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RESEARCH INVESTIGATION OF THE
STRUCTURAL PROPERTIES OF KORFIL HI-R
CONCRETE MASONRY

RESEARCH INVESTIGATION OF THE STRUCTURAL PROPERTIES OF KORFIL HI-R CONCRETE MASONRY

A REPORT TO:

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March 1986

ABSTRACT

Korfil Hi-R units are pre-insulated concrete masonry units that are designed to be laid up in the conventional manner to form an energy conserving, structural wall system that is also resistant to water penetration. The units are configured such that the cross webs are aligned vertically when laid up in running bond or stack bond. Walls of Korfil Hi-R units may be unreinforced, reinforced with grout placed at intervals, or fully grouted. Insulating inserts are designed so that vertical and horizontal mortar joints are insulated. Cross webs are reduced in height for a portion of their length in order to reduce the area of through-wall heat paths.

The objective of the research described in this report is to determine the structural properties of Korfil Hi-R concrete masonry wall system in order to insure its proper design. The research includes flexural tests on reinforced and unreinforced walls and beams, compression tests on walls and prisms, diagonal tension (shear) tests on walls, and test of component materials.

The research indicates that the Korfil Hi-R wall system provides excellent wall deflection characteristics for flexural, shear and compressive loading. Designed in accordance with the recommendations of this report, the resultant factors of safety for the Korfil Hi-R systems are comparable to conventional concrete masonry construction.

RESEARCH INVESTIGATION OF THE STRUCTURAL PROPERTIES OF KORFIL HI-R CONCRETE MASONRY

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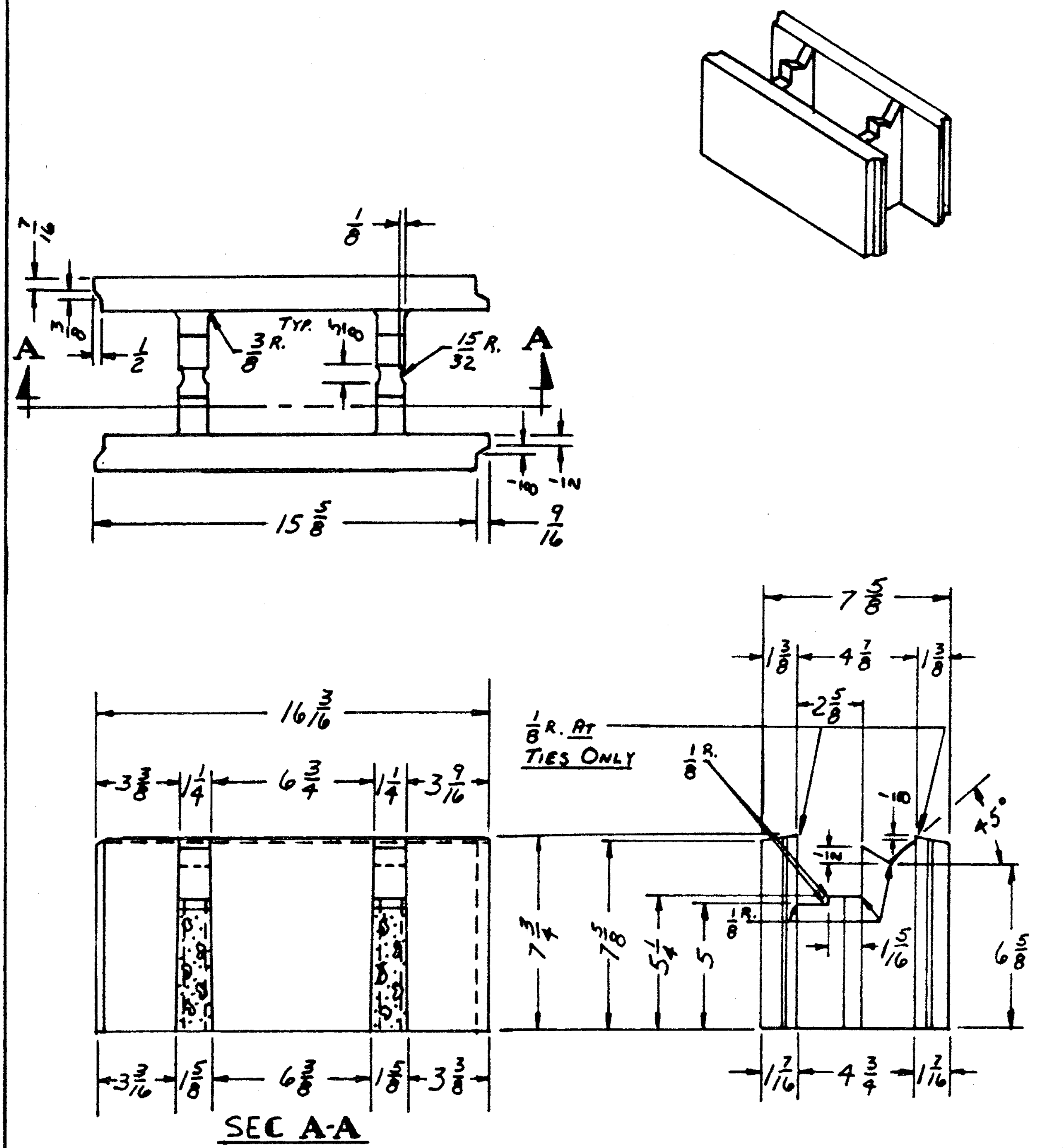
1.0 DESCRIPTION OF KORFIL HI-R MASONRY WALL SYSTEM

The Korfil Hi-R masonry wall system is designed to be an energy conserving, preinsulated structural wall system, which is resistant to moisture penetration and supports both vertical and lateral loads. The stepped head joint of the Hi-R unit interlocks with adjacent units and is designed to be installed faster than conventional masonry units. The two cross webs of the Hi-R unit are designed to align vertically with the cross webs in adjacent courses when the units are laid in either a running bond or a stack bond pattern. This web alignment forms a series of continuous vertical spaces within the wall separated by cross webs. The wall may be partially grouted in which cross webs adjacent to the grouted cell are mortared to confine the grout, or the wall may be fully grouted. Hi-R walls may be reinforced both vertically and horizontally. The top of each web is notched so that horizontal bar reinforcement can be placed in any course. The horizontal reinforcement is positioned and secured against displacement by the notched cross webs. The notched cross webs can also accommodate conduit placed horizontally within the wall.

Two expanded polystyrene inserts (insert "A" and insert "B") are installed in each Hi-R unit at the block manufacturing plant. When Korfil Hi-R units are installed in the wall the two layers of insulating inserts are offset so that they interlock with adjacent units forming a continuous insulation layer which is interrupted only by the depressed webs connecting the face shells of the Hi-R units. The insulation projects across both head and bed joints between units, and continues through bond beam courses and past vertical grout cells. The interlock and the bevelled shape of the edge of the insert where it adjoins adjacent inserts is designed to direct moisture to the exterior of the wall and prevent moisture penetration through the insulation layer.

Illustration of Korfil Hi-R Unit

Figure 1-1



2.0 SCOPE

The objective of this research is to determine the structural properties of the Korfil Hi-R masonry wall system through testing of assemblages and component materials. The research includes flexural tests on reinforced and unreinforced walls and beams, compressive tests on walls and masonry prisms and diagonal tension (shear) tests and tests of component materials including Hi-R units, mortar and grout. A synopsis of the test program is listed in Table 2-1. Details of the test variables for each of the assemblages tested are shown in Tables 2-2 through 2-6.

TABLE 2-1
SYNOPSIS OF TEST PROGRAM

Test Description:	Test Series	Number of Tests	Variables/Load Combinations Tested	Repetitions of Each Variable/ Load Combination
Flexural Tests on Beams:	A	12	4	3
Flexural Tests on Walls:	B	15	5	3
Compressive Tests on Walls:	C	9	3	3
Diagonal Tension (Shear) Tests:	D	9	3	3
Masonry Prism Tests:	E	12	4	3

TABLE 2-2
Synopsis of Flexural Tests on Beams - Test Series A

General Description of Test Series A:
Each specimen consisted of one course of three Hi-R units forming a beam (lintel) 48 inches in length which was grouted. The beam was centered on reaction points 32 inches apart. Load was applied at the third points. An illustration of the test is shown in Figure No.3.1.

Description of Test Variables:	Test Designation			
	A.1.1 A.1.2 A.1.3	A.2.1 A.2.2 A.2.3	A.3.1 A.3.2 A.3.3	A.4.1 A.4.2 A.4.3
Reinforcement one bar, bar size:	#4	#6	None	#4
Approximate Depth of Reinforcement:	6" ±	6" ±	--	2" ±
Insulation- Insert "A":	Installed	Installed	Installed	Removed
Insert "B":	Installed	Installed	Installed	Removed

TABLE 2-3
Flexural Tests on Walls - Test Series B

General Description of Test Series B:
Wall panels were 4 feet wide, and 8 feet in height and constructed of Korfil Hi-R units. Load was applied against the face of the wall. The span between reactions was 90 inches.
An illustration of the test is shown in Figure No.3.2.

Description of Test Variables:	Test Designation B.1.1 B.1.2 B.1.3	B.2.1 B.2.2 B.2.3	B.3.1 B.3.2 B.3.3	B.4.1 B.4.2 B.4.3	B.5.1 B.5.2 B.5.3
Grout Spacing Inches:	24	24	24	24	None
Vertical Bar Size and Spacing:	#4@24"	#4@24"	#6@24"	#6@24"	None
Depth of Reinforcement:	2 1/2"±	5"±	2 1/2"±	5"±	+ -
Bond Beams-					
Top Course:	#4 bar	#4 bar	#4 bar	#4 bar	None
Mid Height:	#4 bar	#4 bar	#4 bar	#4 bar	None
Bottom:	#4 bar	#4 bar	#4 bar	#4 bar	None

TABLE 2-4
Compression Tests on Walls - Test Series C

General Description of Test Series C:
Wall panels were 4 feet wide, and 8 feet in height and constructed of Korfil Hi-R units. Compressive load was applied along the top of the wall at an eccentricity of t/6 from the centerline of the wall toward the insulation side.
An illustration of the test is shown in Figure No.3.3.

Description of Test Variables:	Test Designation C.1.1 C.1.2 C.1.3	C.2.1 C.2.2 C.2.3	C.3.1 C.3.2 C.3.3
Grout Spacing:	24"	Full	None
Vertical Bar Size and Spacing:	#4@24"	#4@24"	None
Bond Beams-			
Top Course:	#4 bar	#4 bar	None
Mid Height:	None	None	None
Bottom:	#4 bar	#4 bar	None

TABLE 2-5
Diagonal Tension (Shear) Tests-Test Series D

General Description of Test Series:
Wall panels were 4 feet by 4 feet and constructed of Korfil Hi-R units. An illustration of the test is shown in Figure No.3.4.

	Test Designation		
Description of Test Variables:	D.1.1 D.1.2 D.1.3	D.2.1 D.2.2 D.2.3	D.3.1 D.3.2 D.3.3
Grout Spacing:	24"	Full	None
Vertical Bar Size and Spacing:	#4@24"	#4@24"	None

TABLE 2-6
Synopsis of Masonry Prism Tests - Test Series E

General Description of Test Series:
Each prism was 16 inches in height and consisted of two Korfil Hi-R' units laid in stack bond. An illustration of the test is shown in Figure No.3.5.

	Test Designation			
Description of Test Variables:	E.1.1 E.1.2 E.1.3	E.2.1 E.2.2 E.2.3	E.3.1 E.3.2 E.3.3	E.4.1 E.4.2 E.4.3
Grout:	Solid	None	None	Full
Motar Bedding:	Faceshell	Faceshell	Full	Full
Insulation-				
Insert "A":	Removed	Installed	Installed	Installed
Insert "B":	Removed	Installed	Installed	Installed

3.0 CONSTRUCTION, CURING AND TESTING PROCEDURES

3.1 Tests of Component Materials

Korfil HI-R Units

Korfil concrete masonry units were tested in accordance with ASTM Method C 140, for compressive strength and moisture absorption properties. Five specimens were used for these tests instead of three required by ASTM C 140. Results are listed in Table 4-1.

Mortar

Mortar used in constructing specimens was Type S, portland cement-lime mortar complying with the requirements of ASTM C 270. Proportions of materials were: 1 part portland cement, 0.37 parts lime, 3.8 parts sand. Mortar cubes for compressive strength tests were obtained by two methods: (1) specimens were molded from mortar taken directly from the mortar pan; (2) specimens were molded from mortar which was first spread on the face of a Korfil Hi R unit to a depth of 3/8" and allowed to remain for 60 seconds before being removed for placement into the cube molds. Mortar cubes were kept in the molds in a moist room for 48 hours after which they were removed from the molds and kept in the moist room until tested. Compressive strength was determined in accordance with ASTM C 109. Water retention was determined in accordance with ASTM C 91. Results of the mortar tests are listed in Table 4-2.

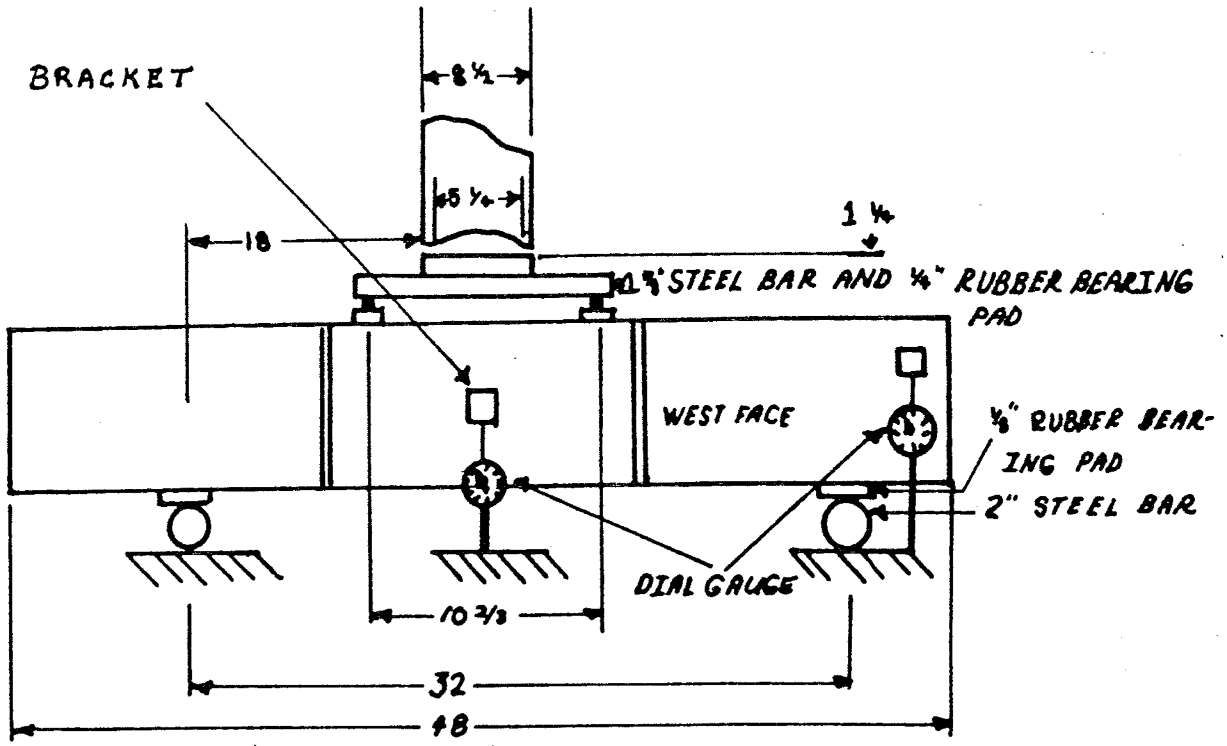
Grout

Grout used in constructing the test specimens was fine grout mixed in accordance with ASTM C 476. Materials were proportioned to produce a grout strength comparable to the strength of the masonry units. The initial grout mix consisted of 1 part portland cement, 5 parts sand with sufficient water to produce a 9 to 10 inch slump. In order to improve the flowability of grout, the mix was modified after the first six batches by adding 0.05 parts of lime by volume. The resulting mix design for grout batches 7 through 30 consisted of 1 part portland cement, 0.05 parts lime and 5 parts sand with sufficient water to produce a 9 to 10 inch slump. Compressive strength tests of grout were performed in accordance with ASTM C 1019. Specimens for compressive strength tests were 3" x 3" x 6" high made in molds formed from Korfil units similar to those used in the tested assemblages. Tests for slump were performed in accordance with ASTM C 143. Results of the grout tests are listed in Table 4-3.

3.2 Flexural Tests on Beams

Beams were made by laying three units end-to-end to form an assemblage having nominal dimensions of 8" wide by 8" high by 48" long. Beams were grouted and reinforced as described in Table 2-2, and were cured in laboratory air until tested. Flexural testing was performed by placing specimens horizontally on supports 32 inches apart, resulting in simply supported beams. Load was applied at the third points of the span at a uniform rate. (See Fig. 3-1.) Deflections at mid-span at various load increments were recorded. Results are listed in Table 4-4 and Table 4-5.

Illustration of Flexural Test on Beams	Figure 3-1



ELEVATION VIEW

3.3 Flexural Tests on Walls

Wall specimens were constructed by an experienced mason under contract to National Concrete Masonry Association Research and Development Laboratory. The mason was assisted by NCMA laboratory personnel. Block were laid up in center running bond with faceshell mortar bedding. As each course was laid the inner insulating insert ("B" insert) was pushed downward to form a thermal insulating barrier over the horizontal mortar joint below. Nominal dimensions of wall panels were 4 feet wide by 8 feet high by 8 inches thick. Details of construction for the five different wall types tested in flexure are summarized in Table 2-3. Where a bond beam course was incorporated in the wall, "B" inserts were removed from the units comprising the bond beam prior to grouting. Wire mesh was used to confine grout to the bond beam course. Vertical reinforcement was positioned in the center of the grout space by using a positioner placed in the mortar joint at the top of the first course of block. Grouting was done in four foot lifts. After pouring each lift, grout was vibrated using a 1-inch "pencil" vibrator. After approximately 10 minutes grout was reconsolidated. During initial vibration of the second lift, care was taken to assure that the vibrator penetrated into the grout from the first lift. Approximately 20 minutes elapsed between successive lifts.

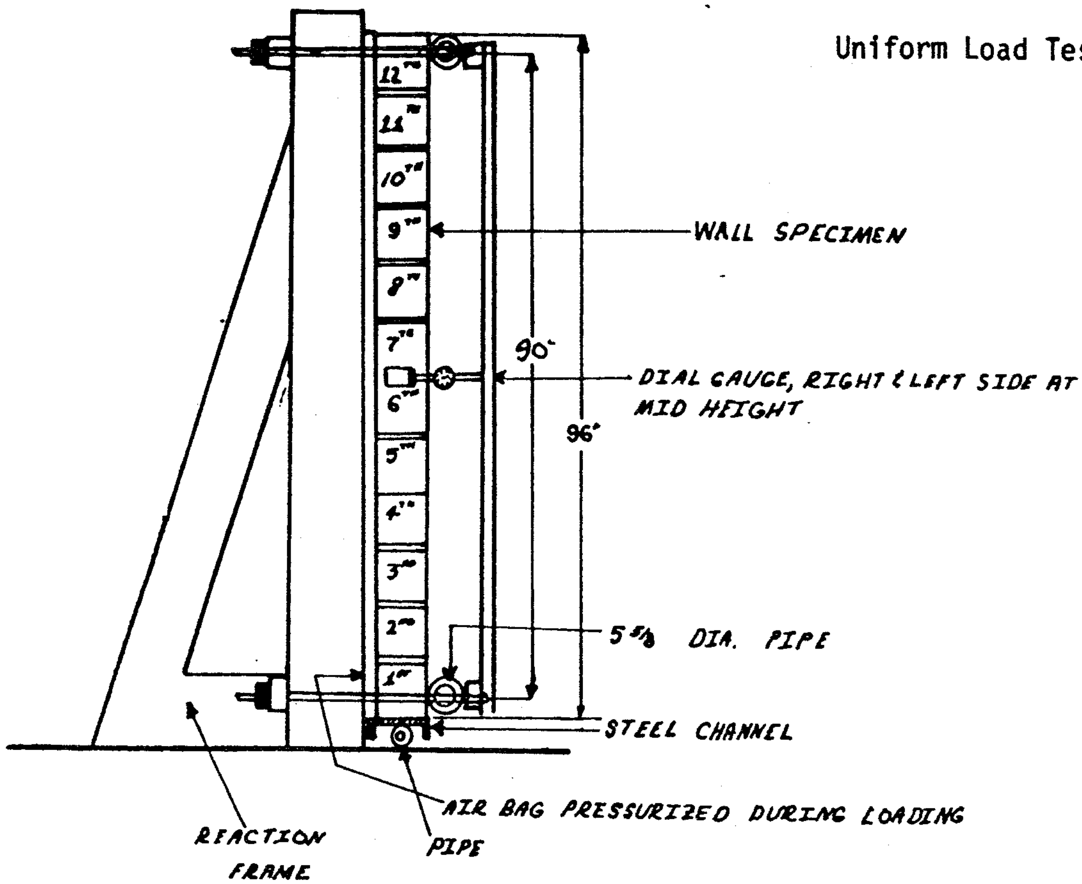
Following construction, the walls were stored in the laboratory for at least 28 days. Flexural tests were conducted in accordance with applicable provisions of ASTM E-72 (flexural strength in the vertical span). The test frame was placed in position and specimens were secured for testing as illustrated in Figure 3-2. A steel channel supporting each wall rested on a round pipe to permit free rotation at the base of the wall. Padded 6 inch steel pipes served as reactions at the top and bottom of the wall. The span between reaction points was 90 inches. Dial gauges graduated to one-thousandth of an inch were used to measure deflection at mid-span along either edge of the specimen.

Uniform transverse load on wall specimens was generated by pressurizing an air bag sandwiched between the test wall and the test frame. Pressure was measured by a U-tube water manometer. Deflection gauge readings were initialized to zero and the bag was pressurized to the first increment of load. Load was then held; deflection readings were recorded; the bag was then deflated to zero pressure and "set" deflection readings recorded. This procedure was repeated incrementally, increasing the pressure for each sequence until failure of the specimen was observed or until capacity of the uniform load frame was reached. Capacity was achieved when either the load exceeded 541 pounds per square foot or deflections became excessive, resulting in possible damage to the air bag (approximately 1 1/4 inches at elevated loads).

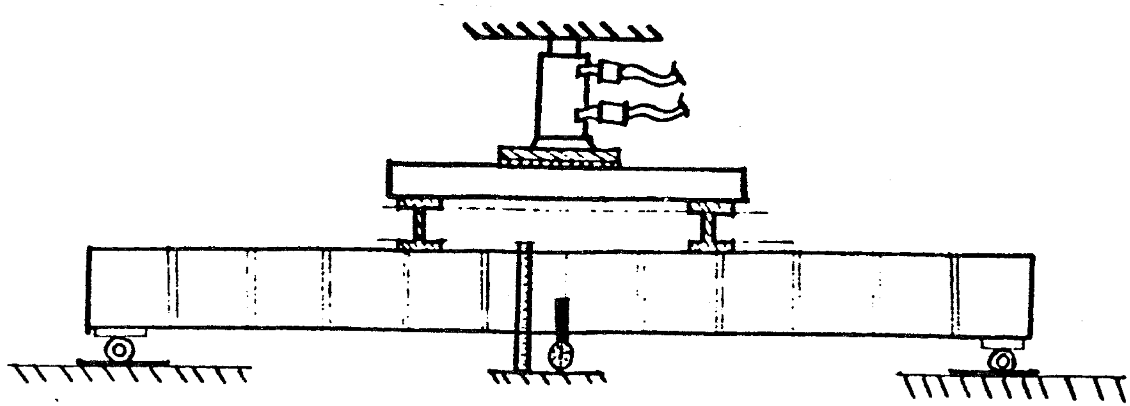
Specimens exceeding capacity of the uniform flexural frame were removed and placed in a third-point loading apparatus (see Figure 3-2. Load was generated through a 30 ton hydraulic ram using an electrically operated pump. A 100,000 pound load cell was used to monitor load at each increment. Dial gauges graduated to one-thousandth of an inch were used to measure mid-span deflections for the first 2 inches of movement. For deflections in excess of 2 inches, steel scales graduated to one-sixteenth of an inch were employed. Gauges were initialized to zero; however, permanent set (from uniform loading), dead load deflections, and superimposed dead load deflections were recorded and included in load-deflection results. Results of the flexural tests on walls are listed in Table 4-6 and 4-7.

Illustration of Flexural Tests on Walls	Figure 3-2

Uniform Load Test



Third Point Load Test

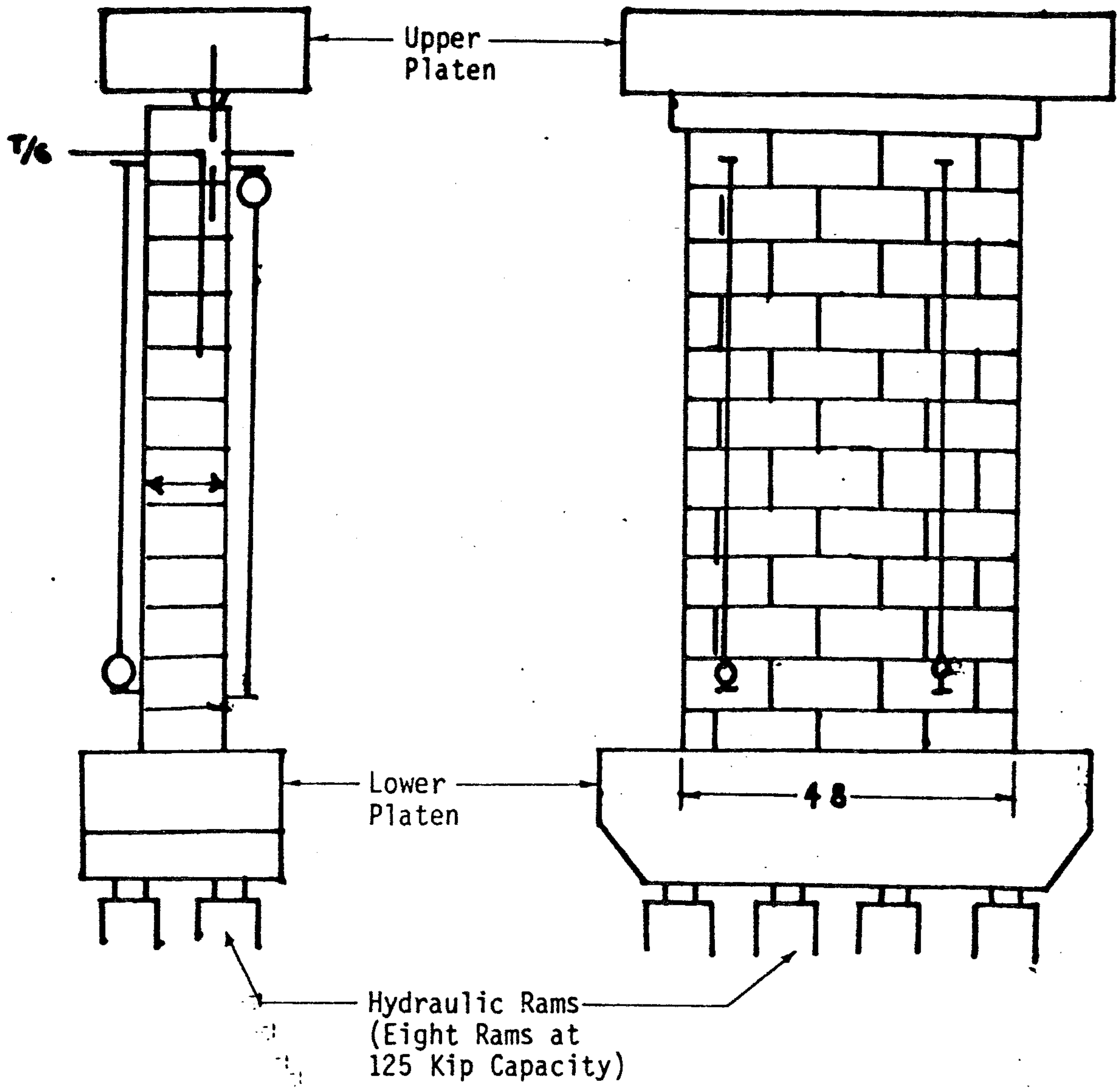


Span = 90 inches

3.4 <u>Compression Tests on Walls</u>

Wall panels for compression strength tests were nominal 4 feet wide by 8 feet high by 8 inches thick. Construction of the walls was the same as for the flexural wall specimens except the bond beam at mid-height of wall was omitted. Three variables were tested as summarized in Table 2-4. The compression tests were conducted in accordance with ASTM E-72. Following construction, the walls were stored in the laboratory to the age shown. Specimens were placed in a calibrated load frame capable of generating two million pounds of force. The tops of the compression walls were capped with "hydrostone" prior to placement in the test frame, and the bottoms of the walls were capped in the machine under a preload of approximately four thousand pounds. A 5 inch diameter, solid steel half-round was placed at the top of the wall and offset a distance equal to one-sixth the wall thickness from the center to apply the eccentric compressive load. Eccentricity for all specimens was towards the insulated face (See Figure 3-3). Shortening of the specimen was measured between points 90 inches apart. A total of four mechanical gauges were mounted to measure shortening, two on each face of the wall and located along the centerline of the second core from either edge. Dial gauges graduated to one-thousandth of an inch were read at each load increment. Gauges were removed prior to specimen failure. Maximum load carried by each specimen was recorded. Results of the compression tests on walls are listed in Table 4-8.

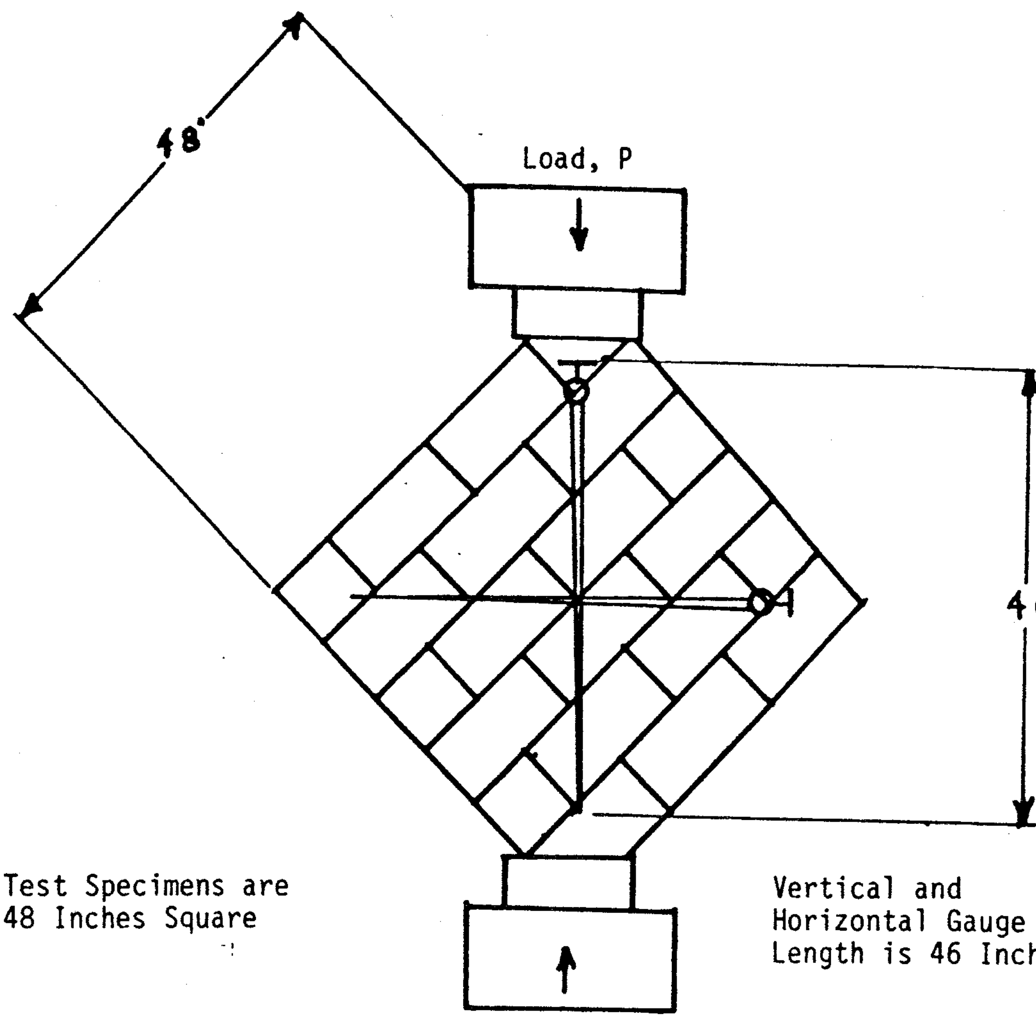
Illustration of Compression Test on Walls | Figure 3-3



3.5 Diagonal Tension (Shear) Tests

Specimens for diagonal tension (shear) tests were constructed the same as the flexural and compressive strength test walls except that the nominal dimensions were 4 feet wide by 4 feet high, and bond beams were not used. Construction details and tested variables are listed in Table 2-5. Tests were conducted in accordance with ASTM E 519. Following construction, wall specimens were stored in the laboratory until tested. Mechanical gages, graduated to one thousandth of an inch were used to measure elongation or shortening along the diagonals as illustrated in Figure 3-4. Gauge lengths were 46 inches in both directions. Deformations were recorded at the various load increments. Results of the tests are listed in Table 4-9.

Illustration of Diagonal Tension (Shear) Tests	Figure 3-4



Test Specimens are
48 Inches Square

Vertical and
Horizontal Gauge
Length is 46 Inches

3.6 Masonry Prism Tests

Masonry prisms for compression tests were made by laying up two units in stacked bond to form specimens having nominal dimension of 8" wide by 16" Mortar joints were tooled with a concave jointer. The four series of prisms tested included three prisms in each series as summarized in Table 2-6. The net area of each prism series was varied in order to establish the effect that various mortar bedding and grout combinations may have on the compressive strength of masonry. Results of the prism compressive strength tests are listed in Table 4-10.

4.0 TEST RESULTS

4.1 Component Materials

Korfil HI-R Units

ASTM C 90, standard specification for Hollow Load Bearing Concrete Masonry Units lists minimum requirements for compressive strength, absorption, face shell and cross web thickness in addition to other provisions. Korfil Hi-R units used in construction of the test specimens were similar in physical properties to conventional hollow units except for the cross webs which are not full-height for a portion of their length. For this reason a comparison of web thickness properties between ASTM C 90 requirements and Korfil Hi-R units is not relevant. In the following Table physical properties of HI-R units are compared with pertinent ASTM C 90 requirements.

Table No. 4-1
Masonry Unit Properties

	Korfil HI-R Units	Lightweight Units, ASTM C-90 Grade N
Unit weight,	101 pcf	85-105 pcf
Absorption, maximum	13 pcf	18 pcf
Compressive minimum		
Gross Area	1220 psi	1000 psi
Net Area	2810 psi	--
Percent Solid	43.3%	--
Face Shell Thickness, minimum	1.36 In	1.25 In
Web Thickness, minimum	--	1"
Equiv. Web Thickness, minimum	--	2.25 In

Mortar

Mortar used throughout the testing program consisted of 1.00 part portland cement by volume, 0.37 parts lime, and 3.76 parts sand, which meets the proportion specification requirements for Type S mortar in accordance with ASTM C 270. The measured water retention for the mortar was 83 percent, which exceeds the minimum ASTM C 270 property specification requirement of 75 percent. Compressive strength of 2-inch cubes of mortar sampled directly from the mortar pan averaged 3084 psi with a range in strength between 2875 and 3400 psi and a coefficient of variation of 8.0. Cubes made from mortar that was subjected to the suction of the block for one minute average 3256 psi in compressive strength with a range of 3100 to 3383 psi and a coefficient of variation of 5.3.

TABLE NO. 4-2
Summary of Tests On Compressive Strength of Mortar

Date Sampled	Cube Strengths Sampled From Pan PSI	Sampled After Block Suction PSI
6/6	--	3308
6/7	2875	3100
6/9	2917	3142
6/13	3142	3383
6/14	3400	3350
Average:	3084	3257

Grout

Compressive strength tests were performed on nine batches of grout taken during the construction of the specimens. Tests were performed at various specimen ages as shown in Table No. 4-3. The average strength of grout specimens that were at least 28 days old was 3489 psi with a range from 3633 to 4789 psi and a coefficient of variation 12.3. The following table summarizes the compressive strength of the grout specimens as listed in order of age at test:

Table 4-3
Summary of Test on Compressive Strength of Grout

Sample	Date Sampled	Age At Test, Days	Compressive Strength psi
K-1	6/15/85	3	1911
K-4	6/26/85	5	3222
K-6	6/28/85	34	3633
K-5	6/27/85	35	3911
K-9	7/17/85	37	3778
K-2	6/15/85	47	3733
K-3	6/15/85	47	3944
K-8	7/5/85	49	4789
K-7	7/2/85	52	4111

4.2 Results of Flexural Tests on Beams

Results of Flexural Tests on reinforced beams composed of Korfil Hi-R units are summarized in Table 4-4. Included are calculated bending moments at maximum loads as well as flexural and shear stresses at maximum load. Both load and deflection was recorded for flexural tests on beams in Test Series A. Deflections were based on the difference between the mid-span deformation and the deformation at the reactions. However, no deformation gauges were mounted adjacent to the north reaction; therefore, the deformation at the north reaction was assumed to equal the deformation at the south reaction for determining deflection values. Deformation gauges were mounted on both the grout and the insulation side of the specimens at mid-span and adjacent to the south reaction. Deflections were determined for both the grouted side and the insulated side of the beam. For test series A.1 and A.2 having insulating inserts and with reinforcement located approximately 6" from the compression face, it was observed that on the insulated side of the specimen, failure generally occurred by diagonal cracking from the area of load application to the mortar joint and extending vertically downward to the bottom of the beam. Where cracking occurred on the grouted face of the beam, it extended diagonally from the reactions to the top middle portion of the beams. However, in two of the specimens no cracks were observed on the grouted side of the beam during testing. Apparently, when the reduced-height portion of the webs of the middle block in the beams sheared off, separating the face shell from the grouted portion of the beam, the loading head rotated toward the insulated side, causing the face shell to further separate from the grouted portion. Because of this separation, averaging deflection value for the grouted and insulated side of the beams would not be representative of the average deflection of the beams. Test Series A-4, with insulating inserts removed, and reinforcement depth was about 2 inches, exhibited crushing in the compression zone of the beams.

TABLE 4-4
Results of Flexural Tests on Reinforced Beams[1]
Test Series A.1, A.2 and A.4

Test	Maximum Load P Lbs	Moments[2] At Maximum Load M In.-Lbs	Bar Depth d Inch	Beam Width b Inch	Flexural Stress[3] At Maximum Load f_b PSI	Shear[4] At Maximum Load V Lbs	Shear Stress v_m PSI
A.1.1	6937	37557	5.88	5.00	1021	3537	120
A.1.2	9052	48851	5.75	5.00	1375	4595	160
A.1.3	8628	46587	6.16	5.00	1179	4383	142
				Average:	1192		141
A.2.1	12827	69010	6.00	5.00	1307	6482	216
A.2.2	11342	61080	5.75	5.00	1240	5740	200
A.2.3	6165	33435	5.88	5.00	654	3151	107
				Average:	1067		174
A.4.1	5789	31540	2.09	7.63	3425	2963	186
A.4.2	6212	33798	2.16	7.63	3482	3175	193
A.4.3	5958	32442	2.38	7.63	2863	3048	168
				Average:	3257		182

Footnotes:
1. Dimensions, loads, span and other parameters of the test are illustrated in Figure No.
2. $M = (P \times 0.445 + 42.8) \times (12 in/ft)$ In - Lbs
3. $f_b = 2\,M / (k\,j\,b\,d^2)$
   Where: $j = 1 - k/3$
   $k = [\,(2\,p\,n) - (p\,n)^2\,]^{1/2} + p\,n$
   $p = A_s / b\,d$

A_s = 0.20 sq.in. for series A-1, A-4
A_s = 0.44 sq.in. for series A-2
$n = E_s / E_m$

4. $V = P/2 + 68.5$ Lbs
5. $v_m = V / (b\ d)$

Results of tests on beams that contained insulating inserts and grout with no reinforcement are summarized in Table 4-5. Observed failure mode was vertical cracking extending upward from the tension face of the beam.

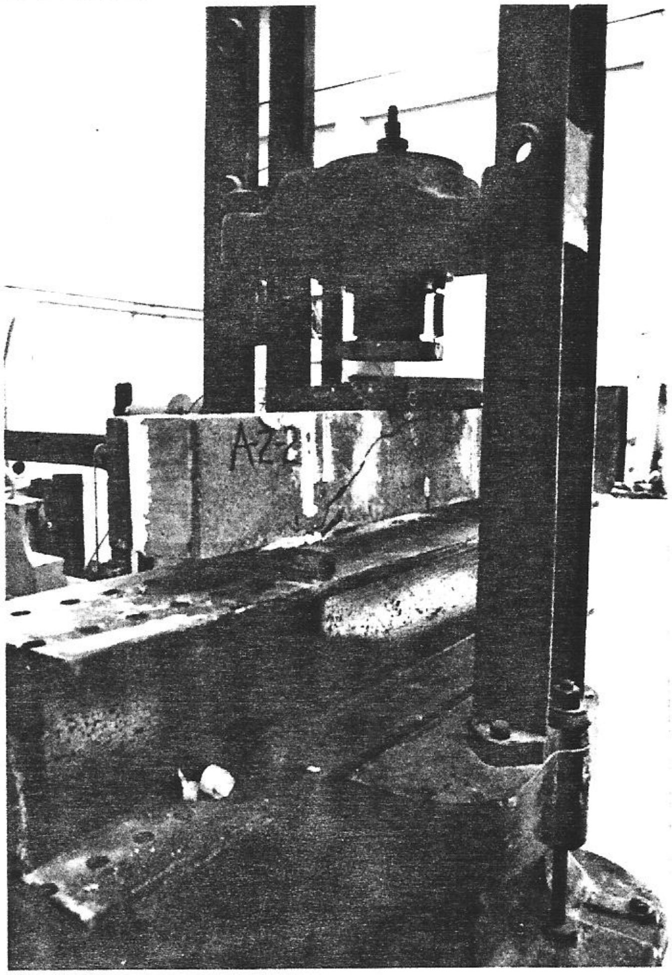
TABLE 4-5
Results of Flexural Tests on Unreinforced Beams[1]
Test Series A.3

Test	Maximum Load P Lbs	Moment[2] At Maximum Load M In.-Lbs	Modulus of Rupture[3] f_r PSI	Shear[4] At Maximum Load W Lbs	Maximum Shear Stress[4] v_m PSI
A.3.1	1935	10847	217	1055	27.2
A.3.2	1696	9570	191	935	24.1
A.3.3	2203	12278	245	1189	30.7
		Average:	218		

Footnotes:

1. Dimensions, loads, span and other parameters of the test are illustrated in Figure No.
2. $M = (P \times 0.445 + 52.2) \times (12in/ft)$ In - Lbs
3. $f_r = M/S$
   $S = b\ h^2 / 6$
   $b = 5$ inches
   $h = 7\text{-}3/4$ inches
   $S = 50$ inch^3
4. $V = P/2 + 87.1$ Lbs
5. $v_m = V/(b\ h)$

Photographs of Flexural Tests on Beams	Figure 4-1



Test Specimen A-2-2, view of west side after testing (i.e. grout side) diagonal (shear) crack extends from north reaction toward load points.

Test Specimen A-2-2 view of east side after testing (i.e. insulation side).

4.3 RESULTS OF FLEXURAL TESTS ON WALLS

In test series B-1 through B-4, the flexural properties of reinforced Korfil HI-R Masonry Walls were investigated. Both the amount of longitudinal reinforcement and load direction were varied in these series of tests. Considering uniform loading (wind pressure type load) against the grouted side of Korfil Hi-R walls, the effect of increasing the area of longitudinal reinforcement from 0.1 square inches per foot of wall to 0.22 square inches per foot of wall resulted in an average increase in flexural resistance of 24,500 in.-lbs. per foot of wall to 43,300 in.-lbs. per foot of wall. Considering uniform loading against the insulated side of Korfil Hi-R walls, the effect of increasing the area of longitudinal reinforcement by the same amount resulted in little change in flexural strength. Table No. 4-6 summarizes the results of flexural tests on reinforced walls.

Table 4-6
Results of Flexural Tests on Reinforced Walls

Test	Maximum Uniform Load PSF	Mid-Span Moment At Maximum Load in.-lb./ft	Bar Depth d,in.	Shear at Reactions At Maximum Load Lbs./ft.
B.1-1	291	24553	2.87	1091
B.1-2	250	21094	2.50	938
B.1-3	333	28097	2.66	1294
B.2-1	530	44719	5.00	1988
B.2-2	489	41259	4.74	1834
B.2-3	546	46069	5.08	2048
B.3-1	541	45647	2.69	2029
B.3-2	458	38644	2.65	1718
B.3-3	541	45647	2.63	2029
B.4-1	499	42103	4.70	1871
B.4-2	541	45647	5.13	2029
B.4-3	541	45647	4.94	2029

In test series B.5, the flexural properties of non-reinforced Korfil Hi-R walls were investigated. These walls were subjected to a uniform lateral load (wind pressure type loading). The mode of failure was bond separation at the block-mortar interface along a mortar bed joint. Maximum recorded mid-span deflection ranged from less than 0.001 to 0.009 inches in the test series. Test specimen B-5-1 was tested with the uniform load applied against the uninsulated side of the wall, while test specimen B-5-2 and B-5-3 were loaded against the insulated side of the wall. Table No. 4-7 summarizes flexural test results of non-reinforced Korfil Hi-R walls.

Table 4-7
Results of Flexural Tests on Non-Reinforced Walls

Test	Maximum Uniform Load PSF	Mid-span Moment At Max Load in-lb/ft	Modulus[1] Of Rupture PSI	Shear At Max, Load Lbs
B.5-1	62	5231	61.3	233
B.5-2	52	4388	51.4	195
B.5-3	62	5231	61.3	233

1. Based on Modulus of Rupture F_r = M/S, where,
   M = mid-span moment at maximum load, in-lb/ft
   S = 85.4 in_3/ft (See Section 4.1)

FIGURE NO. 4-2
FLEXURAL WALL TEST SERIES B-1
MOMENT VS DEFLECTION

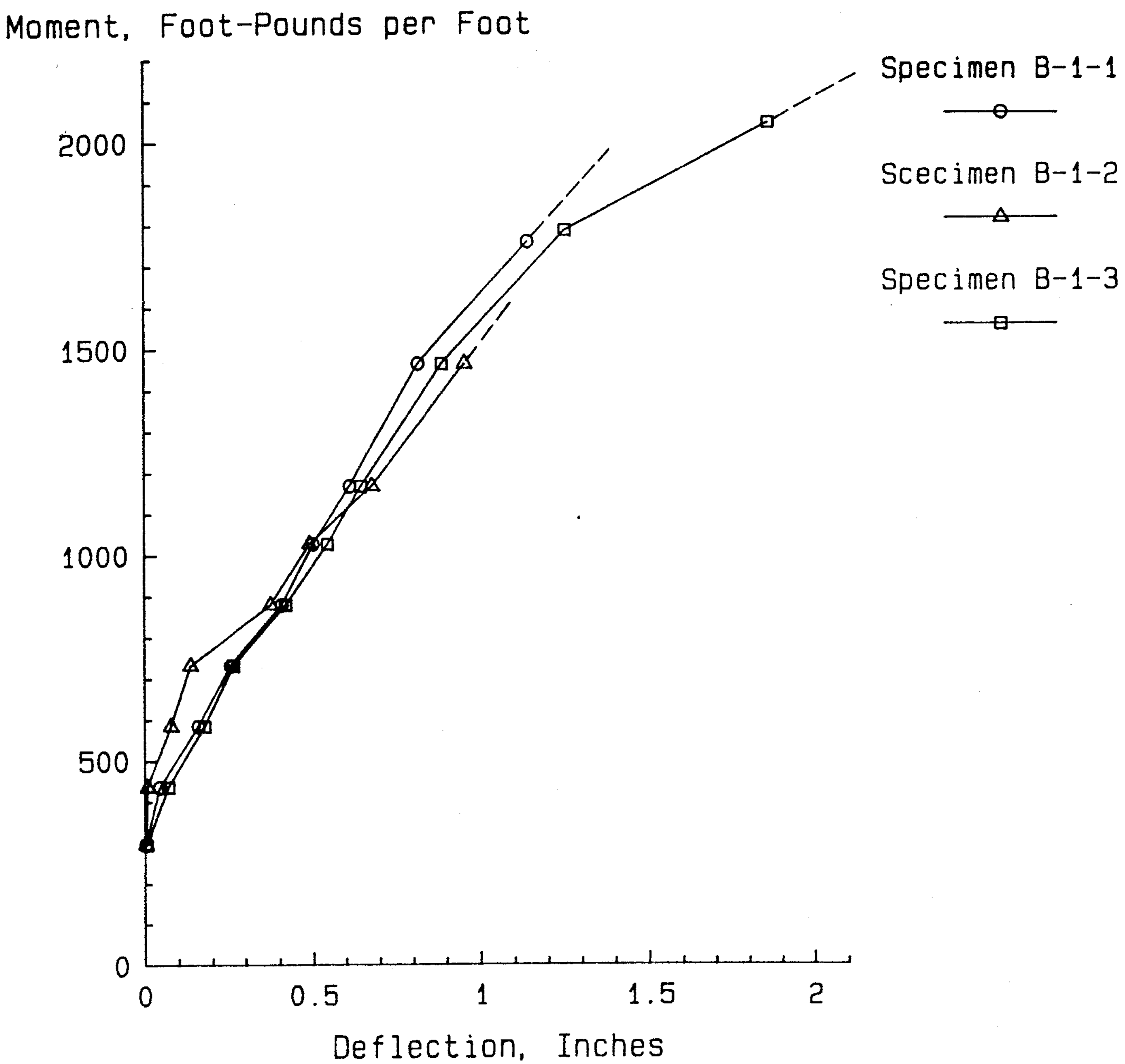


FIGURE NO. 4-3
FLEXURAL WALL TEST SERIES B-2
MOMENT VS DEFLECTION

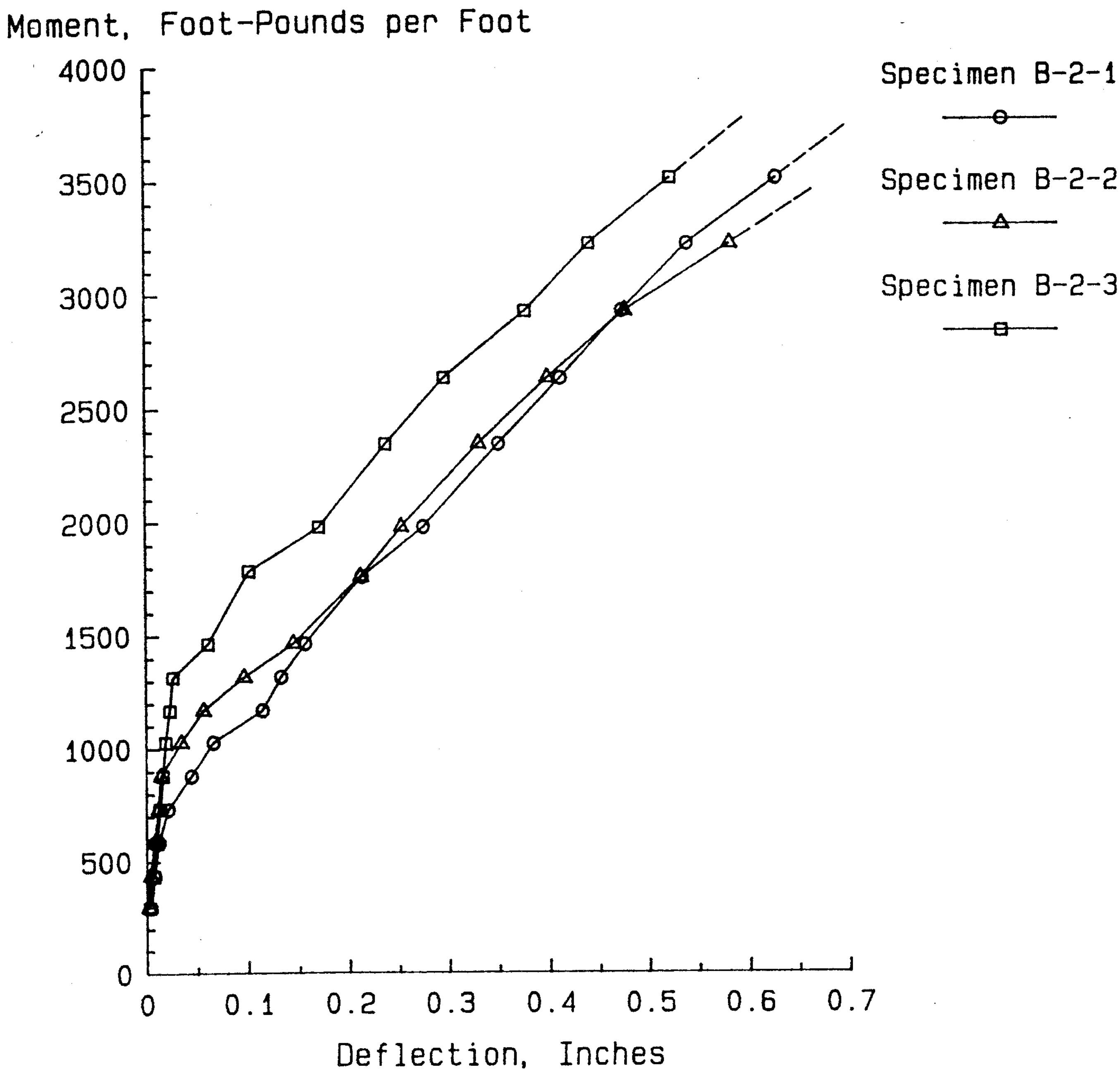
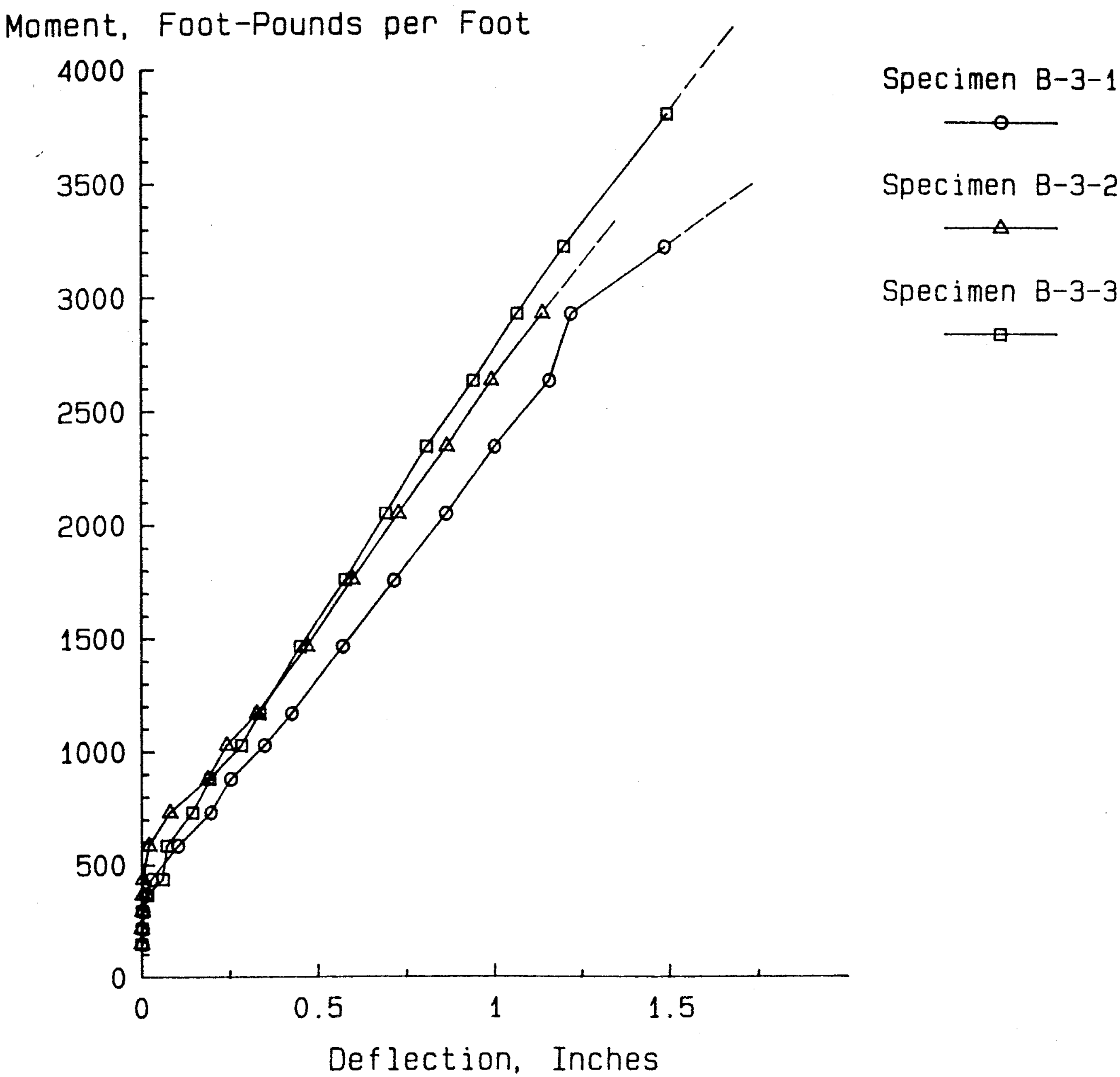
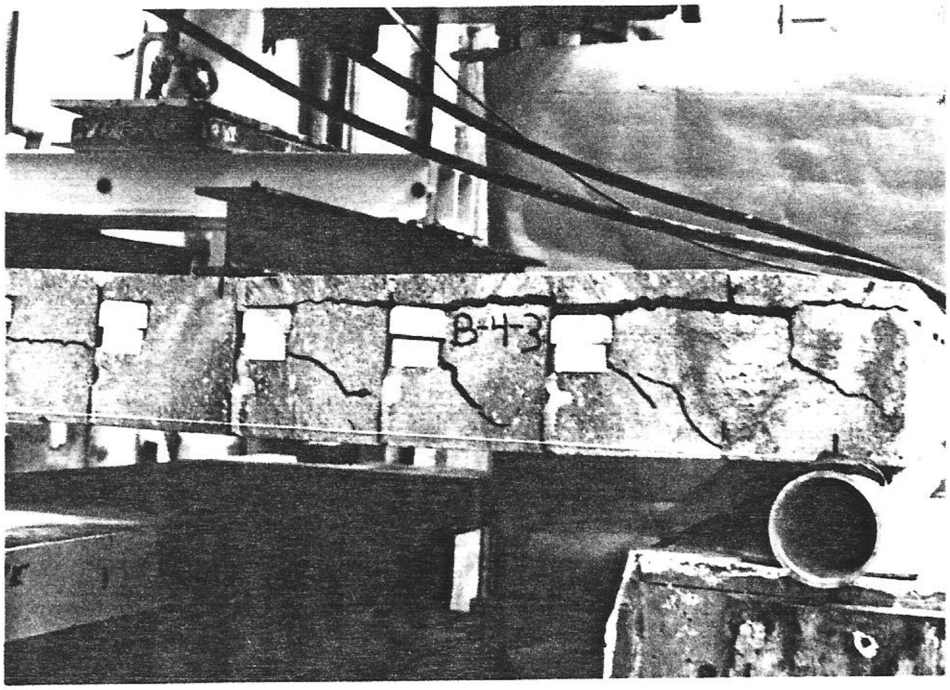
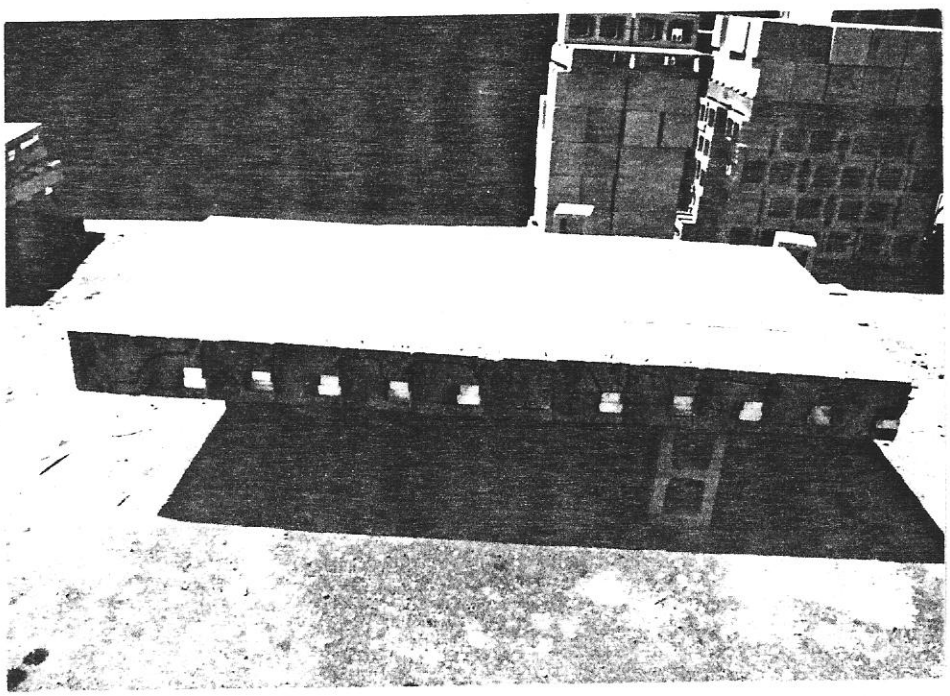


FIGURE NO. 4-4
FLEXURAL WALL TEST SERIES B-3
MOMENT VS DEFLECTION



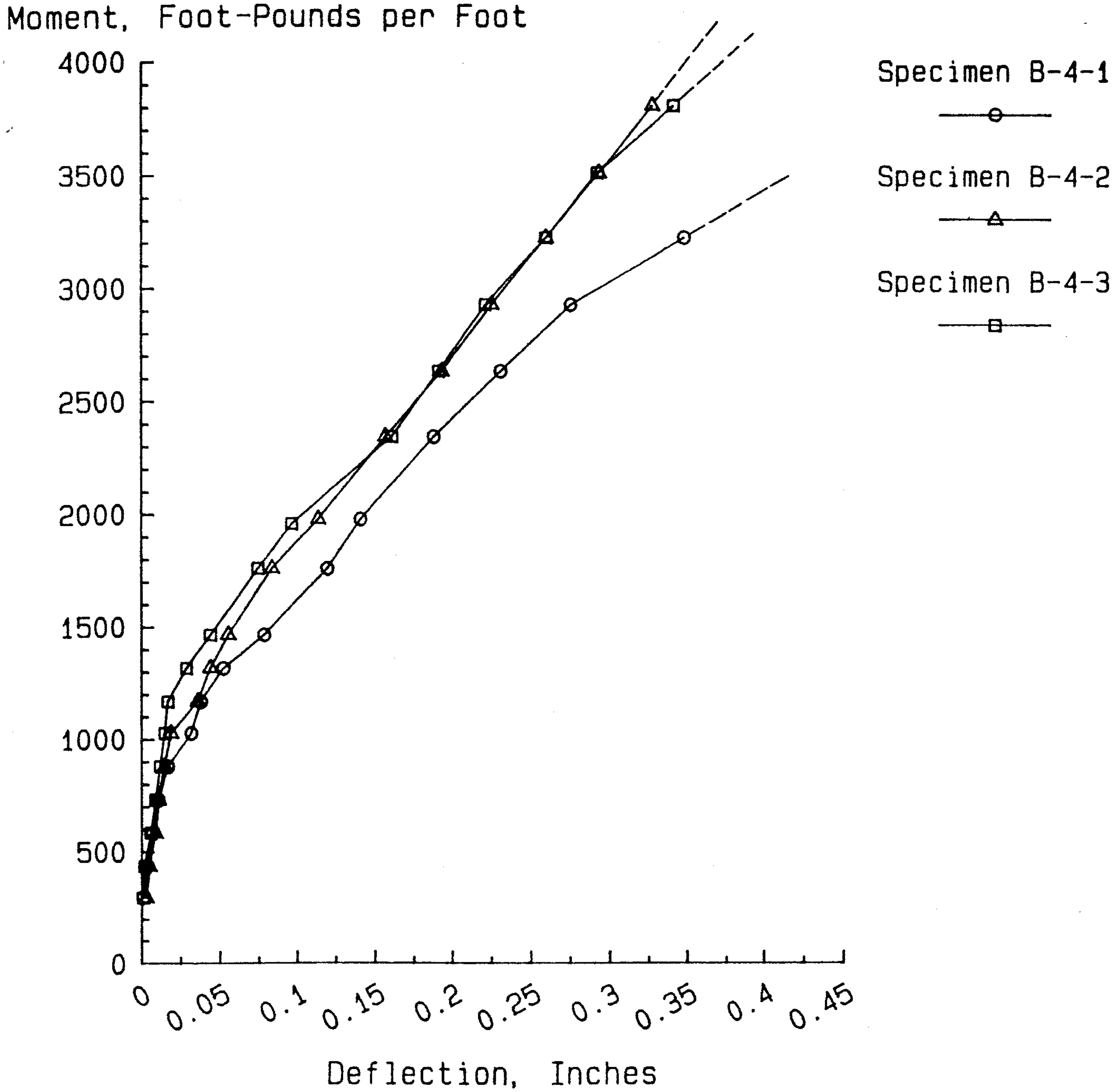


Test Specimen B-4-3 after
third point loading tests



Test Specimen B-3-1 after
flexural testing

FIGURE NO. 4-5
FLEXURAL WALL TEST SERIES B-4
MOMENT VS DEFLECTION



4.4 RESULTS OF COMPRESSION TESTS ON WALLS

The effect of full grouting (Test Series C-2) versus partially grouting (Test Series C-1) was investigated for reinforced walls. Non-reinforced walls (Test Series C-3) were not grouted. All walls were tested with insulating inserts installed. The typical failure mode of compression walls was spalling of the face shells on the insulation side of the wall. Test results are summarized in Table 4-8 and section properties of test specimens are calculated in Figure 4-10.

Table 4-8
Results of Compression Tests on Walls

Test	Maximum Compressive Load, P Pounds	Eccentricity, from Centroid toward Insulation Side Inches	Maximum Moment, M Inch-Pound
C-1-1	237,600	1.55	377,800
C-1-2	176,800	1.55	281,100
C-1-3	231,700	1.55	368,400
Average	215,370		
C-2-1	358,200	1.86	666,300
C-2-2	392,400	1.86	729,900
C-2-3	343,700	1.86	639,300
Average			
C-3-1	151,900	1.27	192,915
C-3-2	129,600	1.27	164,590
C-3-3	237,900	1.27	302,135
Average	173,130		

Properties of Compression Wall Specimens

Test Series	Minimum Net Cross Sectional Area, A Sq. In.	Moment of Inertia, I $In.^4$	Section Modulus, S Grouted Side $In.^3$	Section Modulus, S Insulated Side $In.^3$
C-1	174	1377.5	394	334
C-2	240	1460.2	452	332
C-3	132	1310.0	344	344

FIGURE NO. 4-7
COMPRESSION WALL TEST SERIES C-1
LOAD VS DEFORMATION

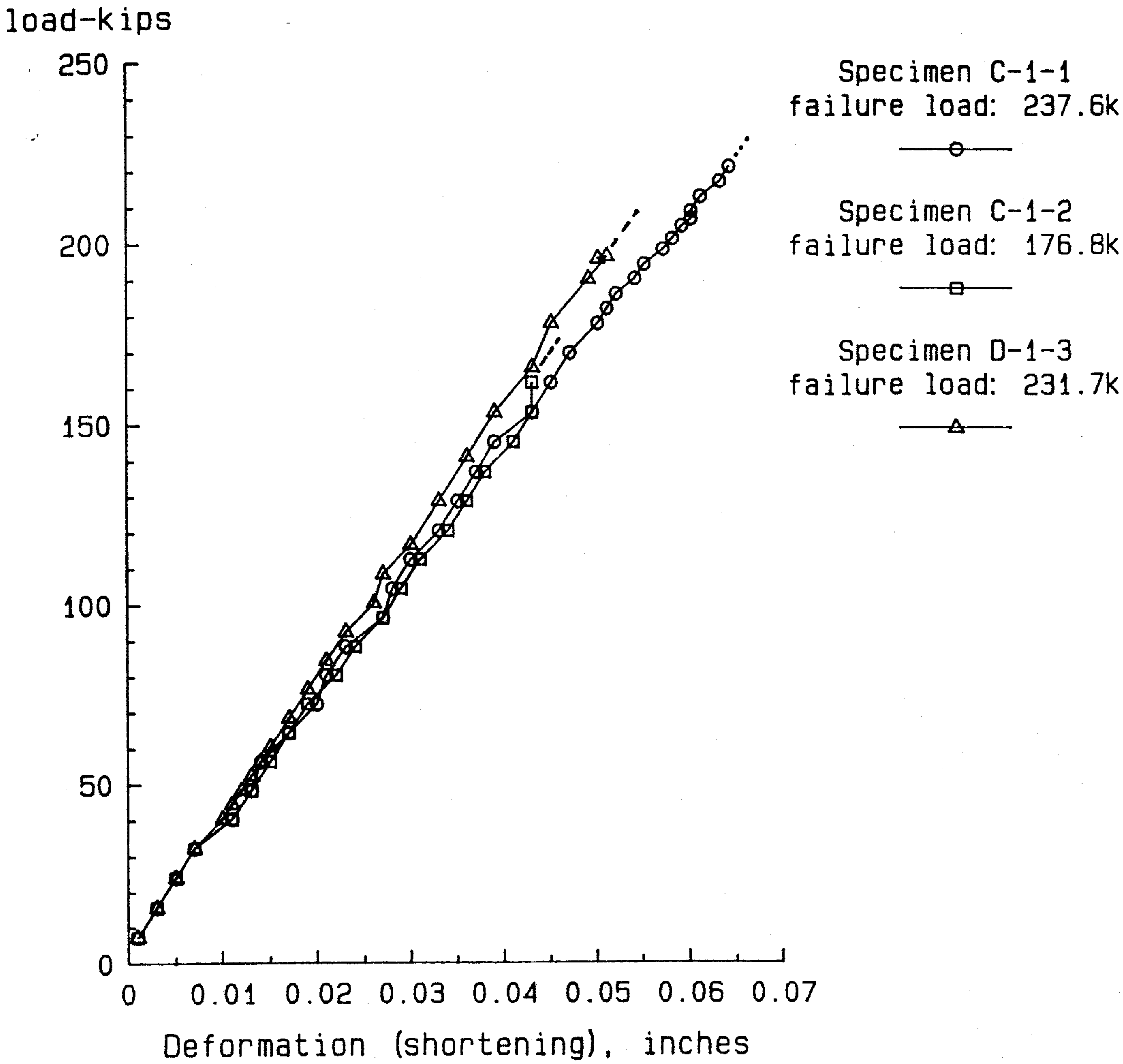


FIGURE NO. 4-8
COMPRESSION WALL TEST SERIES C-2
LOAD VS DEFORMATION

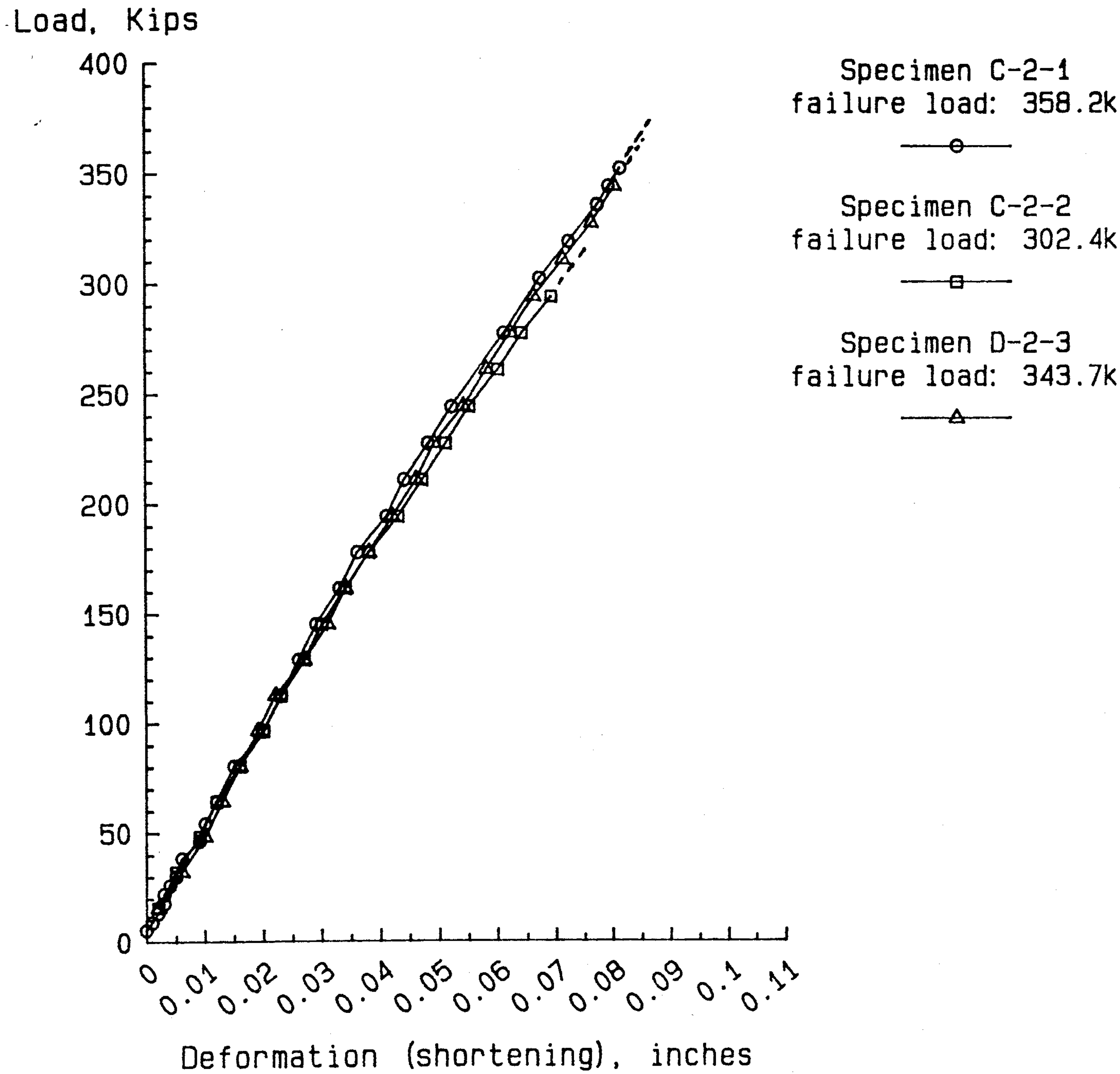
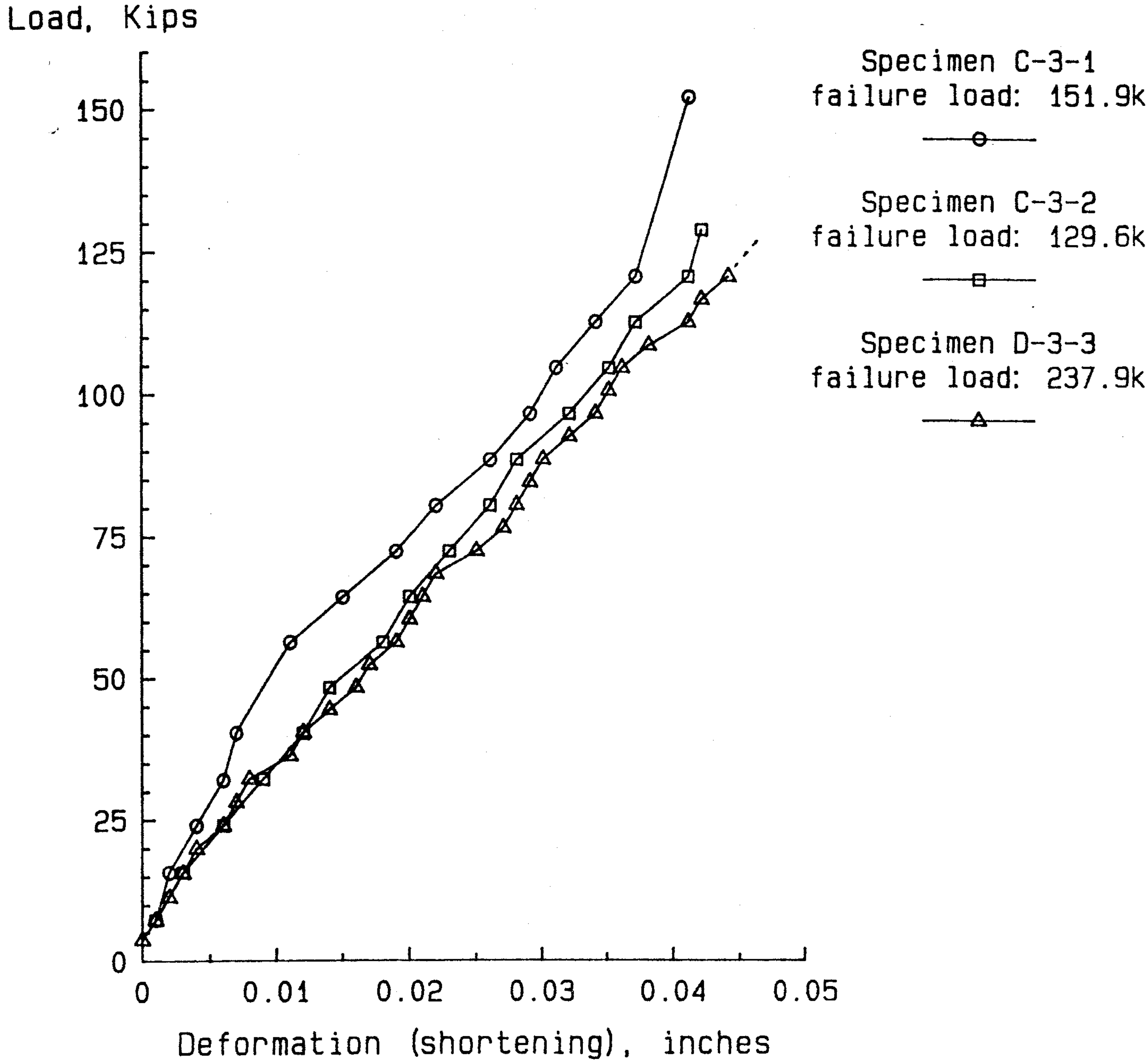


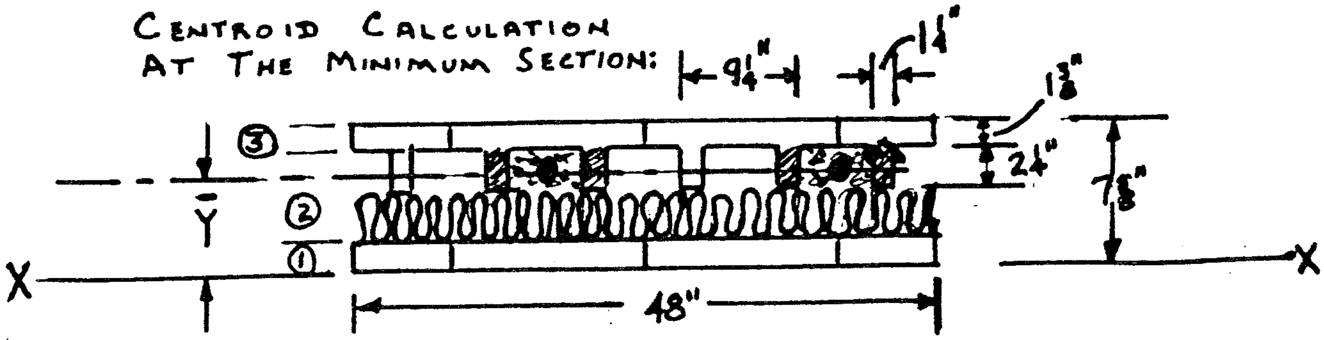
FIGURE NO. 4-9
COMPRESSION WALL TEST SERIES C-3
LOAD VS DEFORMATION



Calculation of Section Properties of Compression Test Wall Specimens for Korfil Hi-R Walls	Figure 4-10

TEST SERIES C-1

CENTROID CALCULATION
AT THE MINIMUM SECTION:



MOMENTS ABOUT X-X	A	Y	AY
① 1.375 x 48 =	66	0.6875	45.375
② 2.25 x 18.5 =	41.6	5.125	213.328
③ 1.375 x 48 =	66	6.938	457.875
	$\Sigma A = 173.6$		$\Sigma AY = 716.58$

$$\bar{y} = \Sigma AY / \Sigma A = 716.58/173.6$$

$$\bar{y} = 4.13 \text{ INCHES}$$

MOMENT OF INERTIA ABOUT CENTROID:

$$I = \frac{bh^3}{12} + Ad^2$$

$$I_{①} = 48 \times (1.375)^3/12 + 66 \times (3.4)^2 = 773.4$$

$$I_{②} = 18.5 \times (2.25)^3/12 + 41.6 \times (1.03)^2 = 61.7$$

$$I_{③} = 48 \times (1.375)^3/12 + 66 \times (2.84)^2 = 542.4$$

$$I = 1377.5 \text{ IN.}^4$$

SECTION MODULUS, S:

$$S = I/c$$

FOR GROUTED SIDE $c = 7.63 - 4.1$ $S = 1378/3.5$

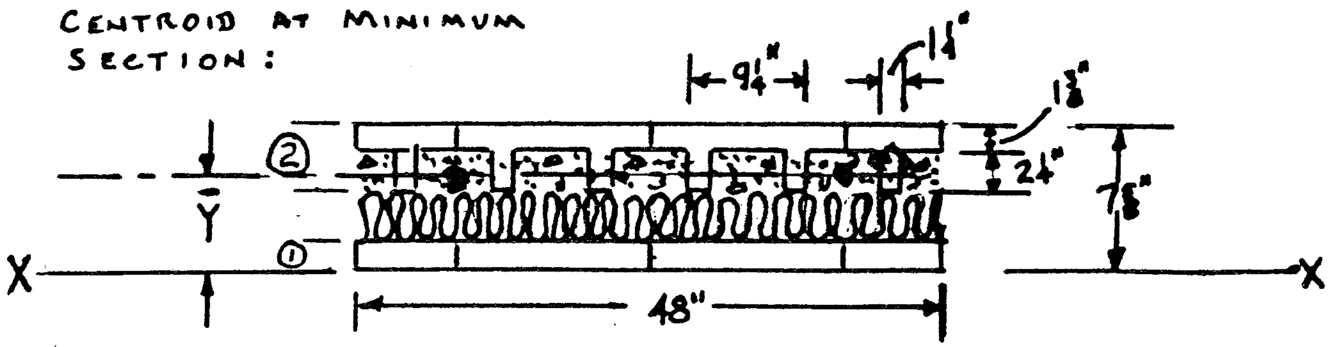
$S = 394 \text{ IN}^3$

FOR INSULATION SIDE $c = 4.1$ $S = 1378/4.1$

$S = 334 \text{ IN}^3$

TEST SERIES C-2

CENTROID AT MINIMUM SECTION:



MOMENTS ABOUT X-X	A	Y	AY
① 1.375 x 48 =	66	0.6875	45.375
② 3.625 x 48 =	174	5.8125	1011.375
	$\Sigma A = 240$		$\Sigma AY = 1056.75$

$$\bar{Y} = \Sigma AY / \Sigma A = 1056.75/240$$

$$\bar{Y} = 4.4 \text{ INCHES}$$

MOMENT OF INERTIA ABOUT CENTROID

$$I = bh^3/12 + Ad^2$$

$$I_① = 48 \times (1.375)^3/12 + 66 \times (3.72)^2 = 923.7$$

$$I_② = 48 \times (3.625)^3/12 + 174 \times (1.41)^2 = 536.5$$

$$I = 1460.2 \text{ IN}^4$$

SECTION MODULUS

$$S = I/c$$

FOR GROUTED SIDE, $c = 7.63 - 4.4$
$c = 3.23$ IN.

$$S = 1460.2/3.23$$

$$S = 452 \text{ IN}^3$$

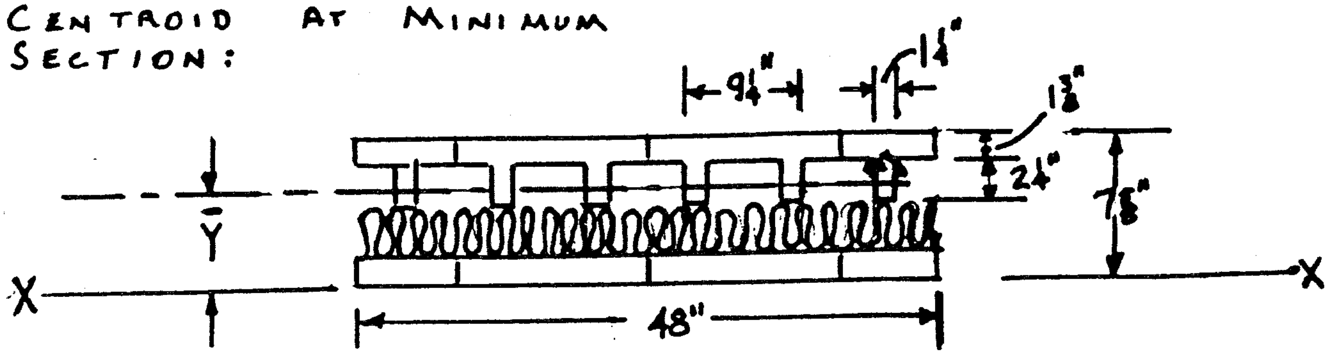
FOR INSULATION SIDE, $c = 4.4$ IN

$$S = 1460.2/4.4$$

$$S = 332 \text{ IN}^3$$

TEST SERIES C-3

CENTROID AT MINIMUM SECTION:



$\bar{Y} = t/2 = 7.625/2$

$\bar{Y} = 3.81$ INCHES

MOMENT OF INERTIA ABOUT CENTROID:

$$I = b\,(h_{solid})^3/12 - b\,(h_{hollow})^3/12$$

$$I = 48 \times 7.625^3/12 - 48 \times 4.875^3/12$$

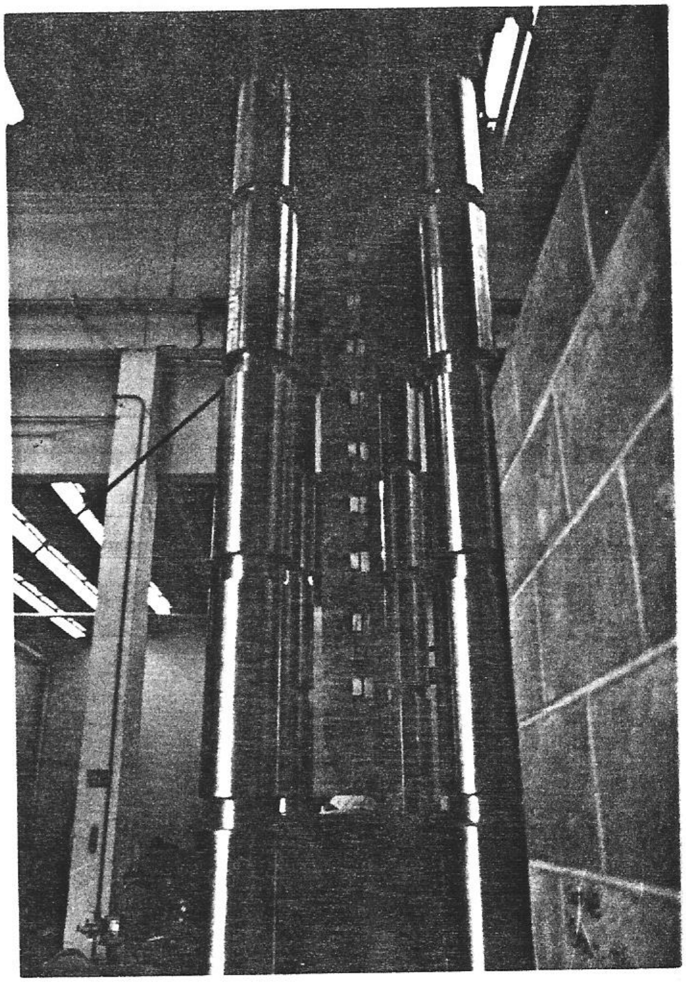
$I = 1310 \text{ IN}^4$

SECTION MODULUS

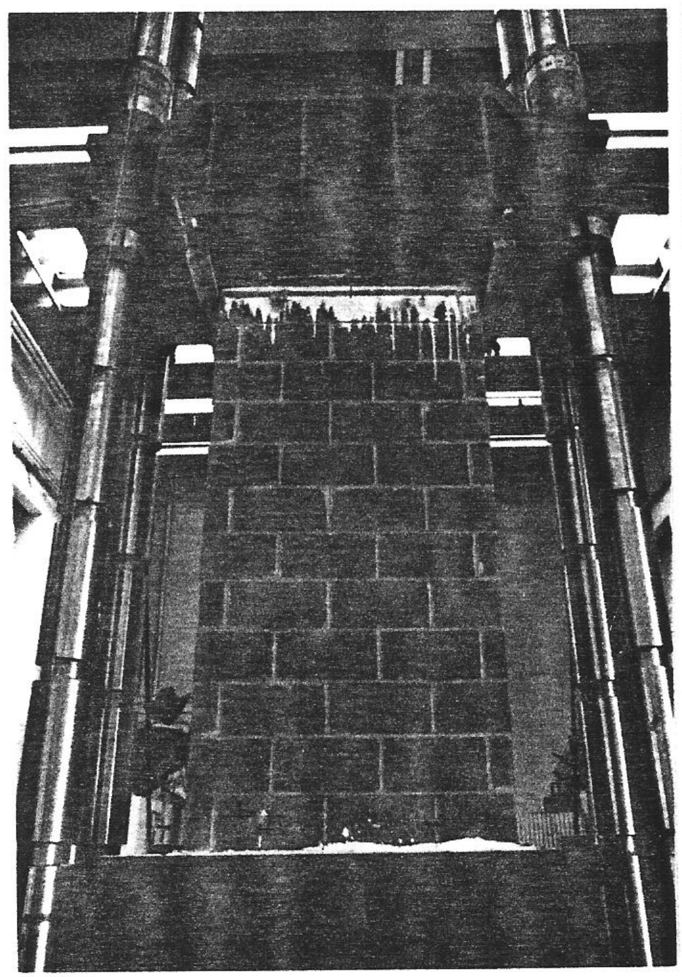
$$S = I/c$$

$$S = 1310/3.81 = 344 \text{ IN}^3$$

Photographs of Compression Wall Tests | Figure 4-11



Test Specimen C-1-3 after compression test



Test Specimen C-2-1 install in compression test frame

4.5 RESULTS OF DIAGONAL TENSION (SHEAR) TESTS

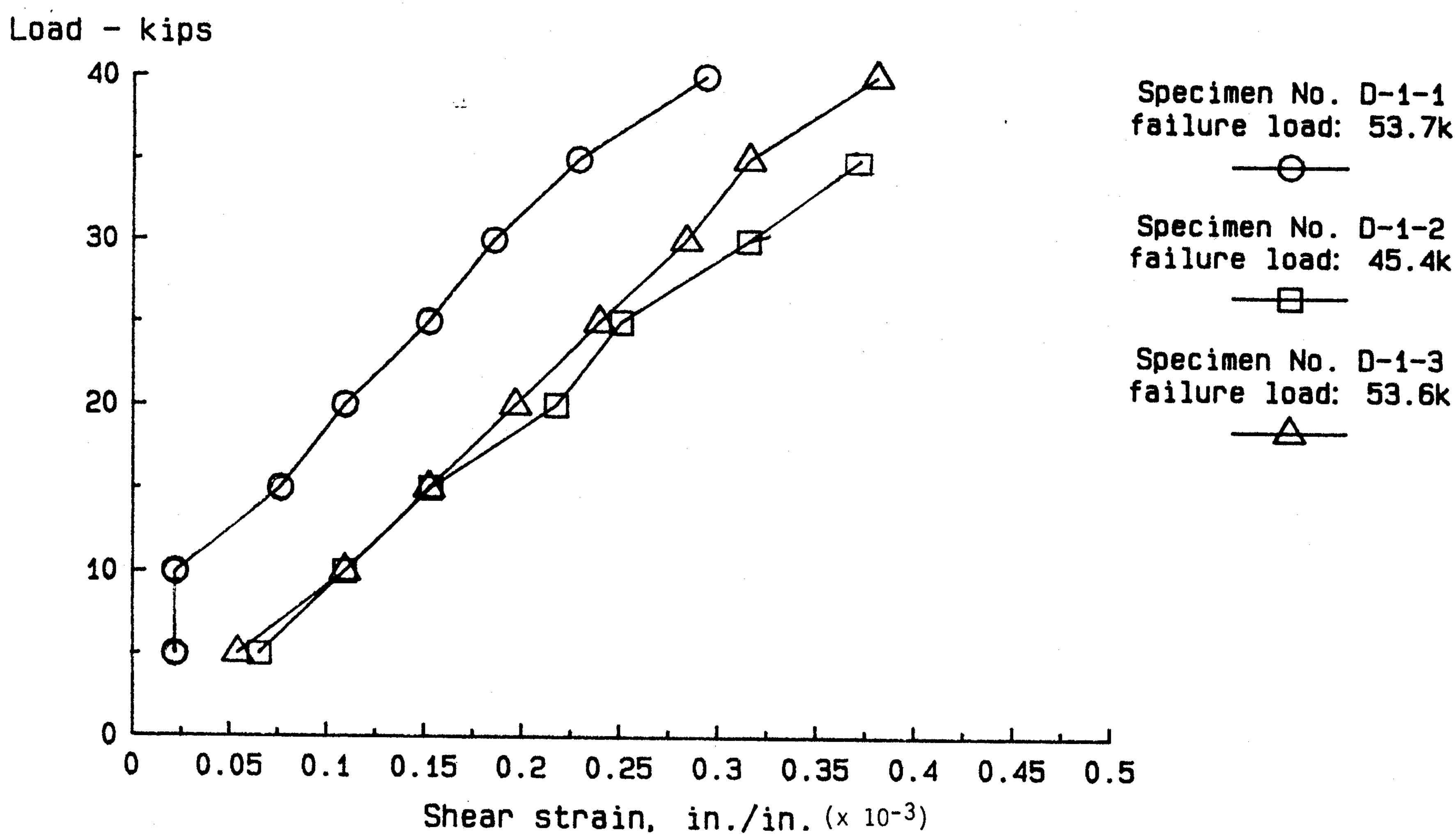
Diagonal tension (shear) tests were conducted on both reinforced (Test Series D-1 and D-2) and non-reinforced (Test Series D-3) Korfil Hi-R Specimens. The effect of partial grouting (Test Series D-1) and full grouting (Test series D-2) on diagonal tension (shear) was investigated for reinforced specimens. No grout was used in the unreinforced test series . The results are summarized in Table 4-9. Shear Stress versus strain is plotted in Figure No.4-12, 4-13, and 4-14 for Test Series D-1, D-2, and D-3, respectively. The typical mode of failure was vertical splitting of the specimen between the loaded corners. Faceshell spalling as well as vertical cracking occurred in one test (Test Specimen D-2-3) on the insulation side at maximum load.

The reinforced, fully grouted walls (test series D-2) were instrumented with gauges to measure the total shortening of the specimen up to and beyond the peak load. This load versus displacement (shortening) data is illustrated in Figure No. 4-15.

Table 4-9
Results of Diagonal Tension (Shear Tests)

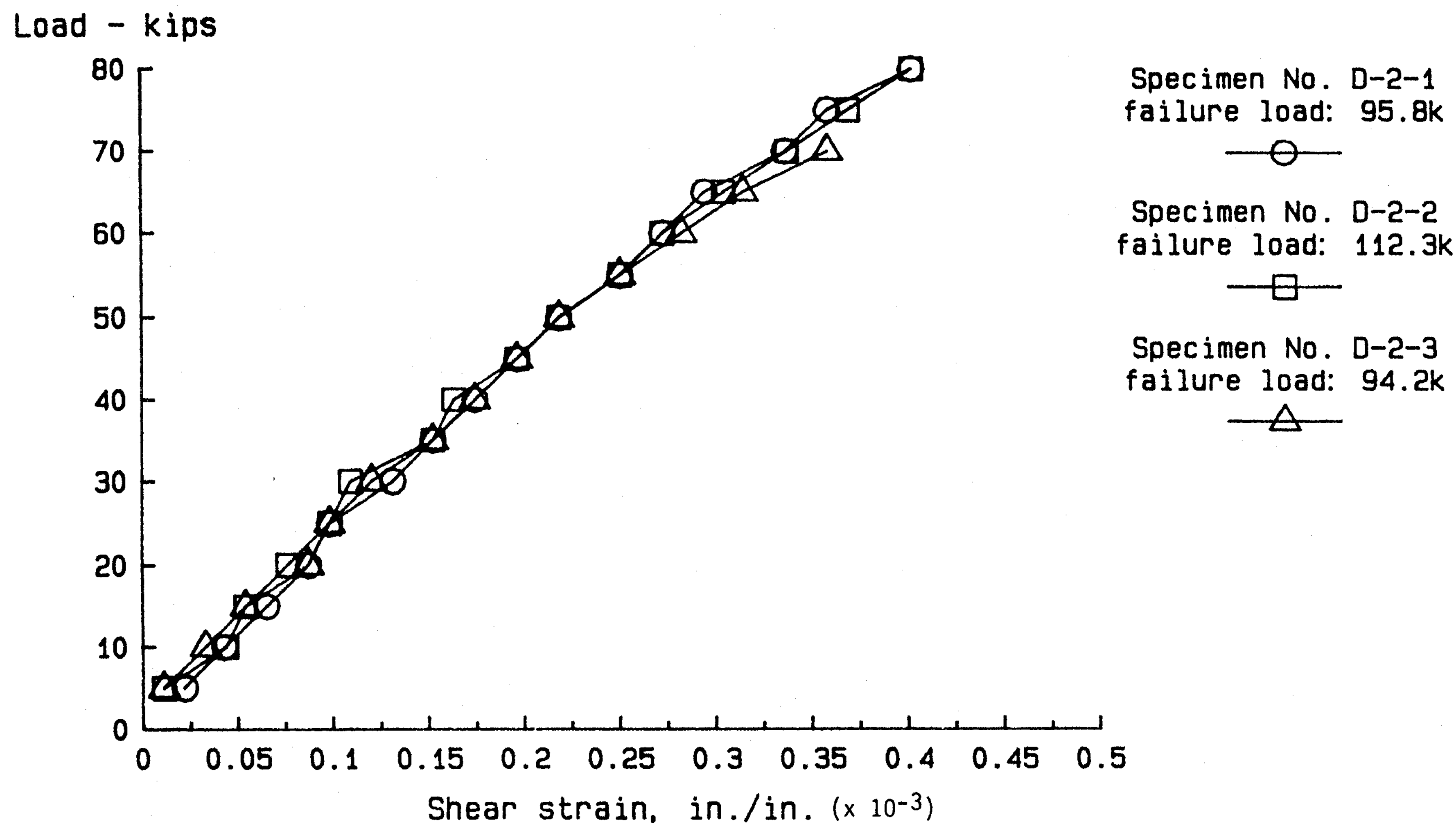
Test	Maximum Load, P, Parallel to Diagonal lbs.	Horizontal Component of P, lbs	Net Area Of Wall Sq.In.	Shear Stress, v, v^m PSI
D.1-1	53,660	37,943	208	182
D.1-2	45,380	32,089	208	154
D.1-3	53,630	37,922	208	182
Average	50,890	35,984		173
D.2-1	95,770	67,720	246	275
D.2-2	112,270	79,387	246	323
D.2-3	94,180	66,595	246	271
Average	100,740	71,234		290
D.3-1	38,110	26,948	157.4	171
D.3-2	34,360	24,296	157.4	154
D.3-3	61,180	43,260	157.4	275
Average	44,500	31,501		200

Figure No. 4-12
DIAGONAL TENSION (SHEAR) - SERIES D-1
LOAD VS STRAIN



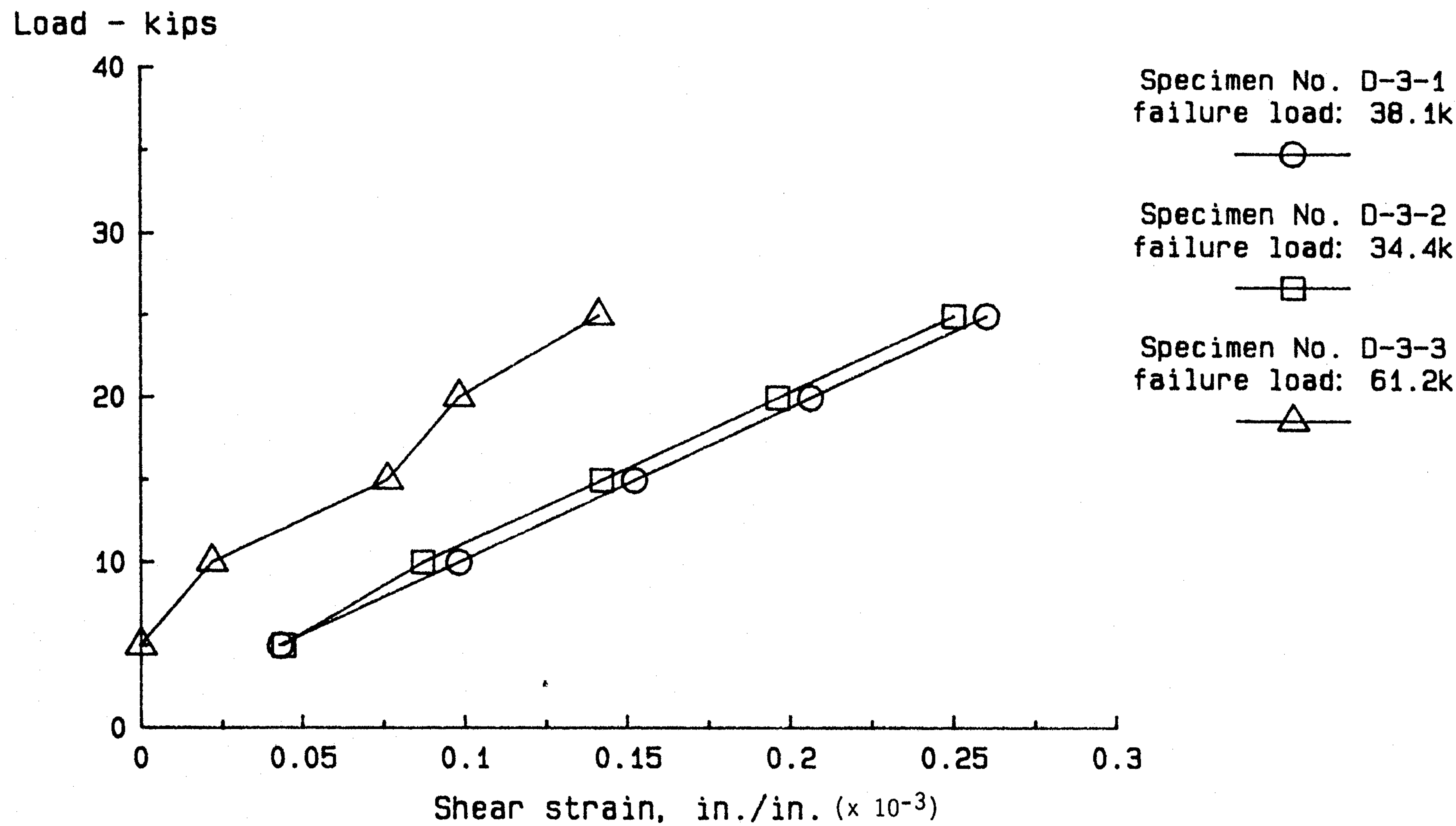
Gauges removed at load indicated on graph

Figure No. 4-13
DIAGONAL TENSION (SHEAR) - SERIES D-2
LOAD VS STRAIN



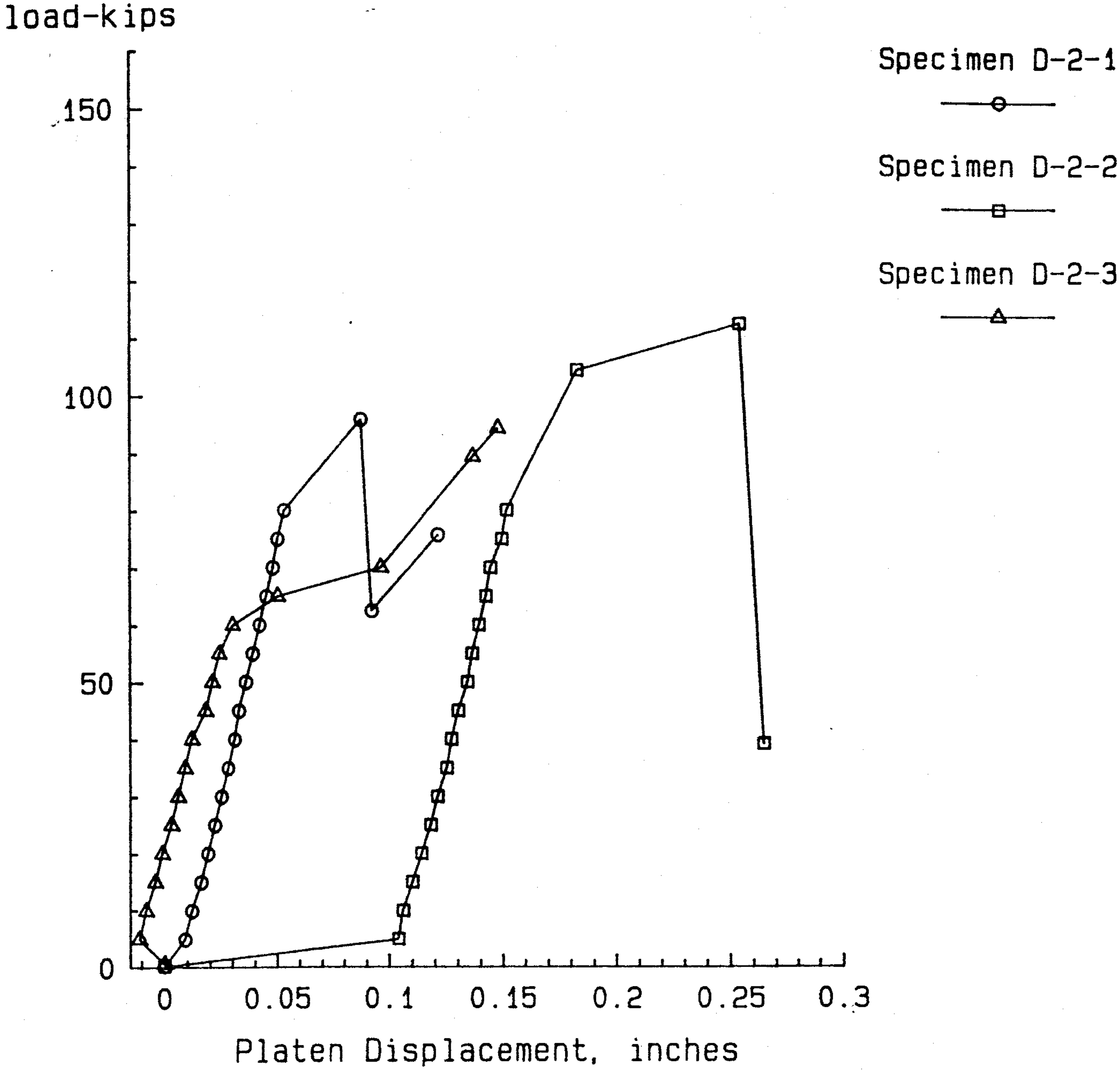
Gauges removed at load indicated on graph

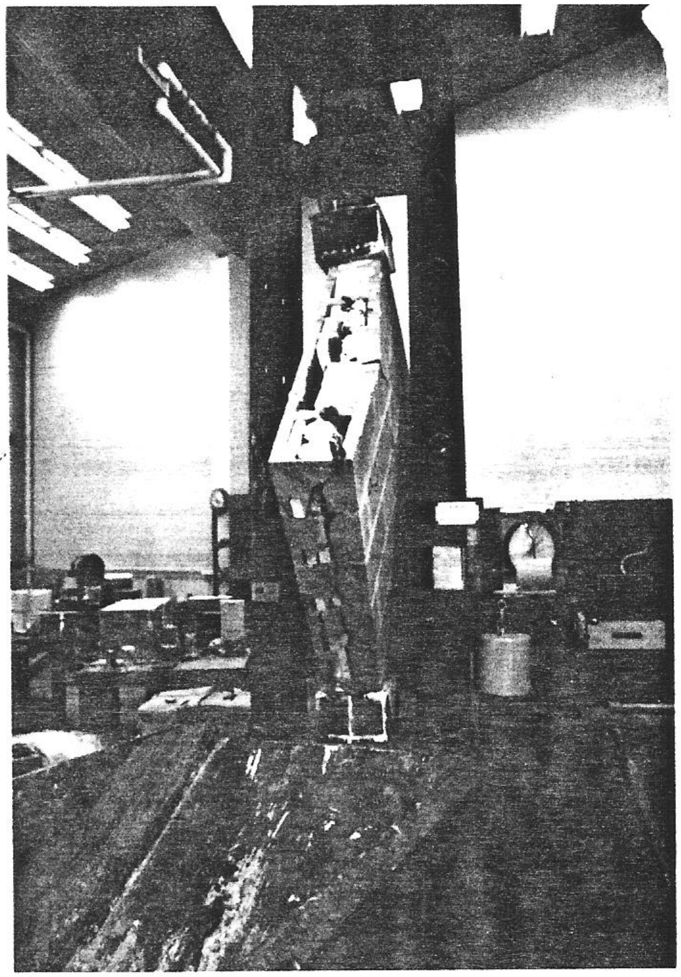
Figure No. 4-14
DIAGONAL TENSION (SHEAR) - SERIES D-3
LOAD VS STRAIN



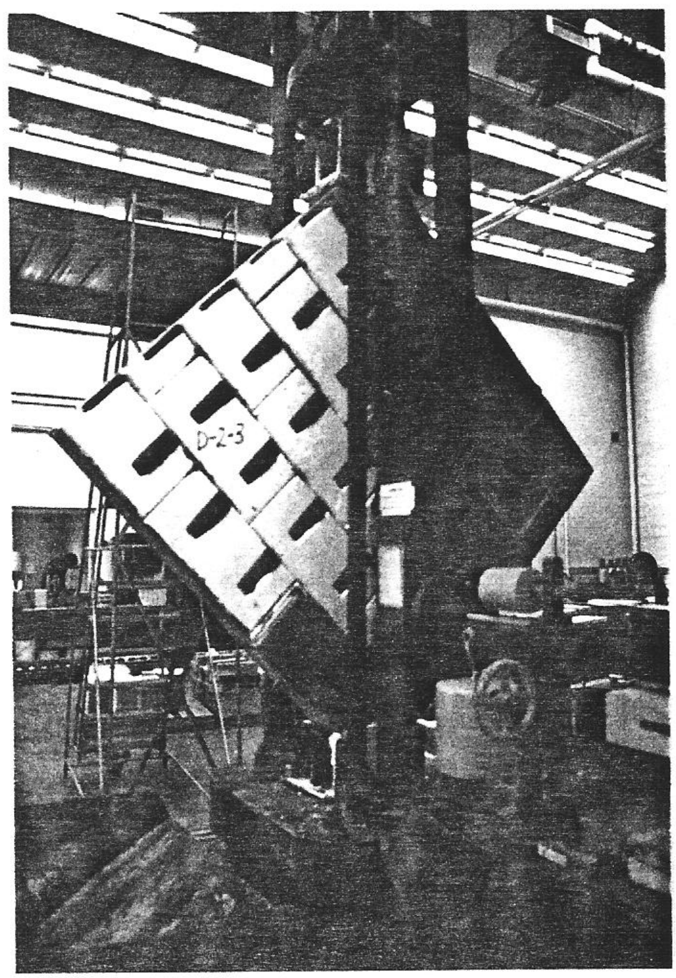
Gauges removed at load indicated on graph

FIGURE NO. 4-15
DIAGONAL TENSION (SHEAR) TEST SERIES D-2
LOAD VS PLATEN DISPLACEMENT





Test Specimen D-1-3 after shear test



Test Specimen D-2-3 after shear test

4.6 RESULTS OF MASONRY PRISM TESTS

Prism tests were conducted on both grouted and ungrouted prisms. Grouted prisms in which insulating inserts were removed (Test Series E-1) as well as grouted prisms with insulating inserts installed (Test Series E-4) were tested. The results of these strength tests are reported in Table 4-10 based on net area which include the cross sectional area of units, mortar or grout, but exclude the cross sectional area of insulation.

Non-grouted prisms included both faceshell mortar bedded prisms (Test Series E-2) as well as full mortar bedded prism (Test Series E-3). Only a portion of the cross webs could be mortar bedded due to the cut down web which accommodates the insulating insert, therefore difference in area between face shell (43.0 sq. in.) and full mortar bedded prisms (48.6 sq. in.) is 5.6 square inches (12%). The difference in average maximum compressive load of full mortar bedded prisms and face shell mortar bedded prism is 6666 pounds, 0.6%.

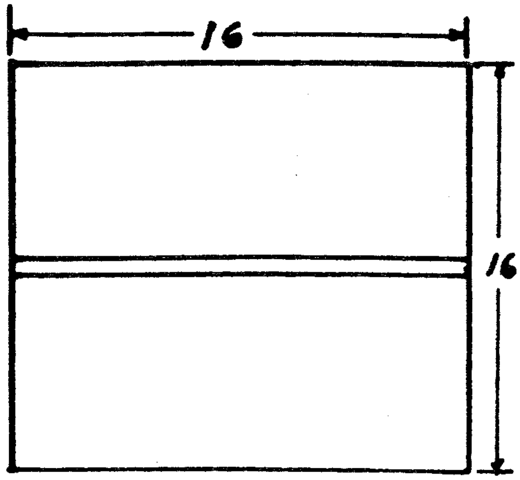
Table 4-10
Results of Compression Tests on Grouted Prisms

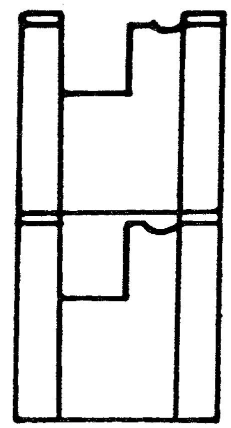
Test	Maximum Compressive Load, lbs.	Compressive Strength of Prism, net area, PSI
E.1-1	276,000	2220
E.1-2	316,000	2550
E.1-3	273,000	2200
	Average	2320
E.4-1	171,000	2085
E.4-2	177,000	2160
E.4-3	136,000	1660
	Average	1970

Table 4-11
Results of Compression Tests on Non Grouted Prisms

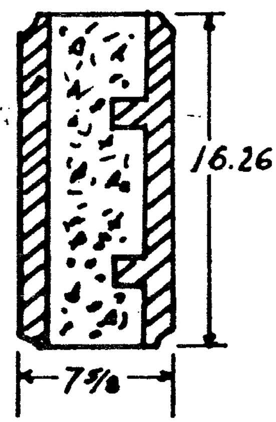
Test Designation	Maximum Compressive Load Lbs		Compressive Based on: Average Net Area of Unit PSI	Strength Based on: Net Mortar Bedded Area PSI
E.2-1	109,000		2110	2535
E.2-2	117,000		2270	2720
E.2-3	116,000		2250	2700
		Average	2210	2650
E.3-1	120,000		2325	2470
E.3-2	112,000		2170	2305
E.3-3	112,000		2170	2305
		Average	2220	2360

Calculation of Cross Sectional Area of Masonry Prisms | Figure 4-17





FRONT VIEW

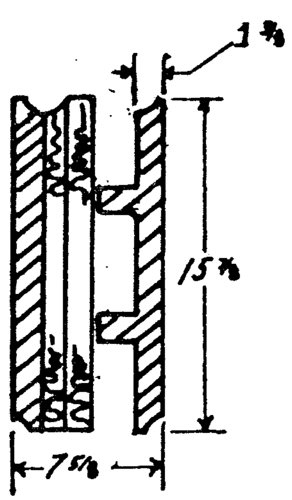


CALCULATIONS OF AREAS:

PRISM E-1 - SOLID GROUT NO INSERTS

NET CROSS SECTIONAL AREA = GROSS AREA

= L×W = 16.26" x 7.63 = 124.1 SQ.IN



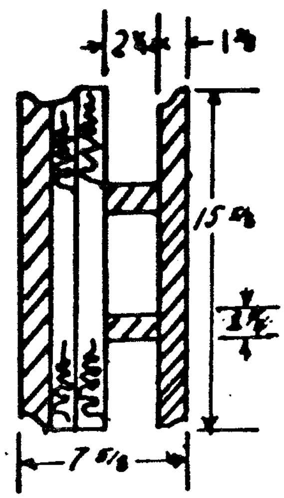
PRISM NO F-2 - NO GROUT WITH INSERTS, FACE SHELL MORTAR.

NET CROSS SECT. AREA = GROSS AREA × % SOLID

= (15.63 × 7.63) × 0.433 = 51.6 SQ. IN.

NET MORTAR SECT AREA = AREA OF FACERS

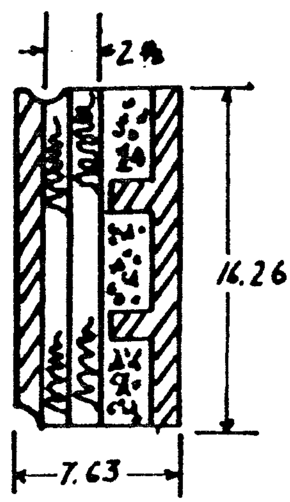
= 2(1.375 × 15.3) = 43 SQ. IN.



PRISM NO. E-3 - NO GROUT, WITH INSERTS; FULL MORTAR

NET CROSS SECTIONAL AREA = GROSS AREA x %
= (15.63 x 7.63) x 0.433 = 51.6 SQ. IN

NET MORTAR BEDDED AREA = AREA OF FACE SHELLS + WEBS
= [2 (1.375 + 2 (1.25) x 2.25]
= 43 + 5.6 = 48.6 SQ. IN



PRISM E-4 GROUTED WITH INSULATED INSERT

NET CROSS SECTIONAL AREA = GROSS AREA
MINUS AREA OF INSULATING INSERTS
= (16.26 x 7.63) - (16 x 2.63)
= 124.1 - 42.1 = 82 SQ. IN

5.0 ANALYSIS OF TEST RESULTS

This chapter presents an analysis of the Korfil Hi-R block test results from the five basic test series, flexure in beams, compression on walls, out-of-plan loading walls, diagonal tension (shear), and prisms. The previous four chapter of this report and the appendices explain the test set-ups and provide a complete digest of the results. The goal of the analysis of the test data is to provide insight into the behavior of the Hi-R block under various loading conditions and provide a comparison between this behavior and that of conventional concrete masonry units under similar loading conditions.

All building code references and section citations refer to the 1985 Uniform Building Code (UBC).

5.1 Flexural Tests on Beams

The tests conducted in this portion of the program were developed to investigate the flexural behavior of beam specimens loaded about their strong axis. These tests were conducted to answer the following questions:

(1) Can the 1985 UBC working stress provisions for in-plane flexure be used to design Hi-R blocks for flexural loads?

(2) What is the safety ratio provided by beams constructed of Hi-R block compared to that possessed by beams constructed of conventional block?

(3) How does the modulus of rupture measured in these tests compare with that obtained from tests of conventional concrete masonry?

Equation 6-6 of UBC Section 2406(c)3 permits an allowable compressive stress due to flexure of F_m=650psi for Test Series A.1 and A.2 with F'_m=2320 psi. A comparison between the measured flexural loads from the tests and the allowable flexural load computed according to the principles of working stress designed to produce the following safety ratios.

Series	Safety Ratio
A.1 (#4 bar, d=6 in.)	1.8
A.2 (#6 bar, d=6 in.)	1.6
A.4 (#4 bar, insulation removed d=2 in.)	4.3

The lower safety ratio noted in Test Series A.2 is principally the result of one test in which the reported results were 50% less than that reported in the specimens of that series. Had additional tests been conducted for the Series A.2 it is likely that the safety ratio would be similar to that reported for Series A.1.

Similar test on conventional block reported in the literature (Converse, 1946 and Mayrose, 1954) indicate that this block can develop safety ratios in the range of 2.4 to 3.5 for beams with a d similar to that in Series A.1 and A.3. The safety ratio for beams with a d similar to that in Series A.4 ranges from 3.6 to 4.4.

The lower safety ratio for Hi-R block is an artifact of the testing procedure in which failure was determined when the face shell on the insulated side spalled. However, the remaining grouted section still possessed significant strength. Had the beam specimens been loaded to the failure of the grouted section, the safety ratio would have been higher and more in line with the results in the literature.

Test Series A.3 was conducted to investigate the modulus of rupture of the Hi-R block beams. The average modulus of rupture of the test specimens is 218 psi with a coefficient of variation of 0.1. This compares favorably with the results reported by Livingston et.al. (1958) for similar conventional concrete masonry units. From the prism tests, the value of F'_m of the grouted, unreinforced blocks is 2320 psi. Based on equation 11-14 of UBC Section 2411(b)4, the modulus of rupture assumed for strength design is

$$
\begin{aligned}
ft &= 2.5\ (f'_m)^{0.5} \\
&= 2.5\ (2320)^{0.5} \\
&= 120 \text{ psi}
\end{aligned}
$$

The actual modulus of rupture is significantly higher than that assumed by the equation. The coefficient of 2.5 in Equation 11-14 would be 4.5 to match the results produced by the tests.

5.2 Analysis of Flexural Tests on Walls

The out-of-plane loading tests on the wall specimens were developed to investigate the out-of-plane flexural capacity of walls constructed of Hi-R block. The unsymmetrical cross section of the block suggested that the load be applied against both the insulated and uninsulated faces. These tests were conducted to answer the following questions:

(1) What is the difference in stiffness and strength when an out-of-plane load is applied to each face of the Hi-R block?

(2) What are the controlling modes of failure of the walls for different steel ratios and loading directions?

(3) What is the ratio of the walls out-of-plane flexural strength to the working design wind and seismic loads?

The difference between the wall out-of-plane stiffness and strength compared to the direction of the applied load can be seen in Figure 5.2.1. This figure shows the load versus deflection curves for all of the noted tests. It can be seen from Figure 5.2.1 that the flexural capacity of the wall is dependent on the direction of the applied load. Test series B.1 and B.3 (d=2.5 in.) describe tests in which the load was applied to the uninsulated face of the block. These test series have a lower flexural stiffness and strength than that associated with test series B.2 and B.4 (d=5.0 in.).

The ratio of the stiffnesses for series B.3 and B.1, computed from the curves in Figure 5.2.1, and for series B.4 and B.2 is equal to 2.2. This represents the ratio of the corresponding steel reinforcement areas. This implies that the flexural stiffness of the Hi-R blocks is proportional to and is governed by the area of steel.

The strength of the wall was not determined using the uniformly applied load from the air bag tests. Because the flexural capacity of the walls was greater than the load that could be applied by the loading frame, we know that the failure load is at least the loading capacity of the testing frame.

In order to fail the walls we used a third point loading set up. There were two distinct failure modes for the walls that depended on the direction of the applied load. When the load was applied to the insulated face, the failure of the test specimens developed in the concrete masonry webs connecting the block face shells. As the loading was increased the shear across the weaker section of the web was also increased until it was greater than the strength of the webs and failure was produced by a spalling of the face shell on the compression side. The strength of the wall when the load is applied to the insulated face is independent of the reinforcement ratio as shown in Figure 5.2.1. When the load is applied to the uninsulated side, the out-of-plane strength of the wall is governed by flexure.

A review of the shear stress developed across the web, obtained from first principle considerations, indicates that the shear stress is on the order of 1000 psi. This compares to the accepted shear strength of concrete masonry of 133 psi for block with a similar value of f'_m (Ref. to ICBO Evaluation Report No. 4115). The greater shear strength developed in the webs of the Hi-R block compared to

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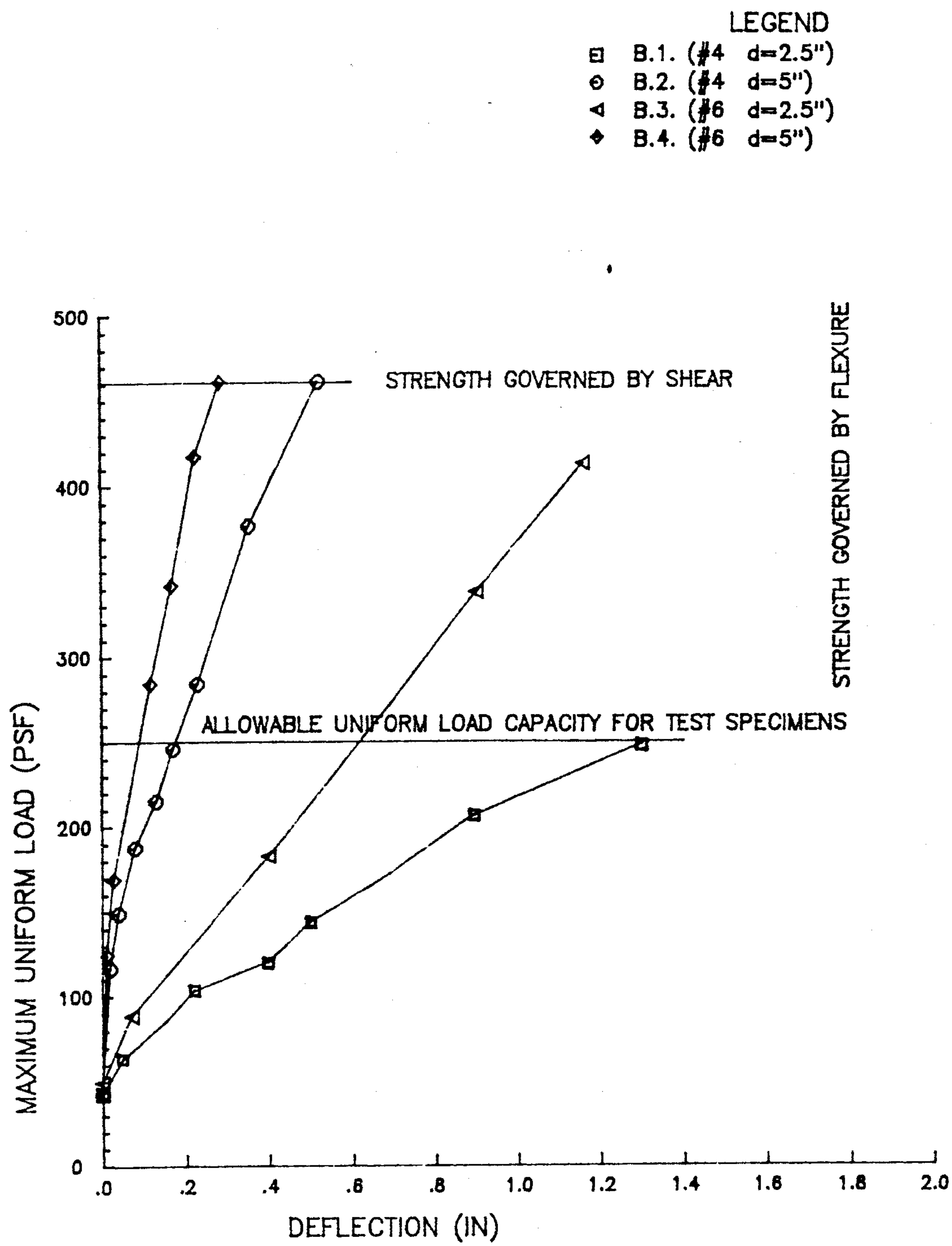

FIG 5.2.1 FLEXURAL WALL TESTS (B)

that accepted in approved strength design methods results from the fact that the shear strength of typical concrete block is based on an average value considering the entire net cross section (i.e., not just the web) to be available to resist the shear. Tests have shown that the concrete shear strength is on the order of 10 to 20 $(f'_c)^{.5}$ for confined areas in the absence of tension [Park and Paulay, 1975]. Inasmuch as most of the cross section in typical flexural elements is under tensile stress, the area capable of resisting the shear is usually a small portion of the cross section. Therefore, typically an average value of about $2(f'_c)^{.5}$ is used to account for the variation in stress across the entire net cross section. However, in the present case the failure plane of the web is near the neutral axis and the greater shear strength can be developed. Therefore, we will typically produce a shear failure mode when the wall is loaded this way, but the shear strength will be very high when the Hi-R block is loaded to failure on the insulated face.

Table 5.2.1 presents a comparison between the calculated flexural capacity of the wall and the moment capacity developed during the tests. Except for the test results given by Test Series B.f, the ratio of the tested moment to calculated moment is approximately 1.5. The lower ratio of tested moment to calculated moment for B.4 results from the fact that flexure was not the failure mode inasmuch as the capacity of the block wall was governed by shearing of the cross webs. This result isn't of great significance, however, because cases B.1 and B.3 would govern the design of the wall with their smaller effective depths, d = 2.75 inches. The ratio of 1.5 obtained for the Hi-R block compares favorably with the results obtained in the literature and summarized in Table 5.2.2.

It can be seen from a review of Figures 5.2.1 through 5.2.3 that the strength of the wall is significantly greater than the seismic loads prescribed in the UBC for out-of-plane loading of walls. Using the most conservative loading assumptions, it can be seen that the maximum wind load prescribed in the UBC is also less than the load corresponding to the out-of-plane strength of the walls in the test program. For purposes of comparison, the wind and seismic loads on a wall corresponding to an h to t ratio of 25 and 36 were also calculated and compared to the strength of a wall constructed of Hi-R block. Table 5.2.3 presents the allowable uniform load based on the tests for an Hi-R wall reinforced with #4 at 24 inches on center a function of the h/t ratio. Loads that exceed the tabulated values in Table 5.2.3 could be resisted with increased vertical steel. Inasmuch as the limit load for the flexural load is less than that for the shear limit load, the allowable uniform load is conservatively considered proportional to the square of the span.

Table 5-2-1
Calculated Moment Capacity and Ultimate Moment From Tests (Hi-R Masonry Walls)

Test Number	d(in.)	a(in.)	Calculated Moment Capacity (In-K)	Tested Moment Capacity (In-K)	Ratio M_{Test}/M_{Calc}
B.1-1	2.87	.276	65.57	98.21	1.50
B.1-2	2.50	.276	56.69	84.38	1.49
B.1-3	2.66	.276	60.53	112.39	1.86
B.2-1	5.00	.276	116.69	178.8	1.53
B.2-2	4.74	.276	110.45	165.0	1.49
B.2-3	5.08	.276	118.61	184.3	1.55
B.3-1	2.69	.606	126.03	182.59	1.45
B.3-2	2.65	.606	123.92	154.50	1.25
B.3-3	2.63	.606	122.87	182.59	1.49
B.4-1	4.70	.606	232.16	168.41	.73
B.4-2	5.13	.606	254.86	182.59	.72
B.4-3	4.94	.606	244.83	182.59	.74

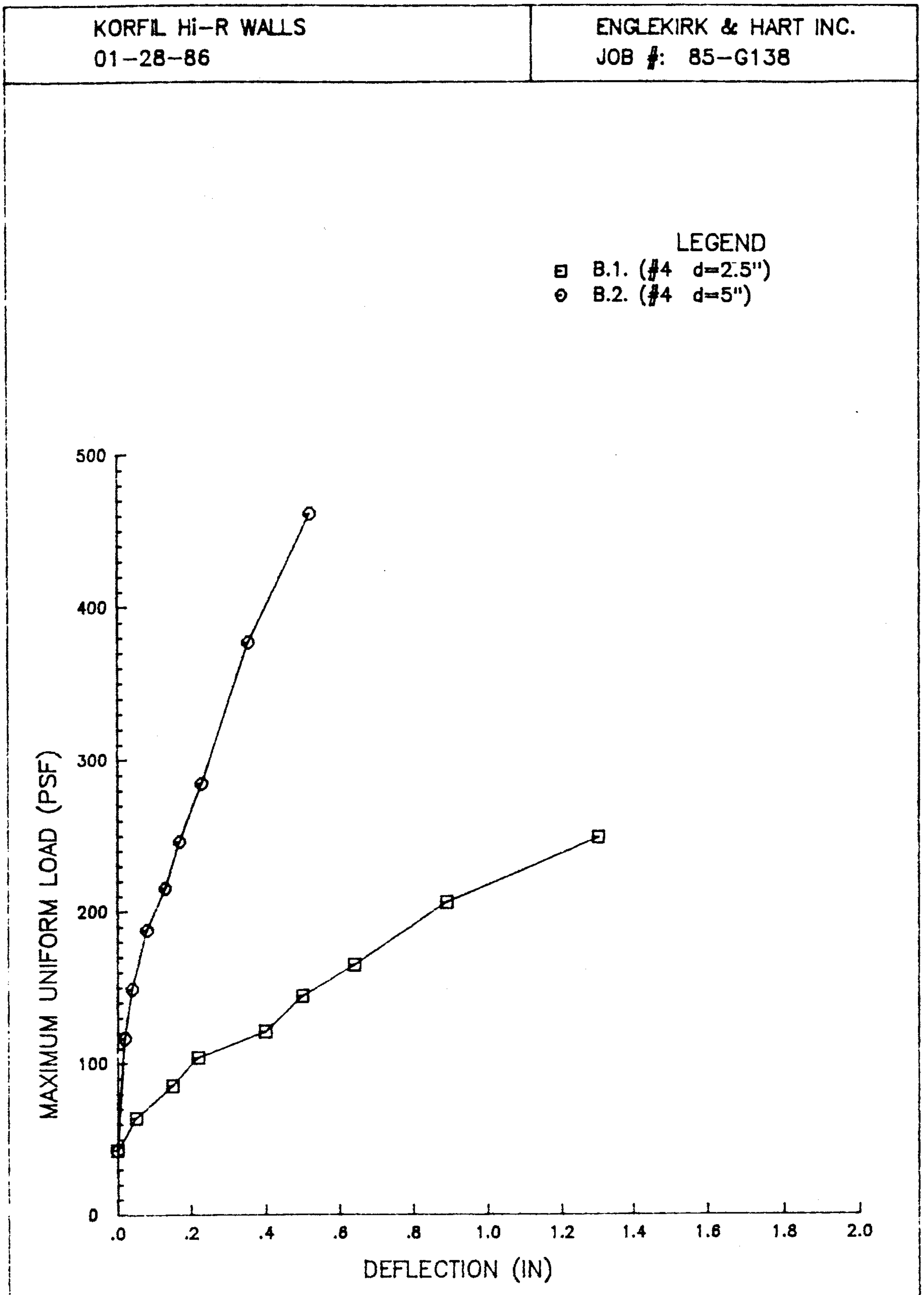

FIG 5.2.2 TESTS B.1 AND B.2

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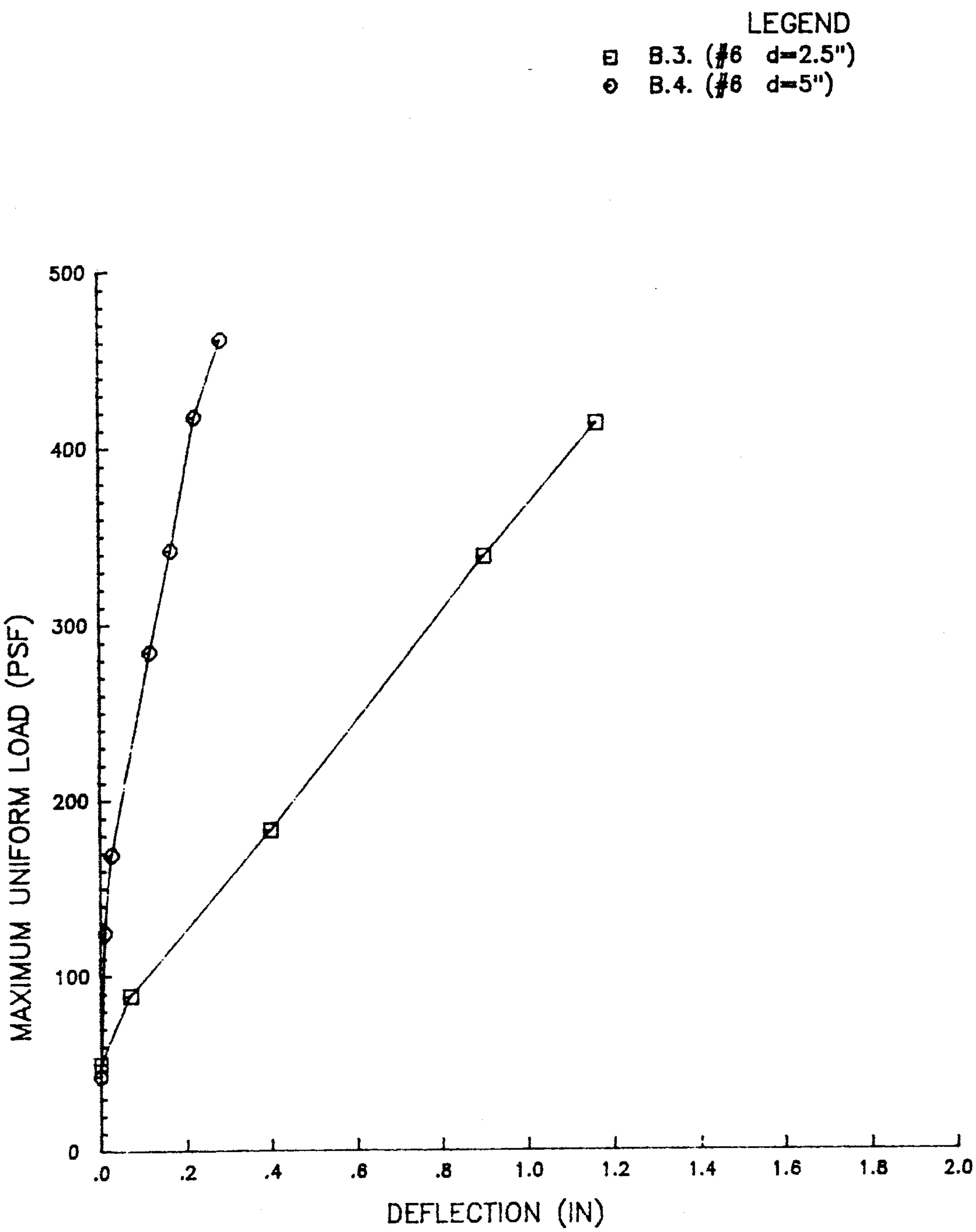

FIG 5.2.3 TESTS B.3 AND B.4

Table 5-2-2
Calculated Moment Capacity and Ultimate Moment
From Tests for Conventional Masonry Walls
(From Literature)

Nominal Cross Section (In.)	Reinforcement	d(in.)	a(in.)	Calculated Ultimate Moment (In-K)	Actual Ultimate Moment (In-K)	Ratio M_{Test}/M_{Calc}
48x9	#5@24"o.c.	5	.60	169.20	270.0	1.60*
48x8	#5@24"o.c.	5	.86	246.60	251.4	1.02*
81x8	#4@40"o.c.	3.87	.23	59.27	108.9	1.84**
81x8	#4@27"o.c.	3.87	.35	88.67	104.6	1.18**
84x8	#4@42"o.c.	3.87	.22	60.16	76.3	1.27**
72x8	#4@36"o.c.	3.87	.26	59.84	67.6	1.13**

*Refers to reference Saemann, 1955.

**Refers to reference Converse, 1946.

Table 5-2-3
Allowable Out-Of-Plane Uniform Load (PSF)
For 8" Hi-R Walls Reinforced with #4 @ 24" O.C.

h/t Ratio	Seismic	Wind
25	30	34
36	15	17

5.3 Analysis of Compression Tests on Walls

The compression test program on the walls was developed to investigate the combined effects of axial compression and bending. These tests were conducted to answer the question: Can the interaction relationship given in Equation 6-21 of Chapter 24 of the UBC be used to design walls constructed of Hi-R block?

Test Series C.1, C.2, and C.3 investigate the applicability of the working stress design method for walls loaded in axial compression and bending for cases using unreinforced and reinforced specimens. The walls are loaded in such way that both compression and flexure are induced into the wall specimen. Conventional working stress design relies on a linear relationship between the compression caused by the axial load and the compression caused by the flexure of the wall to account for these combined stresses. The typical formulation of this combined stress relationship is expressed using an interaction equation and this required that the proportion of the compressive stress caused by the axial load and the proportion of the compressive stress caused by the flexure must sum to less than unity.

5.3.1 Unreinforced Walls

Equation 6-1 of UBC Section 2406(c)2.A permits an allowable compressive stress of F_a=431 psi for concrete masonry with a compressive strength of 2215 psi. The compressive strength of the unreinforced masonry was obtained from Test Series E.2 and E.3 as described in Section 5.5. The allowable compressive stress caused by the flexure developed by the eccentric load is F_m=766 psi from Equation 6-6.

The calculated axial and flexural compressive stresses developed during the tests were f_a=1100 psi and f_m=410 psi, respectively. Substituting these values into the interaction equation we obtain safety ration of 3.1 as shown below.

$$f_a/F_m + f_m/F_m > 1.0$$

$$1100/431 + 431/766 = 3.1$$

Tests on conventional block reported by Fishburn (1961) and Hedstrom (1961) develop safety ratios on the order of 4.0 to 4.5. The interaction relationship of Hi-R and conventional block is presented in Figure 5.3.1. The reduction in the capacity of the Hi-R block may be explained by the existence of the smaller cross sectional area of the cross web. Although somewhat lower than for conventional block, the compressive strength of Hi-R block is still significantly greater than that required by the working stress design equations for unreinforced masonry.

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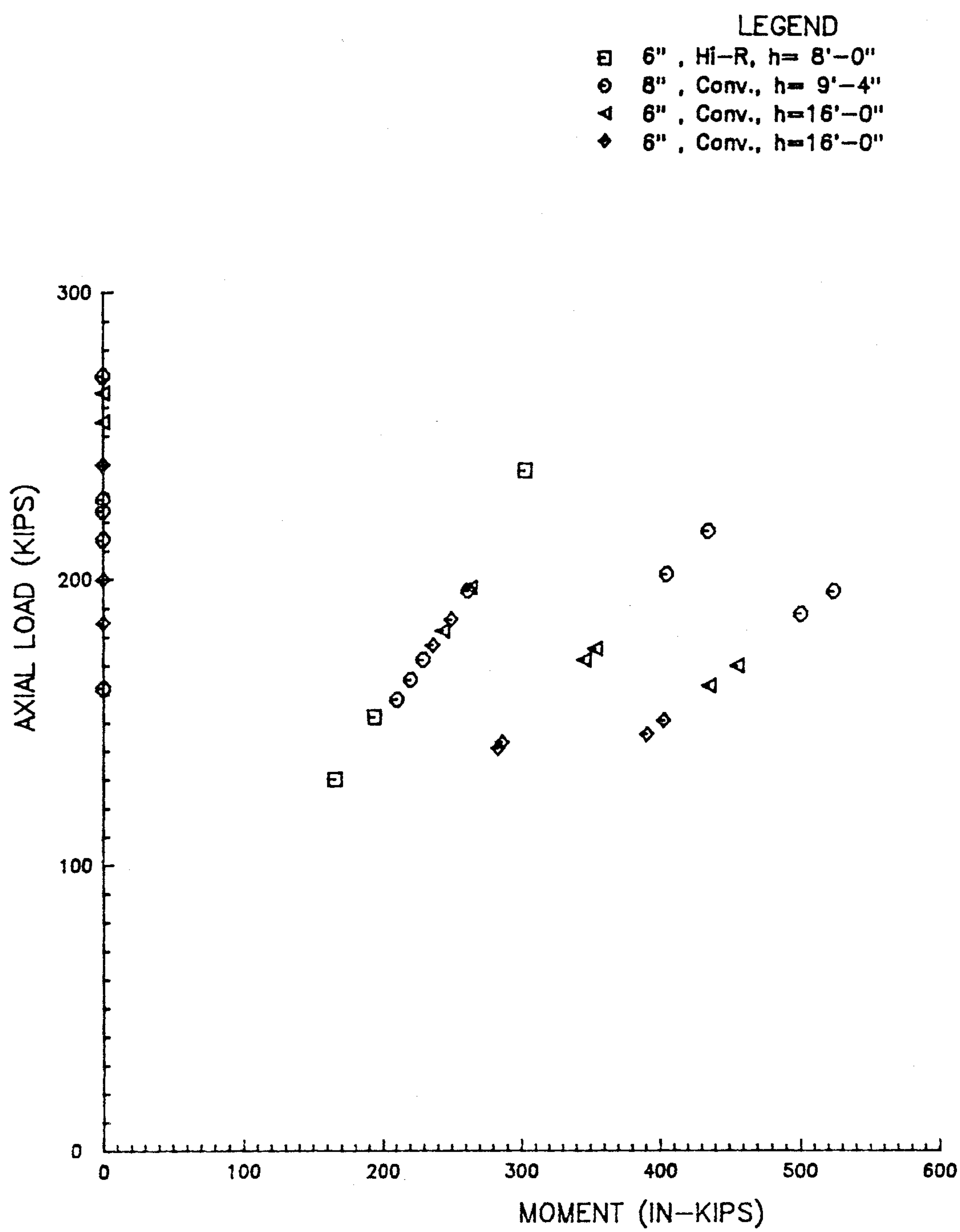

FIG 5.3.1 COMPARISON Hi-R – CONVENTIONAL BLOCK INTERACTION DIAGRAMS – UNREINFORCED (NCMA)

5.3.2 Reinforced Walls

5.3.2.1 Partially Grouted Walls

Equation 6-3 of UBC Section 2406(c)2.C permits an allowable compressive stress of F_a=415 psi for concrete masonry with a compressive strength of 2133 psi. The compressive strength of the unreinforced masonry was obtained form Test Series #.w2, E.3 and #.4 as described in Section 5.4.2.2 and 5.5. The allowable compressive stress caused by the flexure developed by the eccentric load is F_m=704 psi from Equation 6-6.

The calculated axial and flexural compressive stresses developed during the tests were f_a=1035 psi and f_m=869 psi respectively. Substituting these values into the interaction equation we obtain a safety ration of 3.7 as shown below

$$f_a/F_a + f_m/F_M > 1.0$$

$$1035/415 + 869/704 = 3.7$$

Using data from the back-up to the NCMA Code, the interaction relationship of Hi-R and conventional block is presented in Figures 5.3.2 and 5.3.3.

The interaction diagram is obtained by plotting the axial strength of the wall against the corresponding moment induced by the eccentric axial load for 6 inch conventional block in Figure 5.3.2. This figure develops the trend that for longer spans the moment and axial strength of the wall is less than that for a shorter span. A review of the interaction diagram for the conventional block and the points obtained from the 8 inch Hi-R block, figure 5.3.3, shows that both curves possess similar shapes although the 8 inch Hi-R block would possess less strength than a conventional 8 inch block.

One of the results obtained from Test C.1.2 is significantly lower than the other values obtained from that tests series. If this data point is disregarded in estimating the interaction curve for the Hi-R block, the interaction diagram moves out beyond the capacity indicated for the 6 inch conventional block wall with the longer span (9'4"). Nevertheless, if the anomalous result is included in the estimate of the interaction curve, the basic trend of the Hi-R interaction curve is quite similar to that of conventional block. Additional tests should resolve the proper location of the Hi-R interaction curve.

5.3.2.2 Fully Grouted Walls

Equation 6-3 of UBC Section 2406(c)2.C permits an allowable compressive stress of F_a=383 psi for concrete masonry with a compressive strength of 1970 psi. The compressive strength of the unreinforced masonry was obtained from Test Series E.4 as described in Section 5.5. The allowable compressive stress caused by the flexure developed by the eccentric load is F_m=650 psi from Equation 6-6.

The calculated axial and flexural compressive stresses developed during the tests were f_a=1483 psi and f_m=1493 psi, respectively. Substituting these values into the interaction equation we obtain a safety ratio of 6.2 as shown below:

$$f_a/F_a + f_m/F_m > 1.0$$

$$1483/383 + 1493/650 = 6.2$$

Based on this analysis and the comparison with the conventional block, it appears that the UBC interaction relationship produces reasonable design values for the Hi-R block.

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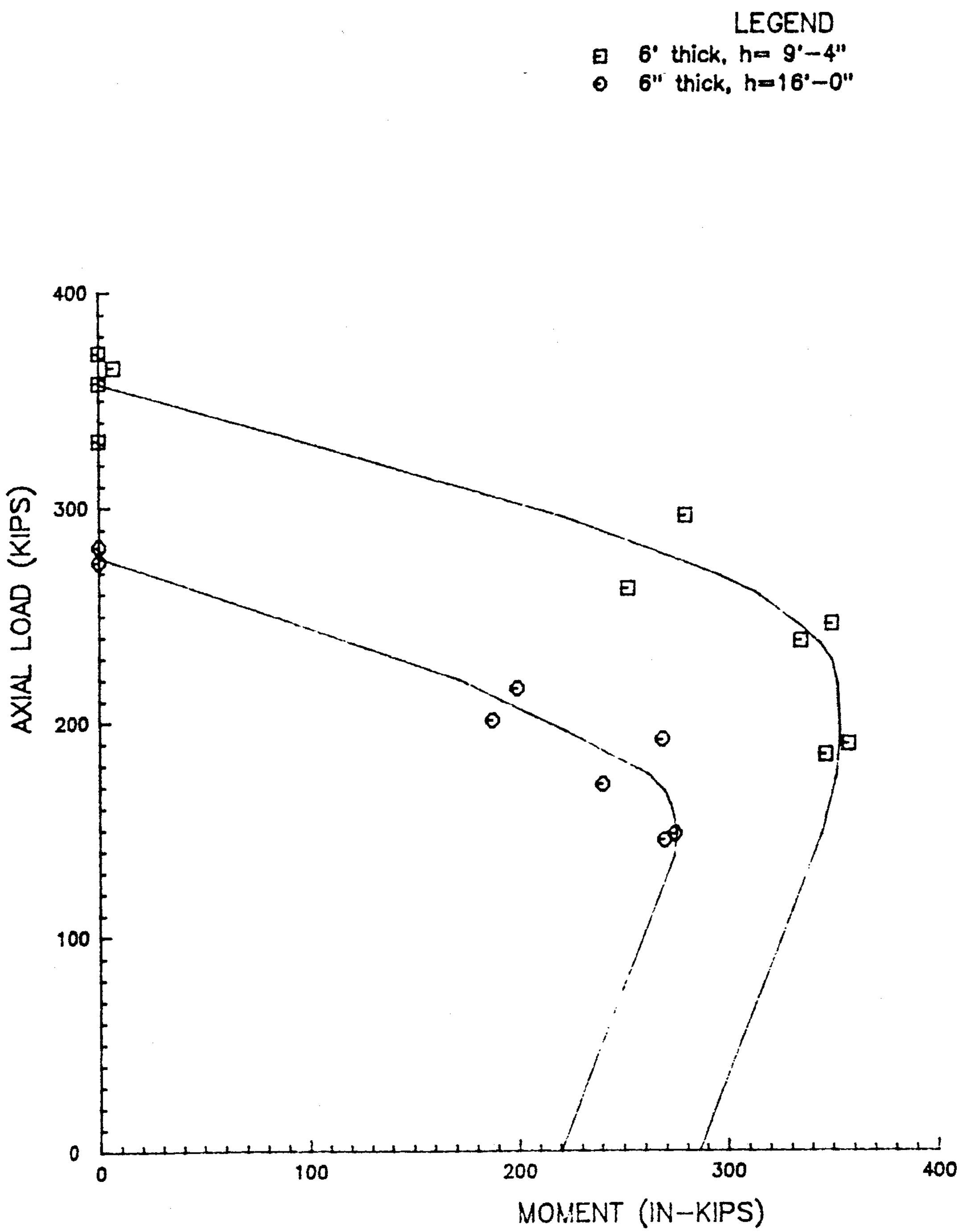

FIG 5.3.2 INTERACTION DIAGRAMS FOR CONVENTIONAL BLOCKS - PARTIALLY GROUTED (NCMA)

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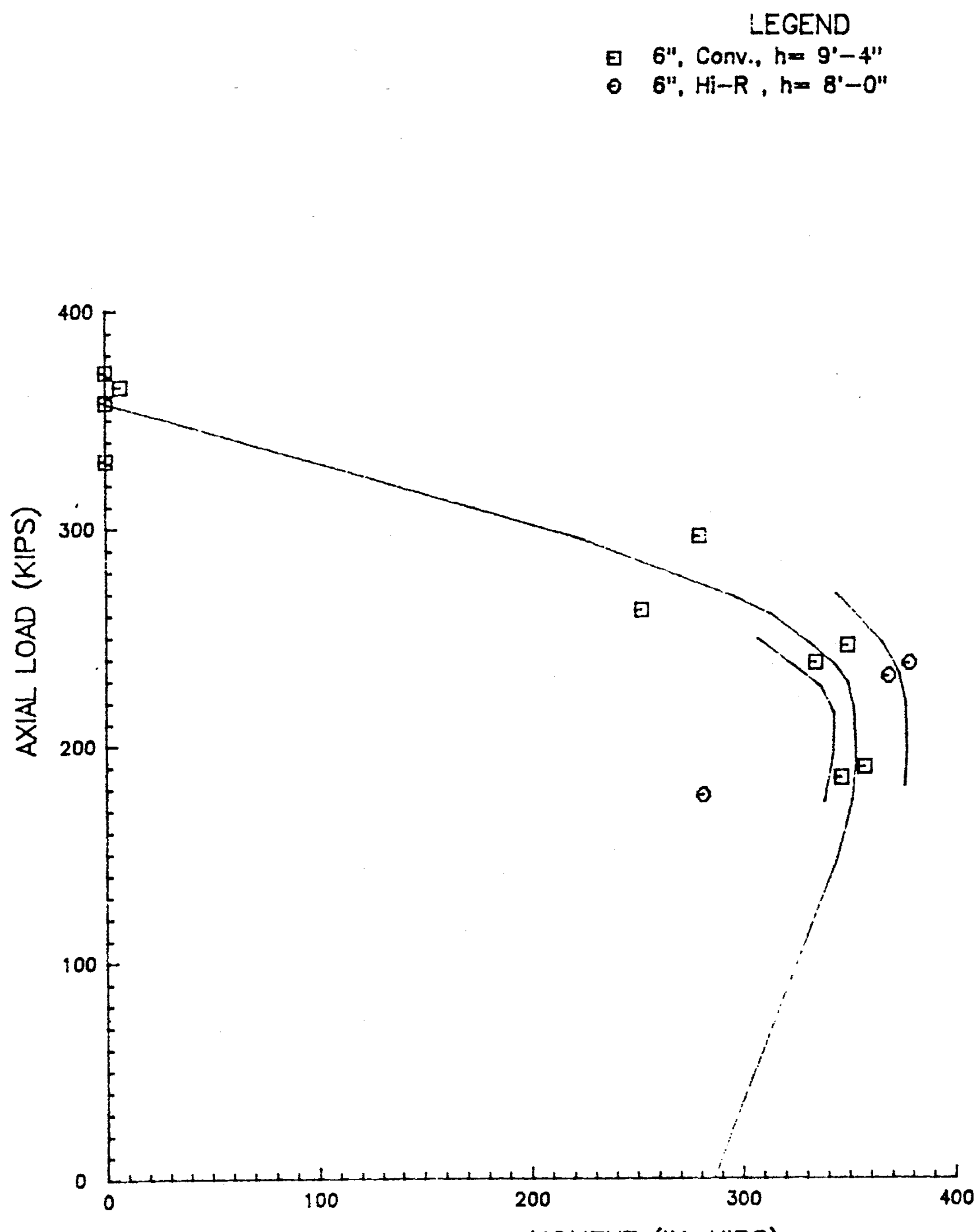

FIG 5.3.3 COMPARISON Hi-R – CONVENTIONAL BLOCK INTERACTION DIAGRAMS – PARTIALLY GROUTED (NCMA)

5.4 Analysis of Diagonal Tension (Shear) Tests

The out-of-plane loading test on the wall specimens was developed to investigate the out-of-plane flexural capacity of walls constructed of Hi-R block. The unsymmetrical cross section of the block suggested that the load be applied against both the insulated and uninsulated faces. These tests were conducted to answer the following questions:

(1) What is the shear coefficient (maximum shear strength from tests divided by the square root of f'_m) for each combination of grouting and reinforcement tested?

(2) Can the 1985 UBC provisions for shear stress be used to design Hi-R block for shear?

(3) What is the safety ration provided by Hi-R blocks in shear compared to conventional block using the comparisons described in (2)?

A comparison between the performance of Hi-R block and conventional block is included in the evaluation of the tests results. Inasmuch as the safety ratios reported in the literature are calculated without taking into account the 1.33 stress increase for wind and seismic loads, a similar procedure is adopted in this evaluation.

5.4.1 Ungrouted, Unreinforced Blocks (Test Series D.3)

The results of the diagonal tension (shear) tests on the ungrouted, unreinforced blocks produce a mean shear stress of 162.5 psi. The result of the test D3.3 was not included in the calculation of the mean because this specimen was inadvertently grouted. The reported result of the tests is significantly higher than that obtained from the other specimens and, although the grout was removed from the specimen, this grouting and then partial removal probably accounted for the anomalous result.

Assuming f'_m=2215 psi, computed from test series E.2 and E.3 using UBC Section 2406(b)2.A, the value of the shear coefficient is 3.5. As a point of comparison, the allowable shear stress for unreinforced masonry is 34 psi. The resulting safety ratio for the HI-R block is 162.5/34=4.8. This safety ratio compares favorably with the tests of conventional block used in the development of the NCMA Code (Fishburn and Cyrus, 1961). The reported safety ratio for conventional block is 3.9.

Based on an analysis of the tests results and a comparison to the results obtained from tests of conventional block, it appears that the UBC working stress value of 34 psi for unreinforced, ungrouted Hi-R block is a reasonable design value.

5.4.2 Reinforced, Grouted Block (Test Series D.2)

The samples used in this test series contained at least the minimum reinforcement required in the UBC. In Seismic Zones 3 and 4, the minimum steel ratio shall not be less than 0.0007 based on the gross cross section. The reinforcement ratio provided in the test specimens is 0.0011.

5.4.2.1 Reinforced, Fully Grouted Blocks

The results of the diagonal tension (shear) tests on the fully grouted, reinforced blocks produce a mean shear stress of 290.0 psi.

Assuming f'_m=1970 psi, computed from test series E.4 using UBC Section 2604(b)2.A, the value of the shear coefficient is 6.5 Using section 2406(c)7B, the allowable shear stress given by Equation 6-10 with M=0 is 60 psi. The resulting safety ratio for the HI-R blocks is 290/60-4.9. This safety ratio compares favorably with the tests of conventional block used in the development of the NCMA Code (Schneider,1959). The reported safety ratio for conventional block is 2.9.

Based on an analysis of the test results and a comparison to the result obtained from tests of convetional bloc, it appears that the UBC working stress values based on the referenced equations from the UBC for fully grouted, reinforced Hi-R block produce reasonable design values.

The improved performance of the fully grouted blocks compared to the performance of the partially grouted blocks may be due to the fact that the weak line in the failure mechanism is the failure of the cross webs in tension. This tension is produced as a result of the tendency of the face shell to buckle on the insulated side.

5.5 Analysis of Masonry Prism Tests

The prism test program on the Hi-R block specimens was developed to investigate the compressive strength (f'_m) of the masonry for grouted and ungrouted sections. These tests were conducted to answer the following questions:

(1) Is the compressive strength of the Hi-R prism sufficient to permit the use of Section 2406(b)4 of the UBC to obtain the specified compressive strength in lieu of prism tests?

(2) What is the impact of the HI-R inserts on the net compressive strength for grouted units?

(3) How does the statistical scatter of the prism test data for Hi-R block compare with the statistical scatter reported for prism tests of conventional block?

(4) Does the type of mortar bedding have a significant influence on the net compressive strength?

If prism tests are not conducted, the UBC permits an assumed value of f'_m to be used. The value of the f'_m can be obtained from Table 24-D through an interpolation of the compressive strength of the Hi-R units, 2810 psi and the mortar type, Type S. For this combination of material strengths, the assumed value of f'_m is 1700 psi.

The statistical scatter of the prism tests appears to compare favorably with the scatter reported in the literature. Considering all of the test data, the mean value is 2147 psi with a coefficient of variation of 9.5%. This scatter compares to that reported by Yokel et al. (1970) as 11% and 16% for 6 in. and 8 in. conventional units, respectively. If two extreme results are eliminated from the tests, Sample E.1-2 and E.4-3, the mean changes only slight to 2147 psi but the coefficient of variation is reduced to 3.2%. Thus, it seems reasonable to conclude that the statistical scatter of the compressive strengths of the Hi-R block will not produces significant variations in the actual values of f'_m and lends further support to the use of the assumed values from Table 24-D if desired by the design engineer.

Test Series E.2 and E.3 compared the effect of full mortar bedding with face shell bedding. The mean compressive strength is 2210 psi and 220 psi for Test E.2 and E.3. The difference of 0.5% is insignificant compared to the scatter of the test results and there does not appear to be any difference in the compressive strength of the fully bedded and face shell bedded prisms.

In conclusion, the results reported for the prism tests support the use of compressive strengths obtained from the assumed values in Table 24-D or from prism tests. The statistical scatter of the data is sufficiently small to maintain confidence that the assumed values of compressive strength will be less than the actual compressive strength of the prisms. The effect of mortar bedding on the compressive strength appears to be insignificant.

5.6 Design Recommendations

Korfil Hi-R Masonry is recommended to be designed in accordance with conventional masonry design provisions except as modified by this section. For those jurisdictions governed by the Uniform Building Code, pre-insulated Korfil Hi-R masonry should be designed in accordance with UBC Chapter 24 except:

(1) The allowable shear stresses permitted by UBC Section 2406(c)6.A. and Section 2406(c)6.B. shall be reduced by 10% for fully grouted walls.

(2) The allowable shear stresses permitted by UBC Section 2406(c)6.A. and Section 2406(c)6.B. shall be reduced by 20% for nongrouted or partially grouted walls.

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