range from 15.9 to 51.2% with an average value of 27.9% (not including the totally unfilled beam, Beam 1). This compares reasonably well with the findings of other researchers. Orchard[7] states that the shear strength of concrete has been found to range between 12 to 90% of the compressive strength by previous researchers.

In order to obtain a further comparison of the $\frac{v_H}{f'_c}$ ratio which would be more relevant, three direct shear tests were run on core-filled blocks as outlined in the Appendix. These tests yielded a ratio of shear strength to compressive strength of concrete of 23.7%.

This value agrees very well with the ratio of 27.9% obtained from

TABLE 24

SUMMARY OF BEAM RESULTS

A. CORE-FILLED BEAMS

BEAM NO.	M_u (IN. KIPS)	M_{test} (IN. KIPS)	V_{test} (KIPS)	v_H (P.S.I.)	$\frac{v_H}{f'_c}$ (%)	DEFLN. AT ULT. LOAD (IN.)	MODE OF FAILURE
1	2,235	191	2.7	46	2.3	3.0*	shear
2	2,210	866	9.7	805	22.2	0.237	shear
2A	2,210	839	9.6	719	32.7	0.519	shear
3	2,360	1,016	11.2	844	20.4	0.626	shear
3A	2,210	1,344	15.0	1,130	51.2	0.589	shear
4	3,350	702	8.2	597	15.9	0.082	shear
5	3,300	836	9.6	665	24.9	0.121	shear

B. END-FILLED BEAMS

BEAM NO.	M_u (IN. KIPS)	M_{test} (IN. KIPS)	V_{cu} (KIPS)	V'_u (KIPS)	V_u (KIPS)	V_{test} (KIPS)	DEFLN. AT ULT. LOAD (IN.)	MODE OF FAILURE
7	2,850	2,950	22.7	29.4	52.1	31.8	0.687	flexure**
7A	4,290	3,850	23.4	29.4	52.8	41.1	0.827	shear
8	2,630	2,760	23.4	19.5	42.9	29.8	0.805	flexure**
8A	4,290	3,640	23.4	19.5	42.9	38.9	0.719	shear
9	2,520	2,280	22.4	9.8	32.2	24.8	0.622	flexure**
9A	3,200	2,560	24.5	9.8	34.3	27.8	0.607	flexural** -shear

TABLE 24

(continued)

C. TOTALLY FILLED BEAMS

BEAM NO.	M_u (IN. KIPS)	M_{test} (IN. KIPS)	V_{cu} (KIPS)	V'_u (KIPS)	V_u (KIPS)	V_{test} (KIPS)	DEFLN. AT ULT. LOAD (IN.)	MODE OF FAILURE
1A	3,300	695	26.2	0	26.2	8.4	0.365	shear
6	4,140	2,830	22.4	0	22.4	30.6	0.537	shear
6B	4,260	3,820	26.2	0	26.2	35.1	0.420	flexural-shear
9B	4,250	4,220	26.8	9.8	36.6	45.6	0.433	flexural-shear

* Estimated deflection

** Flexural failure occurred by crushing of concrete blocks and mortar in top courses.

the beam tests. Thus, on the basis of the shear to compressive strength ratio, the experimental capacities of the core-filled beams are closely approximated by theory. Furthermore, both sets of tests indicate that the shear strength of concrete as analyzed by the ACI code[1] is quite conservative. The Code limits concrete shear strength not to exceed 3.5 $\sqrt{f'_c}$. The tests carried out herein show that the shear strength is about 25% of the compressive strength.

The hollow beam, Beam 1 had a comparatively low horizontal shear strength of 46 p.s.i. In effect, this was the shear strength of the mortar which held the blocks together. This value is justified by the fact that the strength of the mortar in shear is extremely low as discussed in the Appendix.

This first series of beams pointed out a need for greater resistance to shearing forces. Beams 2A and 3A contained intermediate cores which were keyed into the bottom course of concrete. The fact that shear to compressive strength ratios were highest in these two beams emphasizes the fact that a lack of continuity between the bottom course and the core concrete in the other beams of Series 1 was an obvious weakness. Consequently, the next series was filled monolithically. By totally filling the end portions and using more shear reinforcing, it was hoped that the shear capacities of these beams would be much higher.

As a result, the applied loads doubled and tripled in the end-filled beams. In fact, the shear resistance of Beams 7 and 8 was sufficiently high enough to induce flexural failures. These occurred by the crushing of the blocks and mortar in the centre of the top courses. In both of these beams, the theoretical ultimate flexural capacities were surpassed by 100 and 150 inch kips. Beam 9 did not yet reach its ultimate flexural capacity when crushing occurred in one of the top mortar joints. It was, however, not far from the ultimate value at this point, and the premature crushing could have been due to a slight inconsistancy in this joint such as a narrower width, a piece of aggregate, a large air void, etc.

The results of Beams 7, 8 and 9 show that the flexural capacity of masonry beams can be quite accurately predicted by ultimate strength theory.

Beams 7A, 8A and 9A had their top central courses filled in order to strengthen the flexural capacity of the beams and attempt to study the shear capabilities. As listed in Table 24, the maximum shear force in each of these beams at failure was below their estimated ultimate values. It must be emphasized here that in each of these beams shear failures occurred in the end which was last filled with concrete. As stated in Part III, the concrete poured into this end of the beams was extremely stiff, had a slump of less than two inches and could not be thoroughly vibrated. An illustration of this concrete is shown in Figures 52 and 57. It could be seen that little or no bond with the reinforcing steel was achieved.

This poor quality concrete must be attributed a major factor in the under ultimate capacity of these three beams. Whether or not the shear resistance would have equalled the predicted ultimate values if the concrete fill was uniform throughout remains a question.

Perhaps a partial answer to this can be found by examining the performance of the totally filled beams. The results of Beam 1A cannot be taken into consideration because the bottom course of this beam was poured separately and undoubtedly damaged severely when the beam was tested as Beam 1. Nevertheless, a comparison between the ultimate and experimental shear values (V_u and V_{test}) of Beams 6, 6B and 9B supports the hypothesis that the shear capacity of end-filled masonry beams is at least that predicted by Ultimate Strength Theory. Beam 9B which failed in flexural shear also reinforces the fact that Ultimate Strength Theory can be used to accurately analyze the flexural capacity of the beams.

2. DEFLECTIONS

The amount of deflection at midspan at full load was less than the ACI allowable value in all but two of the beams. The allowable value of 0.805 inches was greatly exceeded in Beam 1 and only slightly exceeded in Beam 7A. Very low shear strength of mortar is cited as the main reason for the great deflection in Beam 1.

The load deflection curves in Part VI show that where the mode of failure was shear, the beams having shear reinforcing were more ductile than those without. Highest deflections were observed in the end-filled beams. In beams 2A, 3A and 5, higher deflections were recorded at one of the load points than at midspan.

The beams which failed in flexure by crushing of blocks and mortar had almost straight line load-deflection curves. This is characteristic of the brittle failures. In comparison, the two beams which failed by flexural shear (Beams 6B and 9B) were noticeably more ductile near ultimate load.

Lateral buckling was never a factor in these tests, although several bottom bearing failures appeared to take place. An analysis of the bearing strength of the beams shows that these were not actually bearing failures but a continuation of the diagonal tension cracks.

IX. CONCLUSIONS

From the test results and foregoing analysis, a number of conclusions can be drawn. In view of the fact that seventeen beam tests were performed, some of the statements made herein are of a tentative nature. While the test results seem in many respects to warrant definite conclusions in view of their uniformity, in other respects an element of personal judgment enters into the interpretation, particularly as regards the concept of shear capacity of the beams.

(1) The beams of Series 2 which failed in flexure show that the flexural capacity of masonry beams can be accurately analyzed by Ultimate Strength Theory under the "ACI Building Code for Reinforced Concrete."

(2) As indicated in the Evaluation of Test Results, the core-filled and totally filled masonry beams could be safely analyzed with respect to shear strength by Ultimate Strength Theory. The end-filled beams were slightly less than the predicted ultimate shear values. However, it is the author's opinion that this was caused by poor quality concrete and that the shear capacities of these beams also meet the ultimate predictions.

(3) The end-filled beams of Series 2 stress the necessity for using a high slump concrete and providing thorough vibration of concrete fill in masonry beam construction. A slump of seven to nine inches is recommended.

(4) Uniformity of concrete within the block courses is also essential.

Where monolithic pours are not feasible, care should be taken to join different pours together by shear keys and reinforcing dowels.

(5) Only the in-filled sections of the beams can be considered effective for shear. As indicated in the Appendix, a higher bond strength mortar would be required in order that the full cross-section concrete, block and mortar could be considered to work in unity to resist horizontal shear forces.

(6) Portions of the masonry beam can be left hollow and seemingly virtually open to allow for doors, windows and other openings. This is verified by the end-filled beams.

(7) Totally unfilled concrete block beams are structurally inadequate when normal mortar is used. This type of member may be greatly strengthened by using a mortar which is capable of resisting high bond, shear and compressive forces. Unfortunately, it has been the role of industry to manufacture mortar cements which are useful only for non-load bearing elements. Hopefully, this will be changed by present technology.

(8) Six tests on masonry beams with concrete fill in one core every forty-eight or twenty-four inches and three direct shear tests on concrete filled block samples indicate that the ratio of shear to compressive strength of concrete is 0.25. This is contrary to the relationship between these two strengths in the ACI Code for Reinforced Concrete which states that the shear strength of concrete shall not exceed the value of $3.5 \sqrt{f'_c}$.

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APPENDIX

Due to the high frequency of shear failures in the beams, shear tests were performed on mortar and concrete. The purpose was to obtain a comparison with shear strengths displayed in the beam tests.

MORTAR SHEAR TESTS

In these tests three 2" x 4" x 8" concrete bricks were mortared together and tested in double shear. Each brick had one flat side and one side that had a 1 1/4" wide by 1/4" deep groove. The bricks were always laid flat side to grooved side.

Various portions of masonry cement, portland cement, fine sand*, coarse sand, and pulverized lime were mixed together to make nine different mixtures. For each mixture, three samples were tested for shear, one for compression and two 4" x 4" cubes were also tested in compression. All tests were performed after thirteen days of dry curing.

A photograph of the shear test is shown in Figure 79. The results of these tests are listed in Table 25. From three compression tests, the compressive strength of the concrete brick was found to be 5,350 p.s.i.

Mortar type 1 represents the type used throughout the beam con-

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* Sieve analyses of these sands are performed in Figures 26 and 27.

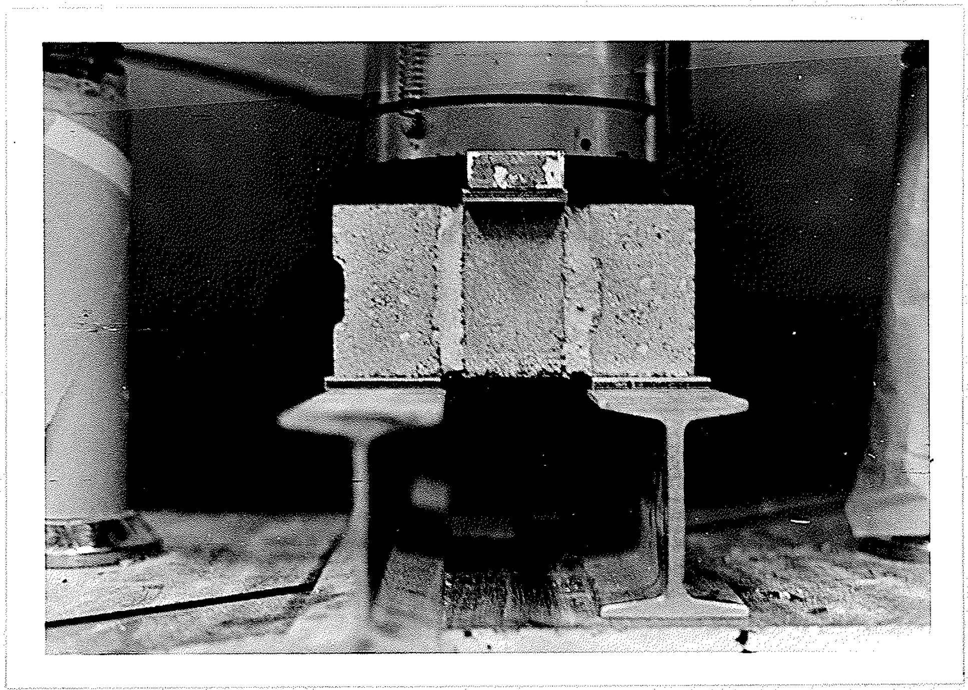

FIG. 79 SHEAR TEST ON MORTAR

TABLE 25

SHEAR AND COMPRESSION TEST RESULTS ON MORTAR

MORTAR TYPE	MIXTURE (PARTS OF MATERIAL)	COMPRESSIVE STRENGTH OF PRISMS (P.S.I.) 1 Test	SHEAR STRENGTH (P.S.I.) Avg. of 3 Tests	CUBE COMP. STRENGTH (P.S.I.) Avg. of 2 Tests
1	3 fine sand 1 mas. cement	2,530	16.8	1,480
2	6 coarse sand 2 port. cement 1 lime	2,850	30.5	2,140
3	6 fine sand 2 port. cement 1 lime	2,350	22.5	2,330
4	6 coarse sand 2 mas. cement 1 lime	2,170	14.0	1,030
5	6 coarse sand 1 mas cement 1 port. cement 1 lime	2,530	27.3	1,610
6	3 coarse sand 1 port. cement	2,710	68.8	4,100
7	3 coarse sand 1 mas. cement	2,280	29.6	1,940
8	3 coarse sand 1 port. cement 1 lime	2,380	28.3	1,150
9	6 coarse sand 2 port. cement 1 lime (wet bricks)	2,750	26.0	2,340
10	plaster of paris	2,500	109.0	400

COMPRESSIVE STRENGTH OF CONCRETE BRICKS (AVG. OF 3 TESTS) = 5,350 P.S.I.

struction. It is evident that this type of mortar has a very low shear strength. Type 6 (3 parts coarse sand, 1 part portland cement) had a shear strength of 68.8 p.s.i. -- far beyond any of the other mortars. For practical applications, this type is not favourable because of the rapid setting time. For this reason, masonry cement or lime additive have been used to increase the "board time" of the mortar. Although they prolong the setting time, these tests point out that mortars which contain these materials are unsatisfactory for structural members.

In all mortars, except type 6, the failure was one of lack of bond between mortar and brick rather than a shearing through the mortar. A typical bond failure is shown in Figure 80 while Figure 81 shows the shear plane through mortar and brick for the type 6 mortar.

From Table 25, it can be seen that there was no correlation between shear strength and compressive strength of the three brick high prisms. There was, however, a direct relationship between shear strength and cube compression strength.

The maximum allowable shear stress for plain and reinforced masonry in the NBC - 1970[6] is based on the prism strength of masonry units. In view of the above tests, a more meaningful determination of allowable shear stress would be on the basis of the compressive strength of mortar cubes.

Clearly, the requirements of a satisfactory mortar for structural applications are:

(1) sufficient compressive strength

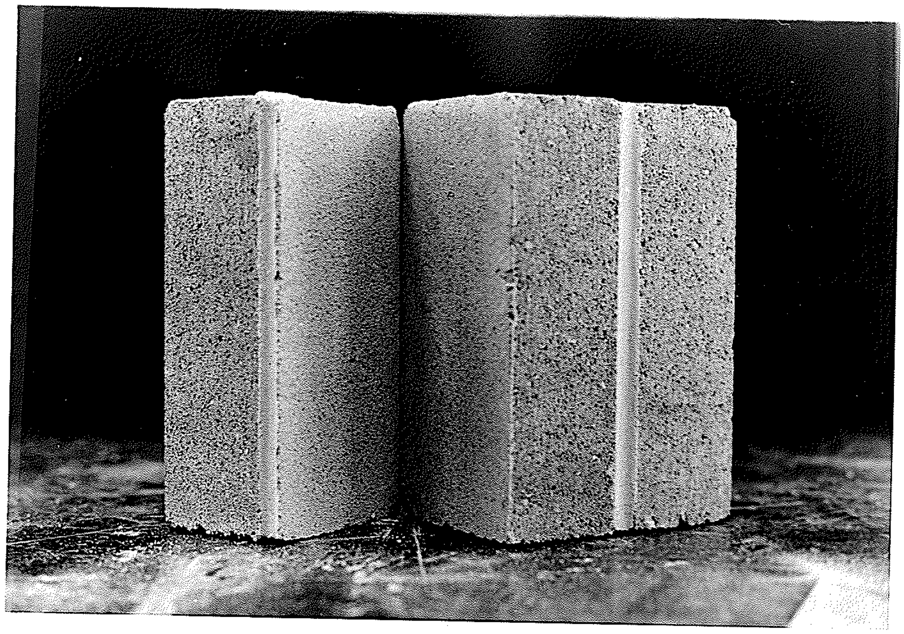

FIG. 80 TYPICAL BOND FAILURE IN SHEAR TEST

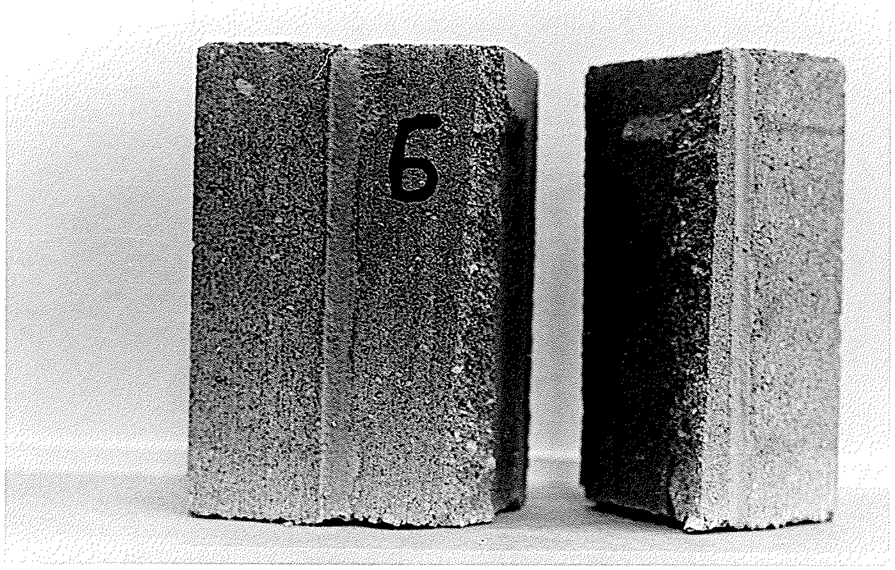

FIG. 81 SHEAR FAILURE OF MORTAR TYPE 6

(2) sufficient shear strength

(3) sufficiently long "board time".

Further research is required to develop a mortar which meets these requirements.

CONCRETE SHEAR TESTS

The purpose of these tests was to obtain a ratio of shear strength of concrete to compressive strength which could be related to the average value of 27.9% from the core-filled beam tests.

Three direct shear tests were run on core-filled standard blocks. In each sample, three blocks were laid one on top of the other and a thin layer of polyethelene was set between the joint faces to minimize frictional resistance. Both cores were filled with the same concrete mix as used in the beam tests. The samples were allowed to cure 14 days and then tested in double shear as shown in Figure 82. Each block had one flat end which was used as a bearing surface. The results are listed in Table 26. In determining the shear strength, the force at which the small core area failed was used. The failure of the larger core area was not purely shear. This occurred with the middle block totally unsupported and it was evident that considerable bending stresses were present. Comparative failures of the small core area (shear) and the large core area (shear plus bending) are shown in Figure 83.

Thus, on the basis of these tests, a ratio of shear to compressive strength of concrete of 23.7% was found. This value is very close to the ratio of 27.9% as obtained from the beam tests.

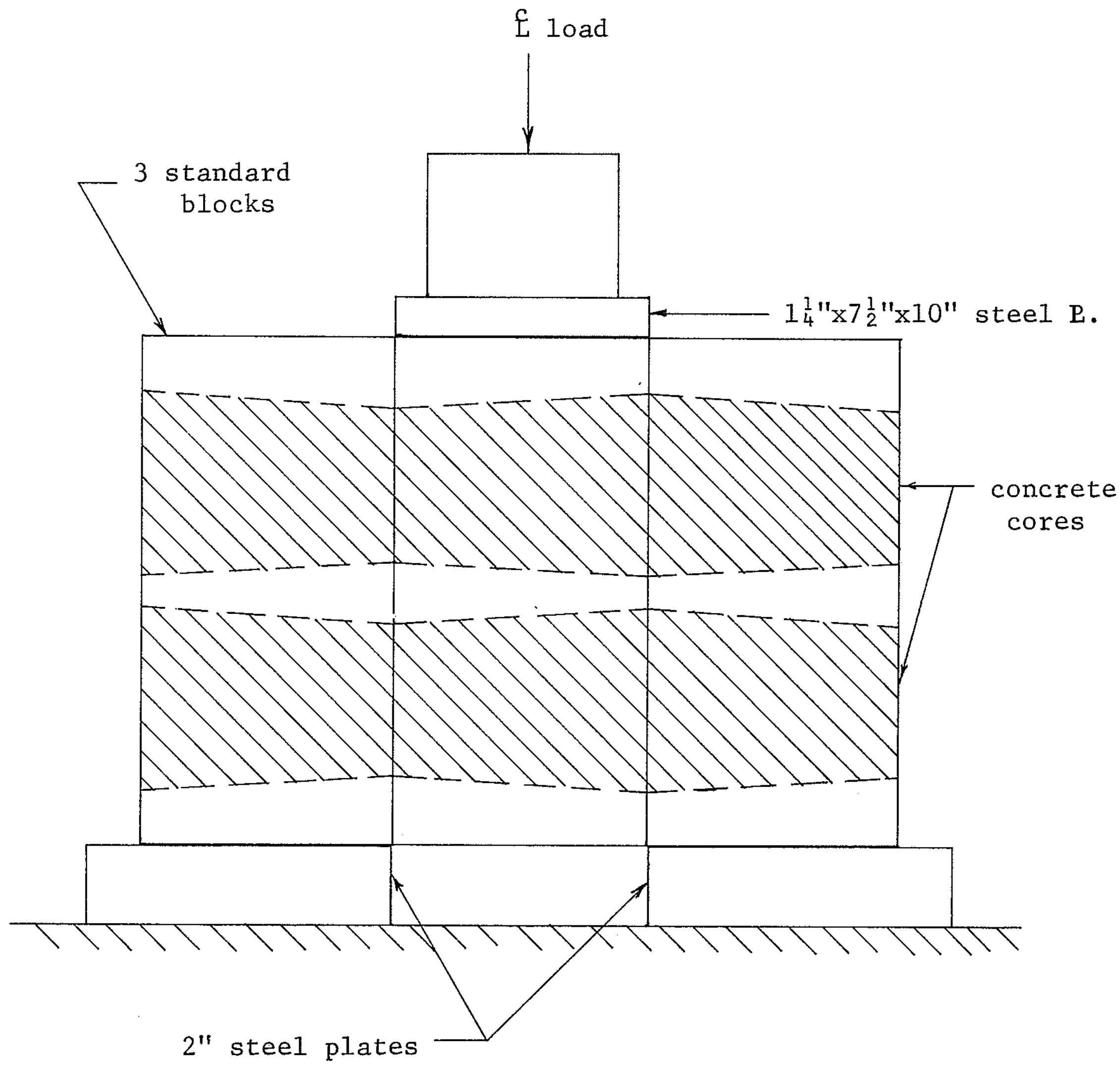

FIG. 82 DIAGRAM OF SHEAR TEST ON CONCRETE

TABLE 26

SHEAR TEST RESULTS ON CONCRETE

COMPRESSIVE STRENGTH (AVG. OF 4 CYLINDER TESTS) =

2,360 P.S.I. AT 14 DAYS

SPECIMEN NO.	FAILURE LOAD OF SMALL HOLE SIDE (POUNDS)	FAILURE LOAD OF LARGE HOLE SIDE (POUNDS)	ULTIMATE SHEAR STRESS IN CONCRETE v_{cu} (P.S.I.)	$\frac{v_{cu}}{f'_c}$ (%)
1	23,000	36,900	564	23.7
2	23,500	37,940	577	24.5
3	21,790	34,800	535	22.8
		AVERAGE =	559	23.7

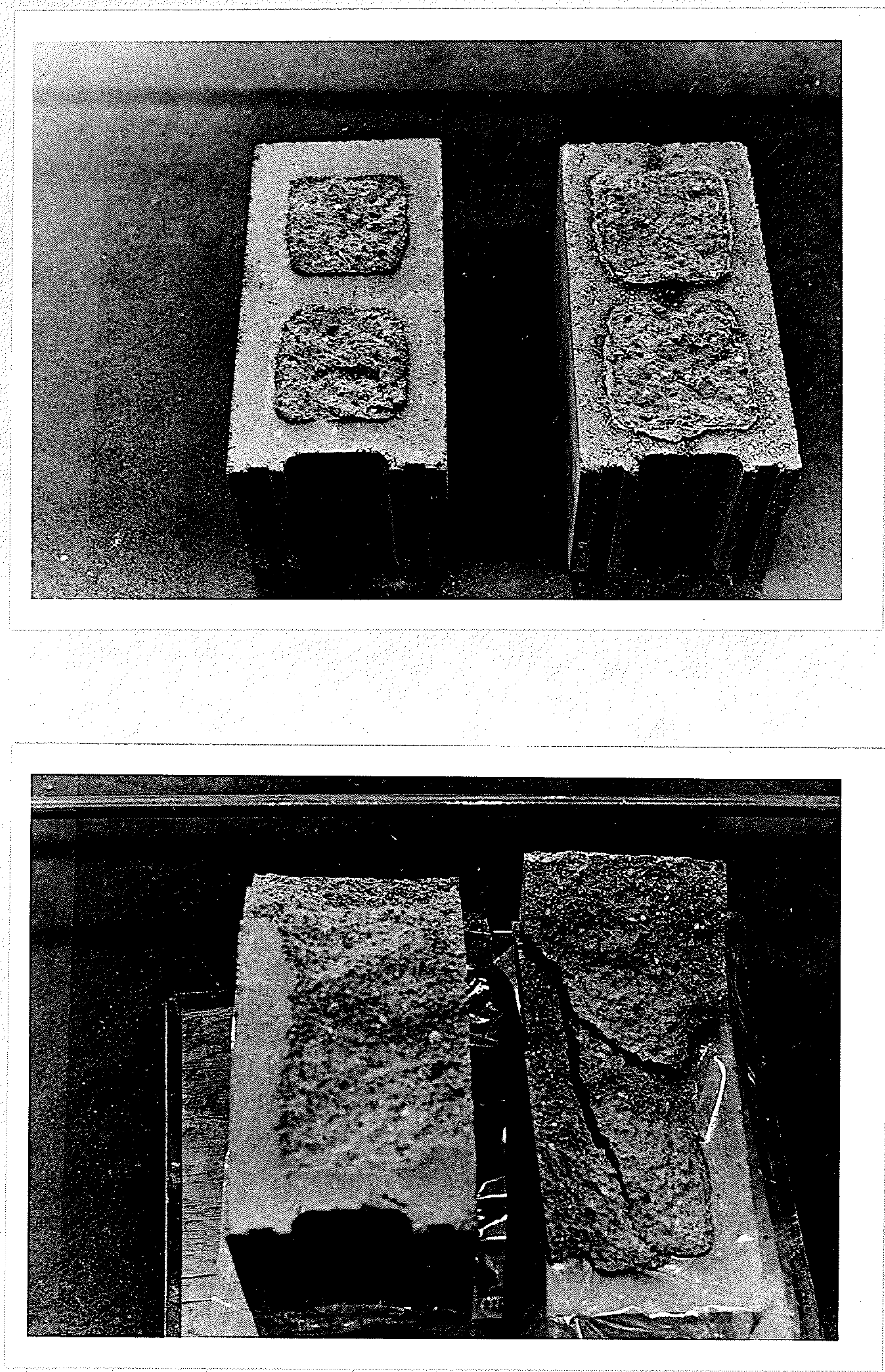

FIG. 83 SHEAR AND SHEAR-BENDING FAILURES OF CONCRETE