A ONE-QUARTER SCALE EXPERIMENTAL STUDY OF THE EFFECT OF PERIPHERAL REINFORCEMENT IN A CONCRETE HOLLOW-BLOCK SHEAR WALL

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by

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ABSTRACT

The objective of this thesis is to investigate the behavior of precompressed masonry square shear walls for a one-storey building. 1/4th scale models of channel shaped walls with and without perimeter reinforcement were chosen as the subject of the investigation. It was assumed that the concrete infill and perimeter function together and a unity of action exists between the infill and perimeter. The strength and stiffness of the shear walls were assumed to be a function of bond strength and slippage at the masonry/mortar interface of the concrete infill, of the continuity of perimeter reinforcement and of the precompressive stress.

In the study it was apparent that there was lack of fundamental knowledge concerning strength and stiffness of these types of structures and the influence of bond strength and slippage at the masonry/mortar interface on the strength and stiffness of the masonry shear wall structures. Secondary tests were performed and on the basis of the results of these tests, formulae for predicting the strength and stiffness of these types of masonry shear walls were proposed. Some recommendations for design and future research are also suggested.

ACKNOWLEDGEMENTS

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CHAPTER 1

INTRODUCTION

The systems providing stability to buildings constructed in early years were essentially an extension of rigid structural frames; the stability of such frames depends largely on the monolithic connection between the columns and beams.

In recent years the number of tall buildings has increased and it has become more important to ensure adequate lateral stiffness and strength to resist load which may arise due to wind, earthquake or blast effect. This lateral stiffness and strength may be achieved in various ways in framed structures. It may be obtained by bracing members, by rigidity of the joints, by complete shear truss assemblies acting in conjunction with the frame or by infilling the frame with shear resistant panels. The most common methods of providing the required rigidity has been by using the following types of structures:

(1) Shear wall structure

(2) Shear walls combined with frames

(3) Infilled frames

In 1967 A. COULL and B. STAFFORD SMITH[5] described in a review of previous research that the first major research into pure shear wall structures was a series of tests carried out in the United States to provide data for the design of blast resistant structures which were tested later in full scale by atomic explosions at Enewetok, and a report was presented by WHITNEY, ANDERSON and COHEN in a comprehensive

paper dealing with the behavior of structures designed to resist blast loads.[5] They proposed approximate methods based on simple "Strength of Materials" theories to predict the stiffness and ultimate load of shear wall structures, subjected to lateral loading. The work was continued in a systematic investigation carried out by JACK R. BENJAMIN and H.A. WILLIAMS, a large number of tests were made on large scale models of single storey brick and reinforced concrete shear walls.[1,8] In 1948 the investigation of infilled frames subjected to racking loads was started by POLYAKOV at the Central Research Institute for industrial structures in Moscow[5]. The research programme included a study of the shear and tensile strength of the brickwork as a preliminary to a large scale experiment on infilled, pin connected steel frames with and without openings. POLYAKOV found that the usual mode of failure of complete infills was by cracking of the mortar joining along the compression diagonal.

In 1945 WHITNEY, ANDERSON and COHEN presented a paper in which they proposed an approximate method of predicting the lateral stiffness and strength of the infilled frames[5]. They assumed that the columns and infills act integrally as the flanges and webs respectively of a vertical cantilever.

In 1951 the project was continued at Stanford University, California, by BENJAMIN and WILLIAMS[8]. Large scale and model infilled frames were tested, the frames were of reinforced concrete and steel, and the infills were of plain concrete, reinforced concrete and brickwork.

In 1961 HOLMS[5] of Leeds University, England, proposed a method for predicting the deflection and strength of infilled frames

based on the concept of the infill being equivalent to diagonal strut, to be of the same thickness and modulus of elasticity as the infill, with a width equal to one-third the diagonal length of infill.

In 1961, J.A. BLUME, N.M. NEWMARK and L.M. CORNING,[11] discussed the behaviour of masonry walls inclosed in reinforced concrete frames. In the particular example they discussed, the frame and infill were analyzed separately and the analysis indicated that the frame alone would resist about 13 kips and the strength of the masonry wall alone 6 kips for a total of 19 kips. However, the combined system was observed to resist 40 kips. Therefore, the authors suggested that reinforced concrete frame with masonry wall cannot be considered as an individual frame and individual wall acting "in parallel" but must be considered as a single structure unit.

C. CARTER and B. STAFFORD SMITH[9] described the behaviour and analysis of infilled frames:

> "When an infilled frame is subjected to a racking load, the frame separates from the infill except in regions around the end of the compression diagonal and the load is transmitted from the frame to the infill through these regions or 'lengths of contact' and in effect, this infill behaves as a diagonal strut to brace the frame."

B.P. SINHA and Professor A.W. HENDRY[10] investigated the rigidity and shear strength of a single-storey shear wall structure comprising three shear panels and having door openings between two of the panels. They also derived formulae for calculating the ultimate shear strength of the precompressed brick shear wall structures.

It is apparent that considerable knowledge has been gained in recent years about the behaviour of shear wall structures but the number of the unknowns and the complexities are still so great that

design methods and procedures of analysis do not yet enjoy a complete mathematical analysis for all types of shear wall structures and for the behaviour of different materials. Engineers have learned from experiments, how to design and construct the shear walls and they have developed some rules as a result of their experience. However, such empiricism, without supporting analysis is not the most satisfactory approach for all types of shear wall structures.

Many types of materials can be used and there are many ways of construction that can be efficient and economical in providing the necessary stiffness and strength in shear walls. It was beyond the scope of this investigation to study the elements of all types of shear wall structures. However, the writer undertook preliminary investigation of one economical way to provide the necessary stiffness and strength in precast concrete hollow block square shear walls for a one-storey building. It is hoped that this investigation will shed some light on the behaviour of such structural units and could be used as the basis for further investigation.

Objective and Scope of Investigation

This investigation was conducted at the Civil Engineering Concrete Laboratory of the University of Manitoba in 1971.

The objective of the investigation was limited to a study of the behaviour of concrete masonry square shear walls for a one-storey building, and on the basis of experiment to a comparison of them to some of the recent researches on masonry shear wall structures for one-storey buildings.

The report does not propose to compare the behaviour of masonry with plain and reinforced concrete shear wall structures because the components of the concrete masonry shear walls are fundamentally different from reinforced concrete shear walls. In concrete masonry shear walls the "concrete" is supplied in three different and distinct ways: The precast concrete in the block units, the mortar, and the concrete fill. In contrast, only monolithic concrete is used in reinforced concrete shear walls.

In order to compare concrete masonry shear wall structures with brick-work and infilled frames, the validity of a number of assumptions and factors must be verified.

A. Assumptions

In this investigation the following assumptions are made:

1. In plain masonry shear wall structures, the strength and stiffness are functions of the bond strength diagonal tensile strength, unit precompressive stress and the joint slippage of the composite masonry unit
2. In masonry walls with perimeter reinforcement the inside masonry work is connected to the perimeter and there is a complete unity of action between the masonry infill and perimeter. Therefore, the strength and stiffness of this type of shear wall is a function of bond strength, diagonal tensile strength, unit precompressive stress unit compressive strength of the masonry work, the amount of steel, and to a certain extent the joint slippage of the unit masonry composite. There will be some other factors that would also have an

effect on the strength and stiffness of the shear wall structure, such as the base rotation and faced walls which will be discussed later.

B. Factors

In this investigation the following factors will be studied:

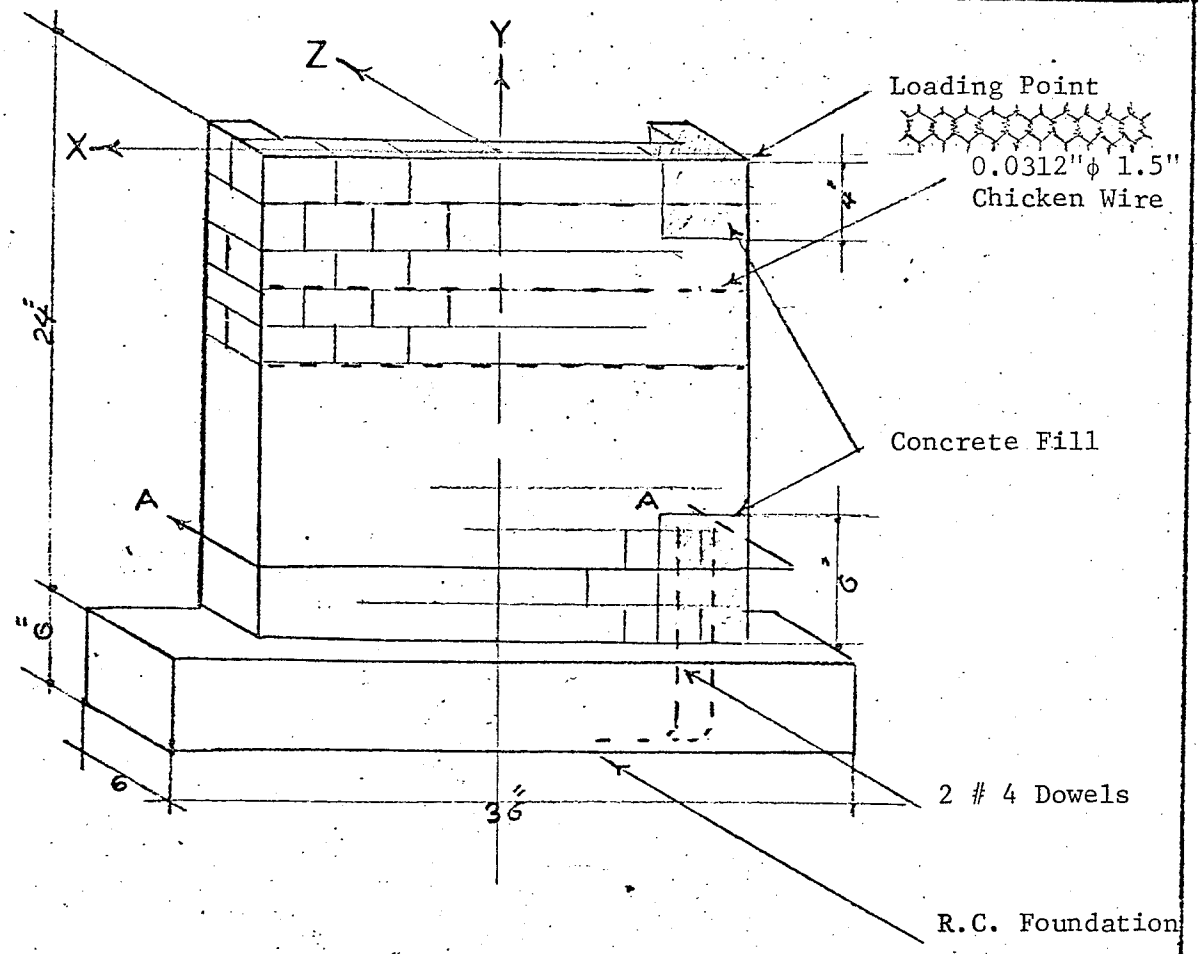
1. The load at first crack, Vcr
2. The deflection at first crack, Δcr
3. The load at failure Vult
4. The deflection at failure Δult
5. The general behaviour, modes of failure and the failure characteristics.

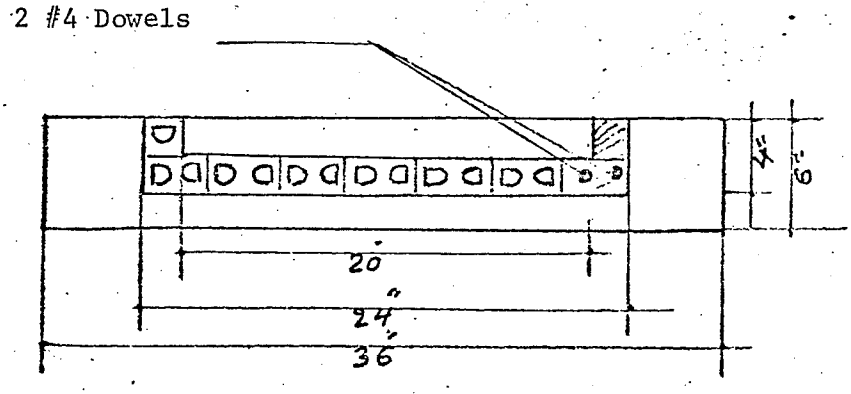
The scope of this investigation was limited to six 1/4th scale models of the following square shear wall structures for a one-storey building:

1. Three concrete hollow block shear walls without perimeter reinforcement as shown in Figure 1.
2. Two concrete hollow block shear walls with spliced reinforcing bars in the perimeter as shown in Figures 2 and 3.
3. One concrete hollow block shear wall with continuous reinforcing bars in the perimeter as shown in Figures 2 and 4.

Since the face walls and precompressive stress have an influence on the strength and stiffness of the shear wall structure, it was decided to build the walls with flanges and subject the walls to precompressive stress to simulate the actual condition of the shear walls in a typical building.

Secondary tests of the bond strength, diagonal tensile strength, joint slippage, compressive strength of the unit masonry composite and their relationships with one another were included in this investigation. It was also the intention of the writer to develop mathematical expressions for predicting the lateral load and deflection of the shear wall structure at first crack and failure.

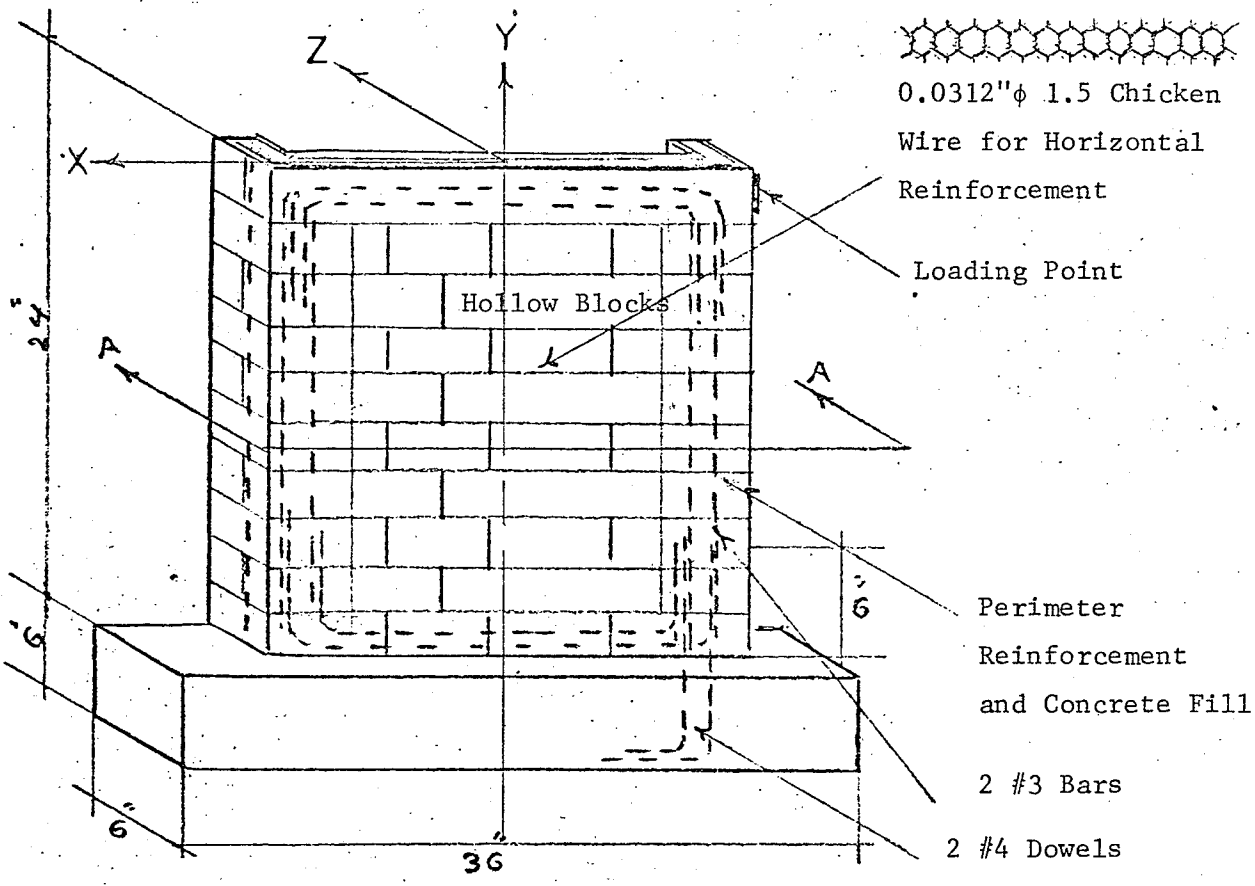


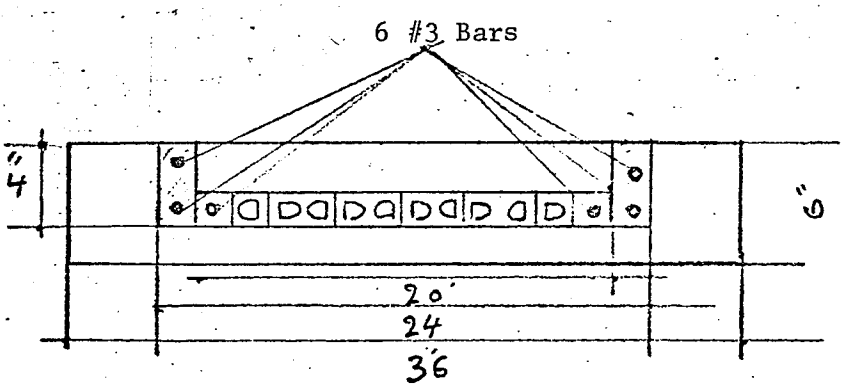


Section - AA

Details of Wall No. 1, 5, and 6

Figure 1.

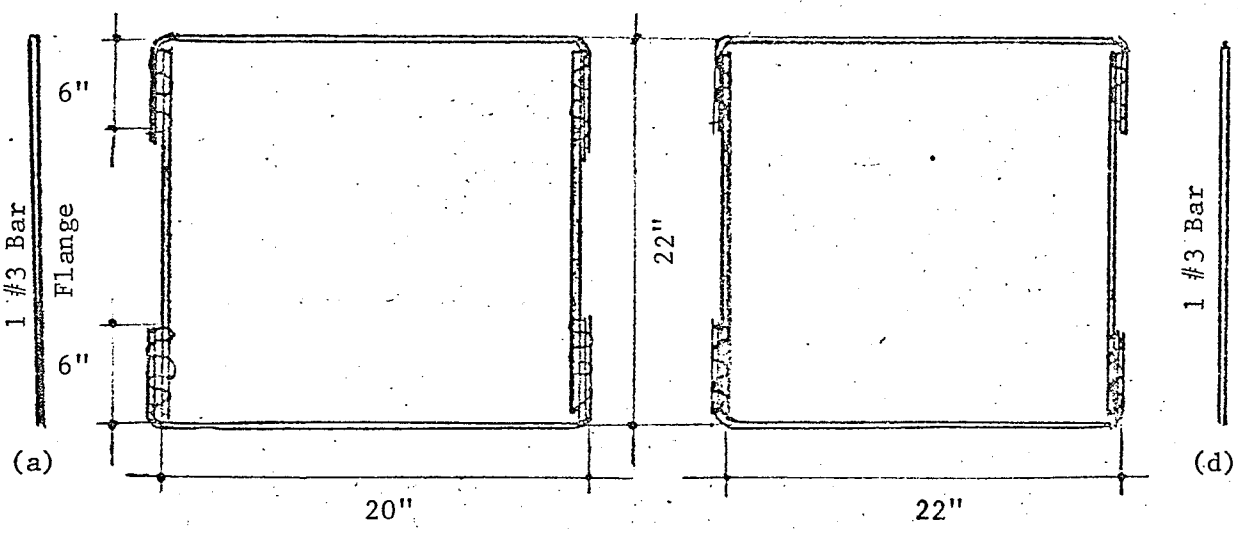




Section - AA

Detail of Wall No. 2, 3 and 4

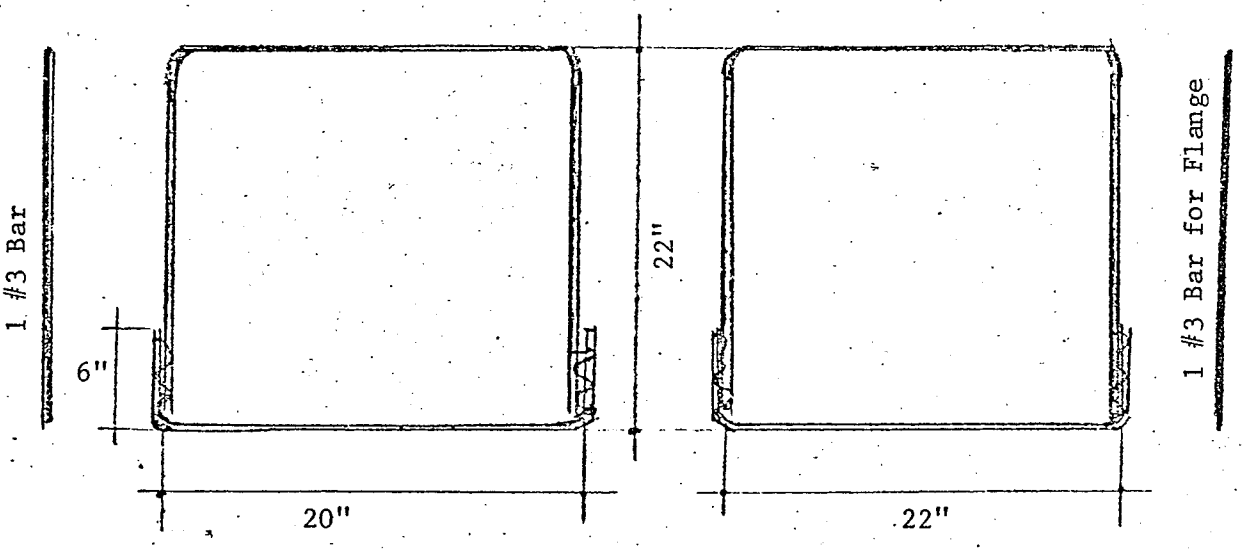
Figure 2.



(b) 1 #3 Bar for Second Core (c) 1 #3 Bar for First Core

Steel Details for Wall No. 2 & 3

Figure 3.



1 #3 Bar for Second Core 1 #3 Bar for First Core

Steel Detail for Wall No. 4

Figure 4.

CHAPTER II

MATERIAL PROPERTIES

The materials used in this investigation had the following properties:

1. Precast Concrete Block Units

One-fourth scale precast hollow blocks were used. The blocks were cast in the Civil Engineering Concrete Laboratory by the writer along with helpers. Size and shape of the blocks are shown in Figures 5 and 6. The concrete for casting the blocks was mixed in a dough mixer in the Road Materials Lab, as shown in Figure 7. Each 9 lb. batch of concrete contained 2 lbs of high early portland cement, 2 lbs, of sand, 4 lbs of coarse aggregate and 1 lb. of water. The concrete in the molds was vibrated by a pencil vibrator as shown in Figure 8. The blocks were removed from the molds after 24 hours and stored in cardboard boxes inside the laboratory for air curing for over 30 days before mortaring the walls. Compression tests were then carried out on the blocks. The results of the sieve analyses of the sand and aggregate and the compression tests are listed in Tables I and IV respectively.

2. Mortar

The mortar for casting the walls was mixed by spoon in 9 lb. batches. Type N mortar was specified in NBC-70[12] with a minimum compressive strength of 750 psi at 28 days. The proportion of cement and sand was 1:3; masonry cement and Selkirk silica sand were used for the mortar; the water cement ratio was 60%. Compression tests were carried out on 1 in.x 1 in. air cured mortar cubes at 14 days, the results of the sieve analyses for the sand and the compressive tests are listed in Tables III and VI.

3. Concrete Fill

The concrete fill for the perimeter was mixed by spoon in 8 lb. batches. The proportion of the cement, sand and aggregate was 1:1:2. High early Portland cement and the same type of sand and aggregate were used as for precast concrete blocks with a water cement ratio of 70%. Compression tests were carried out on 3 in. by 6 in. air cured cylinders at 14 days. The results of the tests are listed in Table V.

4. Reinforcing Steel

For all horizontal reinforcements 1.5-inch hexagonal chicken wire (1/32 inch diameter and 49,600 psi yield strength and for perimeter reinforcement #3 bars with deformed sections and 52,650-psi yielding strength was employed. The result of the tensile tests are listed in Table II.

TABLE I

SIEVE ANALYSIS OF AGGREGATE AND SAND FOR BLOCK

AGGREGATE		FINE SAND	
Sieve Size	% Retained	Sieve Size	% Retained
#4	0.0	#10	2.09
#8	71.0	#20	11.8
#10	8.3	#30	12.9
#16	11.6	#50	47.0
#20	2.9	#100	15.1
#30	2.7	#200	6.3
#pan	3.5	#pan	4.1
Water Content = 5.0%		Water Content = 9.5%	

TABLE II

TEST DATA FOR REINFORCING BARS

#3 Bars (ϕ = 0.375)			1½ Chicken Wire (ϕ = 1/32")		
Test No.	Yield (psi)	Ultimate (psi)	Test No.	Yield (psi)	Ultimate (psi)
1	51300	72200	1	55200	58000
2	52400	73400	2	48200	58500
3	52200	73500	3	46300	57300
			4	46300	64000
Average	52650	73330	Average	42600	59450

TABLE III

MORTAR & PLASTER OF PARIS UNIT COMPRESSIVE STRENGTH

Test No.	Masonry Cement Mortar 1 : 3		Plaster of Paris		Ratio f^p_u/f^m_u
	P_u	f^m_u	P_u	f^p_u	
1	730	730	725	725	0.995
2	610	610	610	610	1
3	640	640	810	810	1.27
Average	660	660	715	715	1.08
Note Test after 14 days Size of cubes 1 x 1 x 1 in.			Note Test after 24 hours Size of cubes 1 x 1 x 1 in.		

TABLE (IV)

PRISM & SINGLE BLOCK UNIT COMPRESSIVE STRENGTH

Test No.	Masonry Prism		Single Block		Ratio
	P_u lbs	f'_m psi	P_u lbs	f'_m psi	
1	11310	2820	13300	3330	1.18
2	10770	2670	15000	3740	1.4
3.	15100	3770	14900	3730	0.995
Average	12393.30	3080	14400	3600	1.157

TABLE V

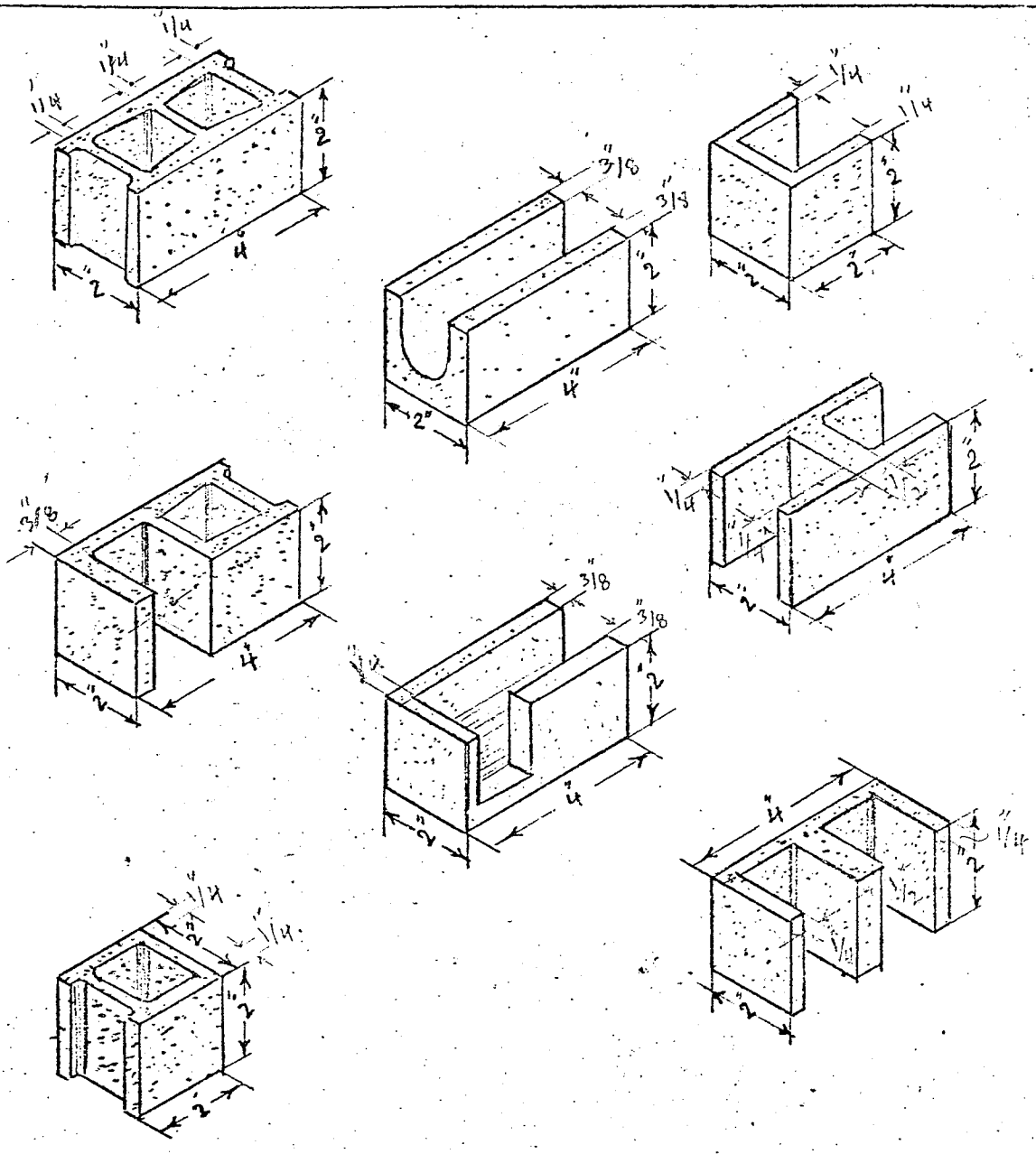
TEST DATA FOR CONCRETE FILL AT 14 DAYS

Test No.	P_u lbs	fc' (psi)
1	25600	3470
2	24000	3260
3	20300	2760
Average	23300	3163
Cylinder - 3 in.x 6 in.		
A = 7.36		

TABLE (VI)

SIEVE ANALYSIS OF SELKIRK SILICA SAND FOR MORTAR

Sieve Size	% Retained
#30	18.2
#50	52.0
#80	24.4
#100	3.24
#200	1.10
Pan	0.9



Nominal Size and Detail of
Precast Concrete Blocks

Figure 5

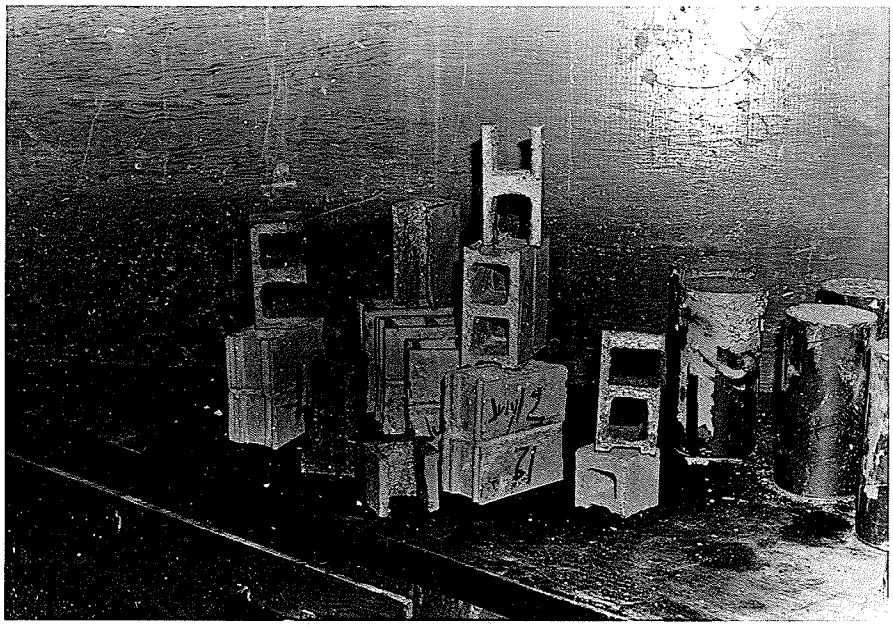


Figure 6 - A Photograph of 1/4th Scale Concrete Blocks.

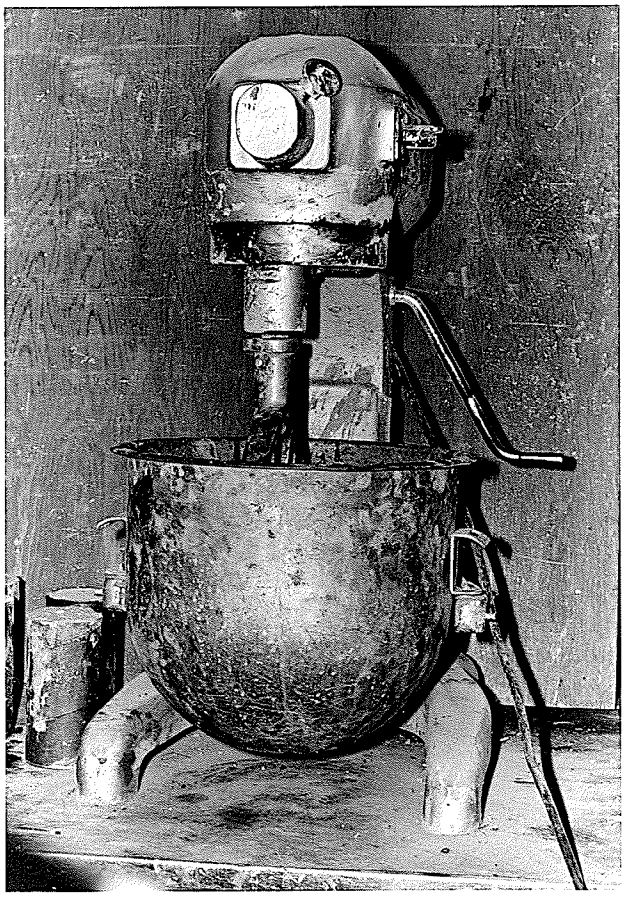


Figure 7 - Concrete and Mortar Mixer (Capacity 3/4 ft.^{3})

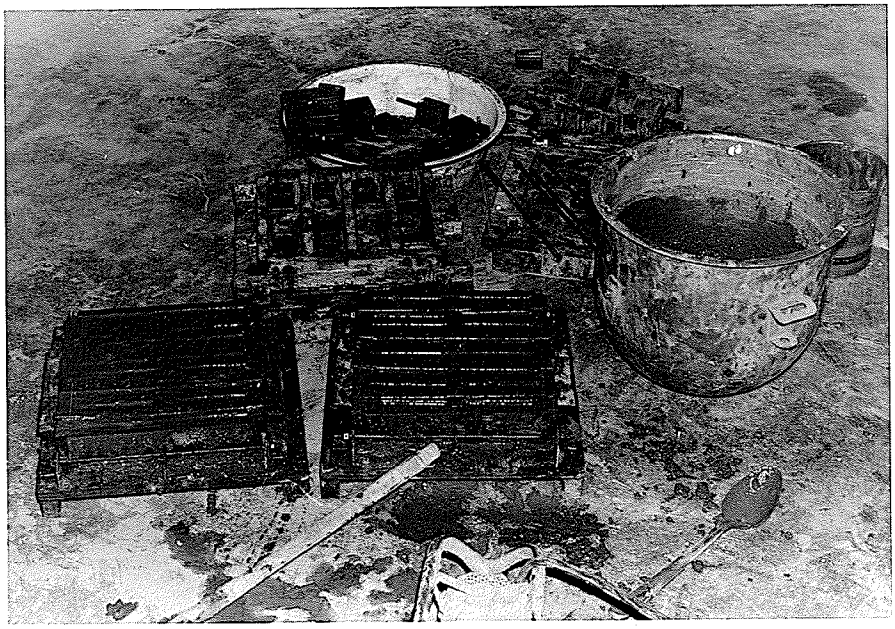


Figure 8 - Showing the Molds for Making Blocks and the Vibrating System.

CHAPTER III

CONSTRUCTION METHOD

There were three stages of construction in the wall models.

(a) Construction of reinforced concrete base.

(b) Construction of three square walls with no perimeter reinforcement.

(c) Construction of three square walls with perimeter reinforcement and special openings at the corners for tying the reinforcement at the joints.

All walls were constructed on the base beams by an experienced brick-layer, along with the writer, using ordinary construction methods. The walls and flanges had a thickness of one block (2").

(a) Construction of Reinforced Concrete Base

For each model 3 ft.x 6 in.x 6 in. reinforced concrete beams were casted and in each beam two dowels were placed during the casting. The details of the beams are shown in Figure 9.

(b) Walls Without Perimeter Reinforcement

Three square walls (24 inches in length, 24 inches in height, and 2 inches in thickness) without perimeter reinforcement were built on the reinforced concrete bases. The surface of the beams were roughened by chiselling and wetted, before building the walls on the base beams, and each block was dipped in water for

one second during construction. The thickness of the mortar joints were approximately 3/16 inch. On every second course of the blocks strips of 1.5-inch hexagonal chicken wire were used for horizontal reinforcement. The cores containing the dowels from the bases were filled with concrete having the same properties as the precast concrete blocks. These details are shown in Figure 10.

(c) Walls With Perimeter Reinforcements

Three square walls (24 inches in length, 24 inches in height, and 2 inches in thickness) with perimeter reinforcement were built on the reinforced concrete bases. The surfaces of the beams were roughened by chiselling and wetted before building the walls on base beams as in the previous walls. Each block was dipped in water for one second during construction, as before. For bottom and top courses, lintel blocks were used , (see Figure 11) and laid in sequence from one end to the other using a steel angle as a guide to maintain alignment. Two #3 deformed bars and concrete were placed in the bottom courses. The corners of the walls were constructed from "H" shaped, "E" shaped, "6" shaped and "U" shaped blocks. The thickness of the mortar was 3/16 of an inch. The details of construction are illustrated in Figures 11 to 16.

Twenty-four hours after mortaring the walls, the vertical and top reinforcing bars were placed inside the

core and tied with wire through the openings at the corners. Strips of 1.5-inch hexagonal chicken wire were used for horizontal reinforcement. The details of the top reinforcement and the openings at the corners before the cores were filled with concrete are shown in Figures 17 to 19. The details of web and flange connections are shown in Figure 20.

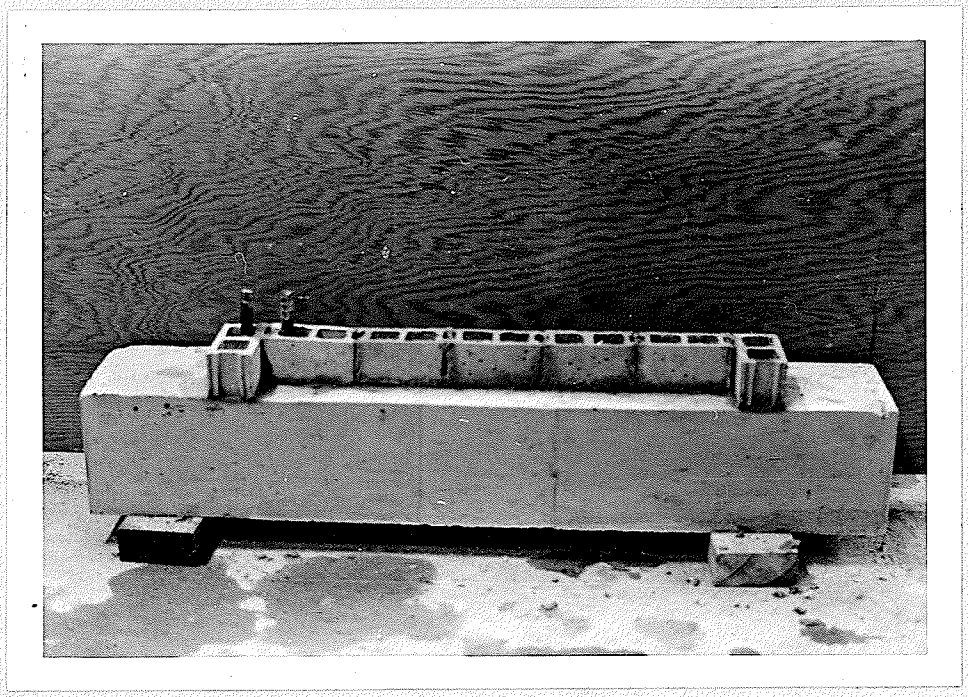


Figure 9 - Showing the Concrete Base, Dowels and the First Course of Plain Masonry Wall.

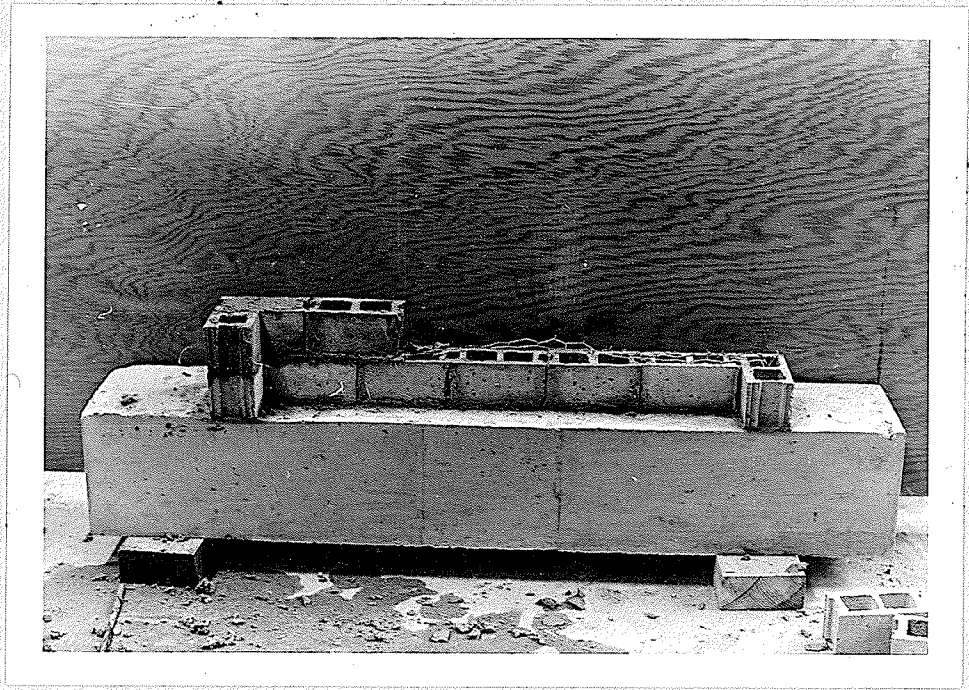
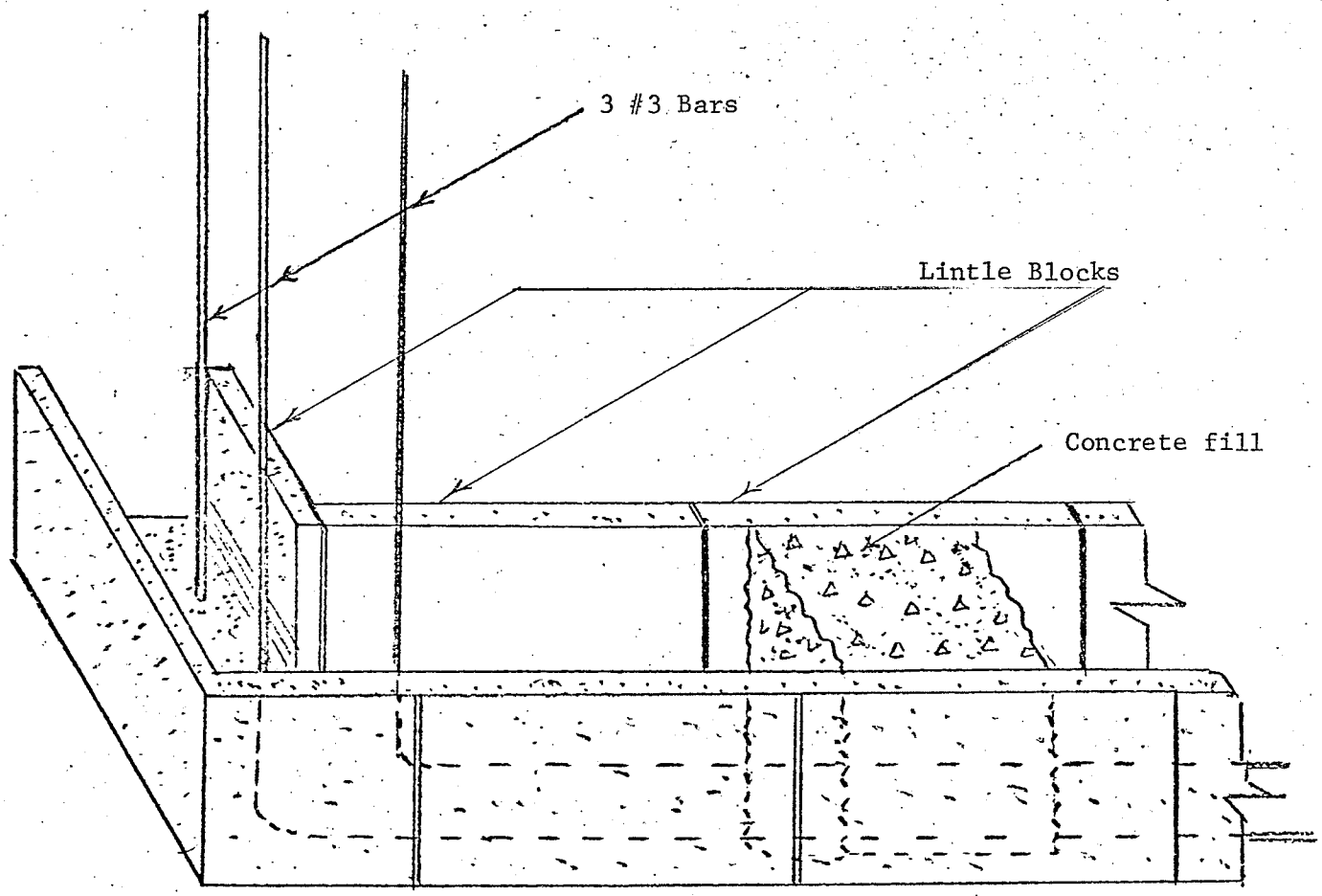
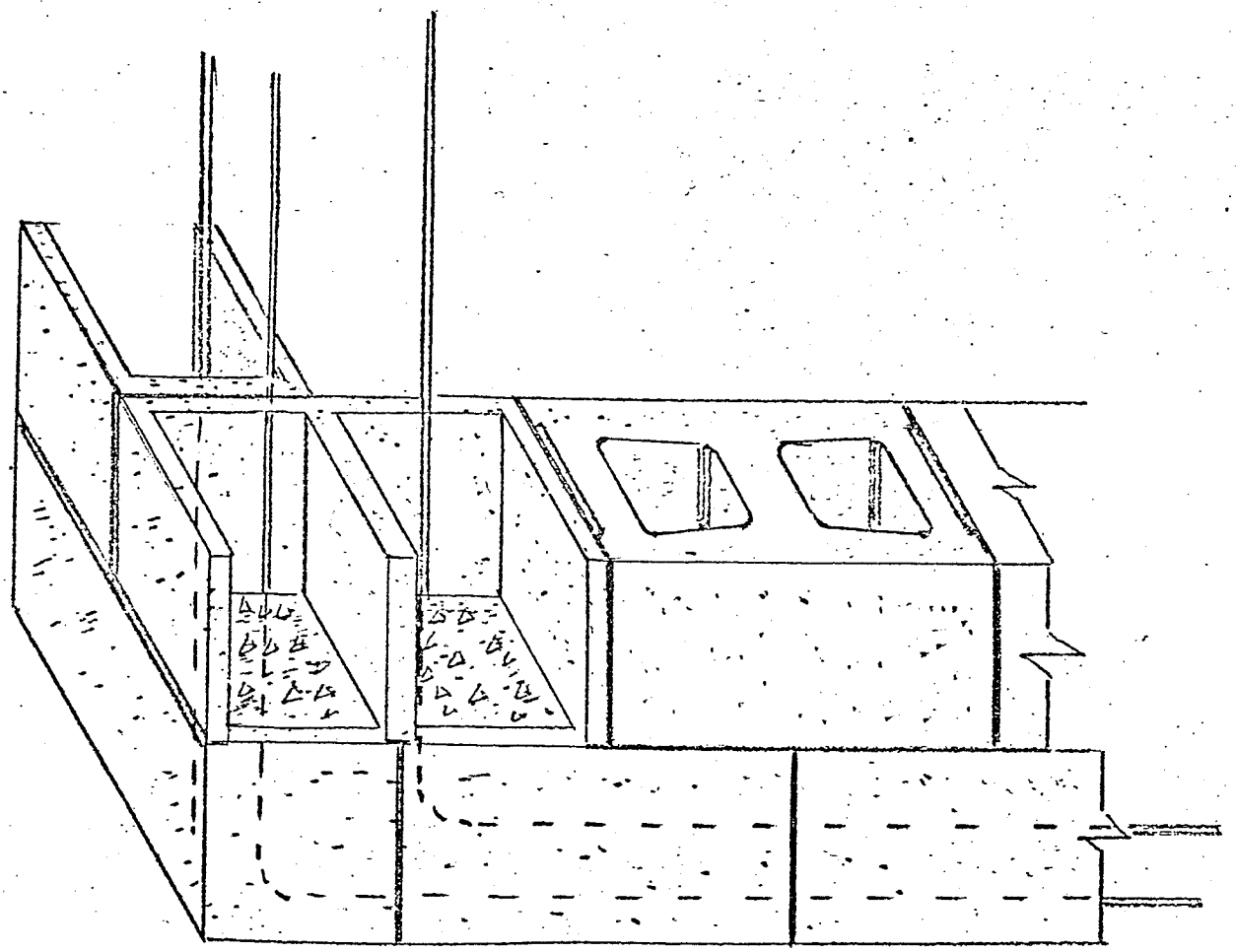


Figure 10 - Showing the Second Course and the Chicken Wire for Horizontal Reinforcement.



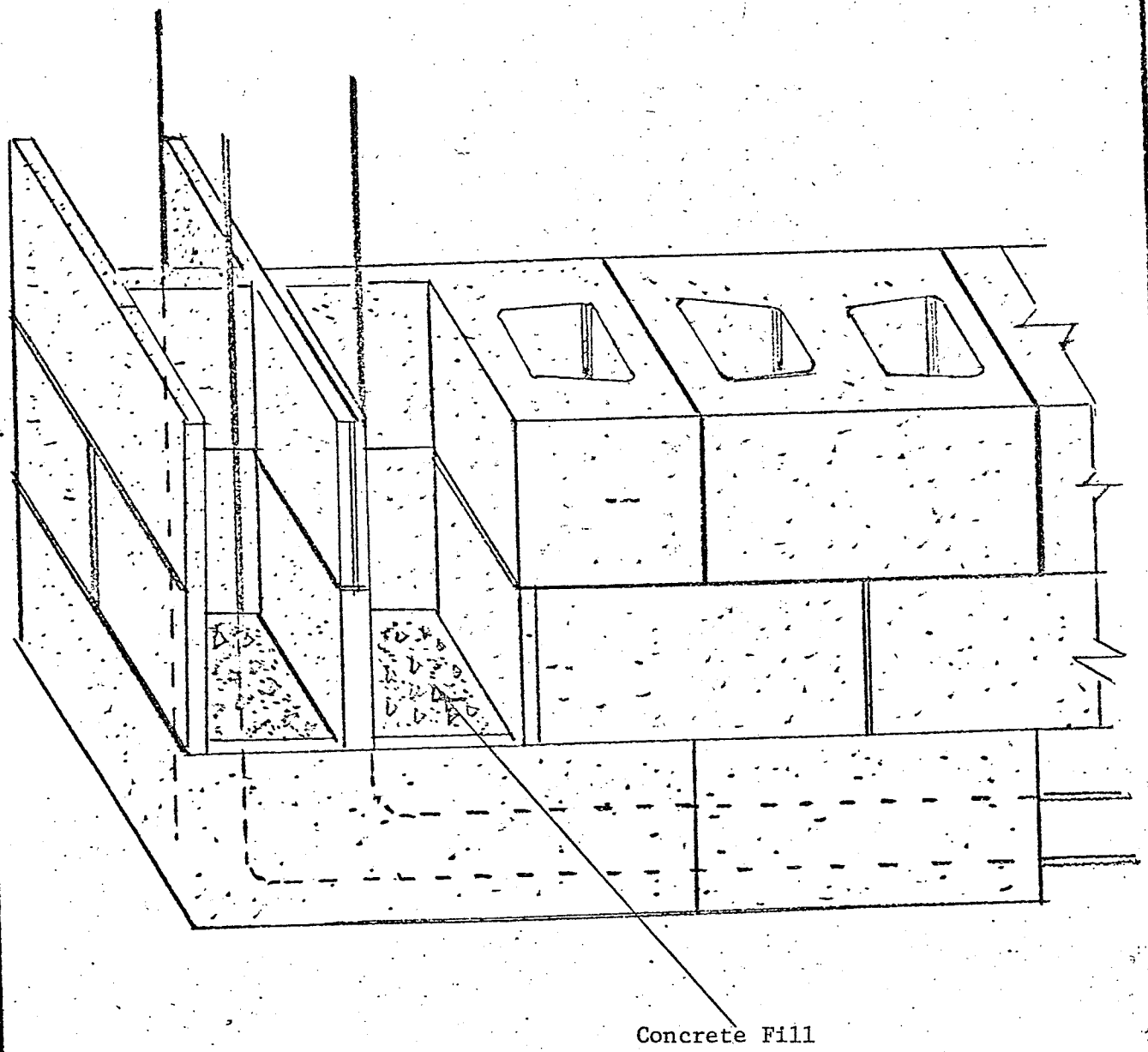
Details of First Course for Walls With Perimeter Reinforcement

Figure 11.



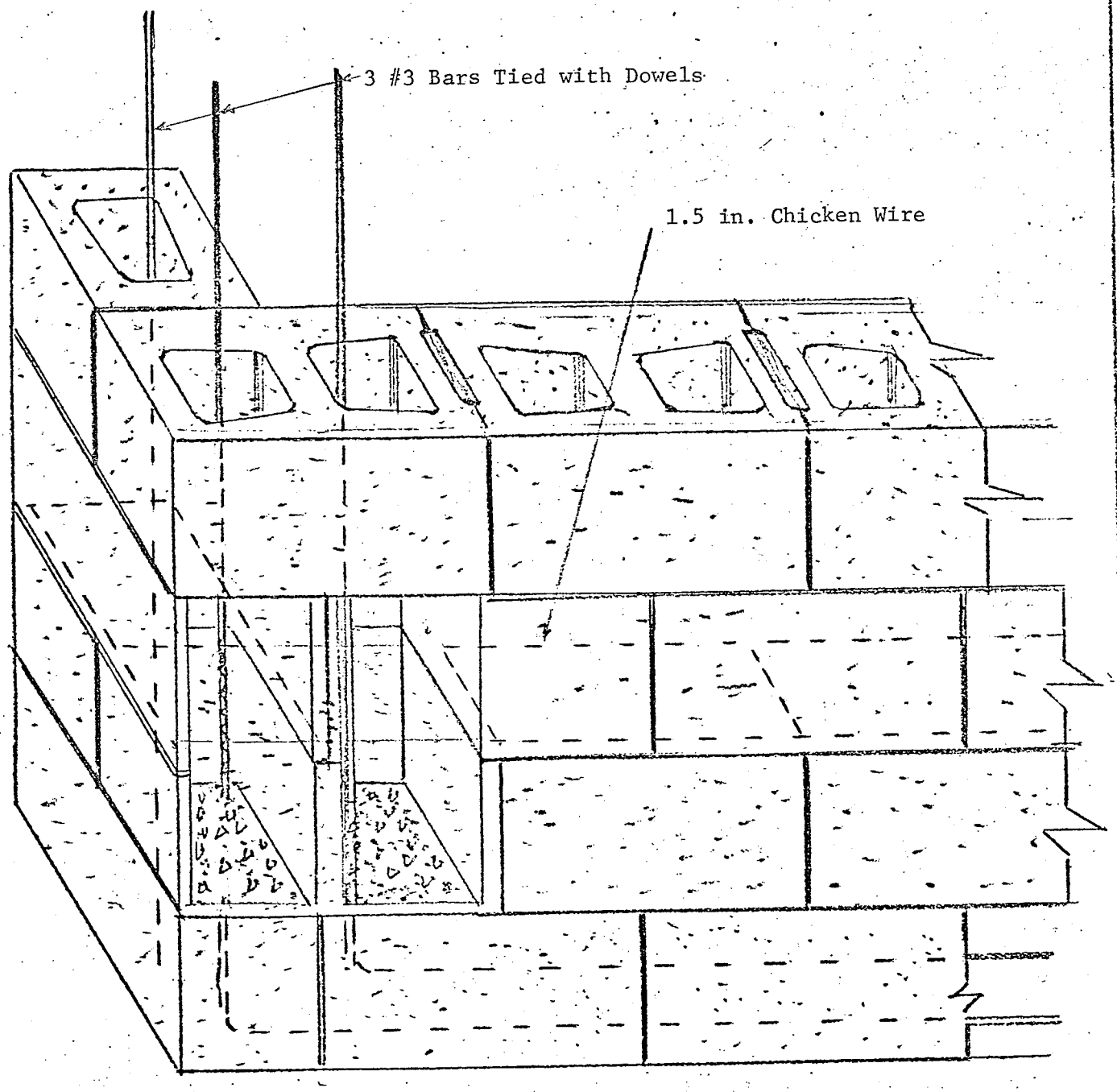
Details of First and Second Course
For Walls with Perimeter Reinforcement

Figure 12.



Details of Three Courses for Walls With Perimeter Reinforcement

Figure 13



Details of Four Courses for Walls
With Perimeter Reinforcement

Figure 14.

Figure 15 - Showing the First Course of Wall with Perimeter Reinforcement.

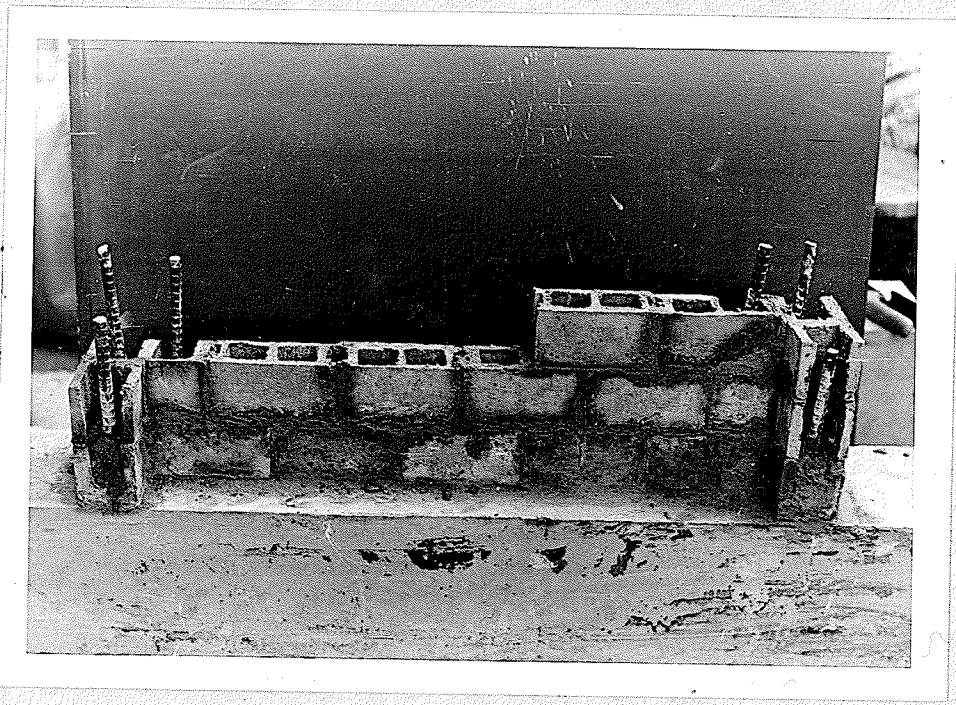


Figure 16 - Showing the Corner Details for Walls With Perimeter Reinforcement.

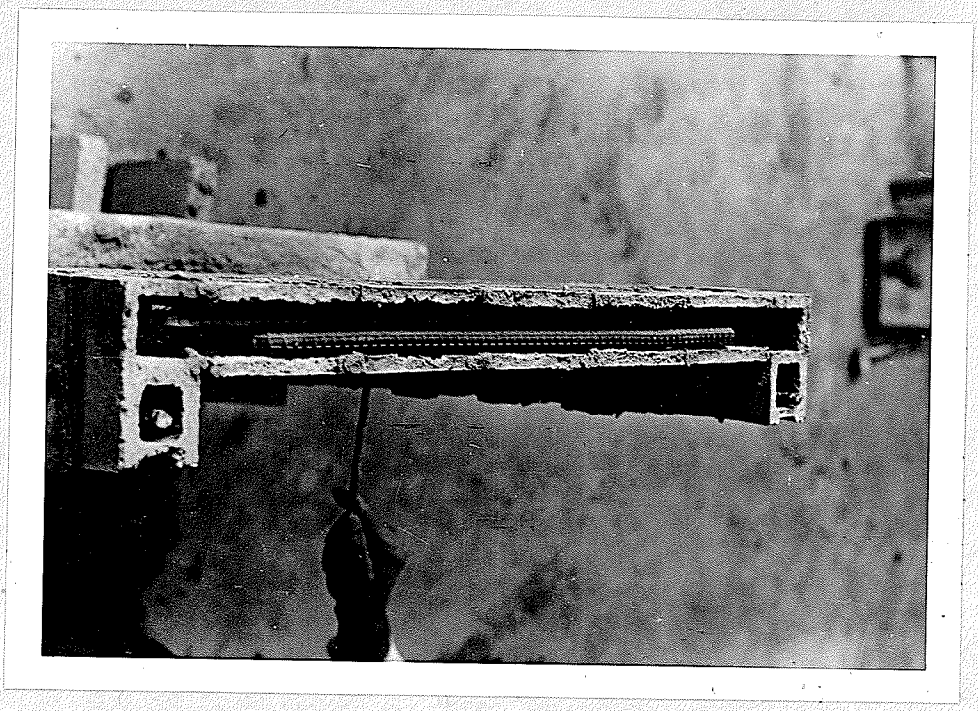


Figure 17 - Showing the Top and Flange Reinforcement Before Filling the Cores with Concrete.

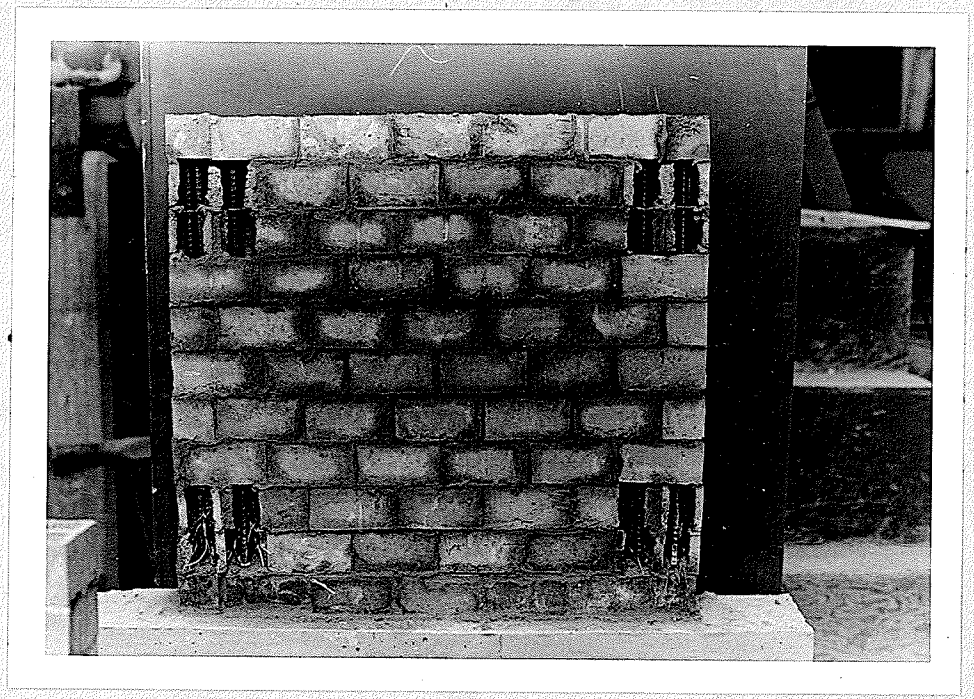


Figure 18 - Showing the Reinforcement and Corner Opening Before Filling the Core with Concrete

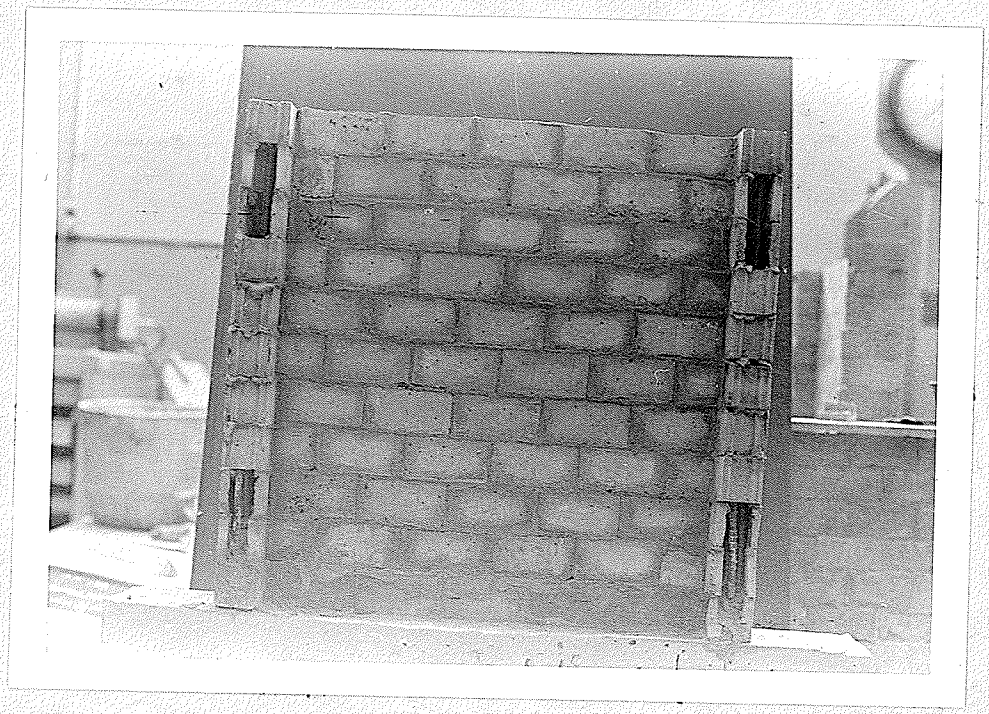
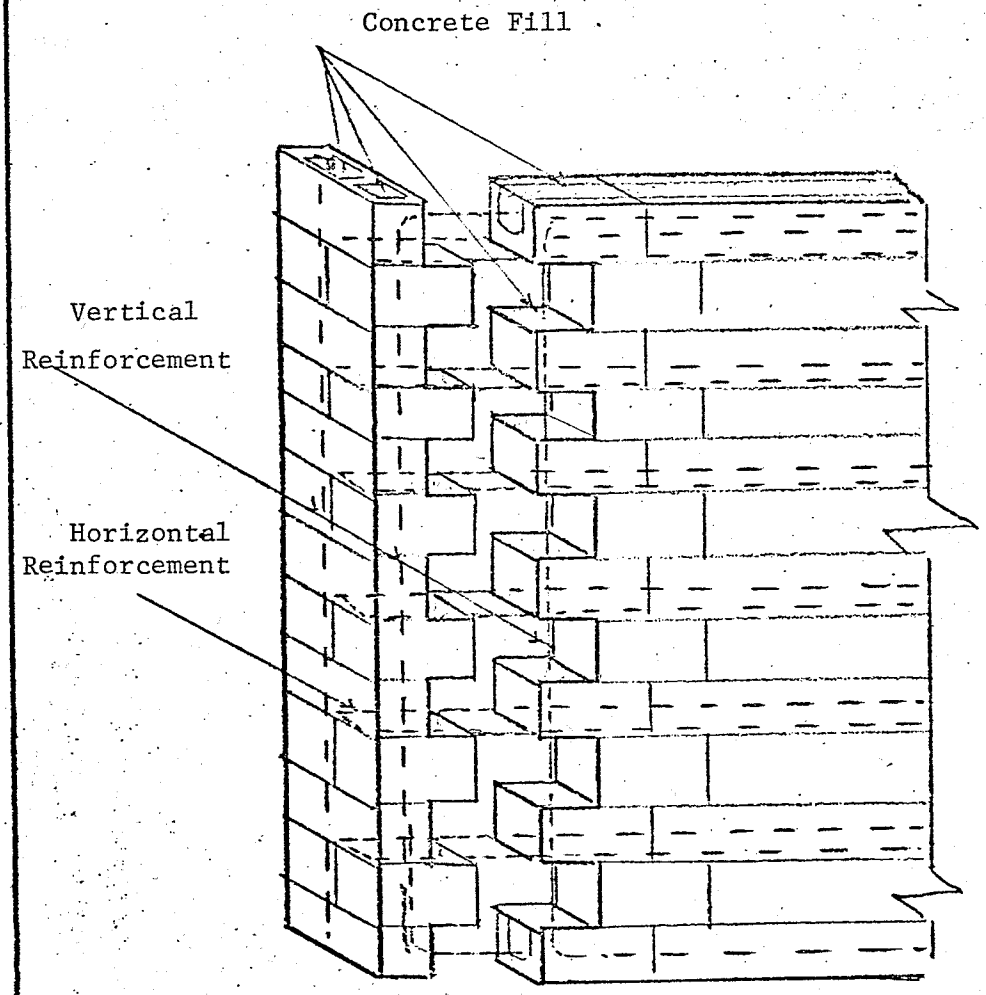


Figure 19 - Showing the Corner Openings on the Flanges



Details of Web and Flange Junction

Figure 20.

CHAPTER IV

METHOD OF TESTING AND INSTRUMENTATION

All the walls were tested after two weeks from the date of mortaring. Two 20,000-lb. hydraulic jacks provided the vertical and horizontal loads, and they were arranged so that the vertical loading would not restrain horizontal movement. The vertical loading jack applied its load through a roller and distributing beam system as shown in Figure 21. Load was applied to the upper distributing beam through 2-inch x 3-inch plate. The load was then transferred from this beam to the lower distributing beam through a block and roller system, as shown in Figure 21, and then to the 2-inch x 3/4 inch capping plate through three sets of 0.5 inch diameter hardened steel roller nests. The capping plate was seated with plaster of paris. The horizontal precalibrated loading cell applied load through a 1.5-inch diameter hemispherical ball bearing on an indented steel plate which was seated with plaster of paris against the web at the vertical windward end of the wall. For measuring the deflection in each direction, a total of 8 dial gauges were placed against the flanges in the "Z" direction, on the centre line of the web in the "Z" direction and on the web at the vertical end of the wall opposite the static racking load in the "X" direction. (See Figure 23 - 28). All gauges were supported by independent stands had a range of 1.00 inch with increments of 0.001".

Vertical load was increased from zero in 1000-lb. increments until a total of 6000-lb. was reached. This represented an average unit stress of 200 psi and 135 psi for walls without and with perimeter reinforcement, respectively. Then the static racking load increased from zero in 400-lb. increments until failure. During the application of the racking load an attempt was made to keep the vertical precompressive load constant until failure.

The walls were white washed in order to make cracking more visible. The order in which cracks appeared was shown within circles adjacent to the cracks, and the loads at various points on each crack was indicated on the crack lines. The loading frame, test set up, and measuring equipment are shown in Figures 21 and 22.

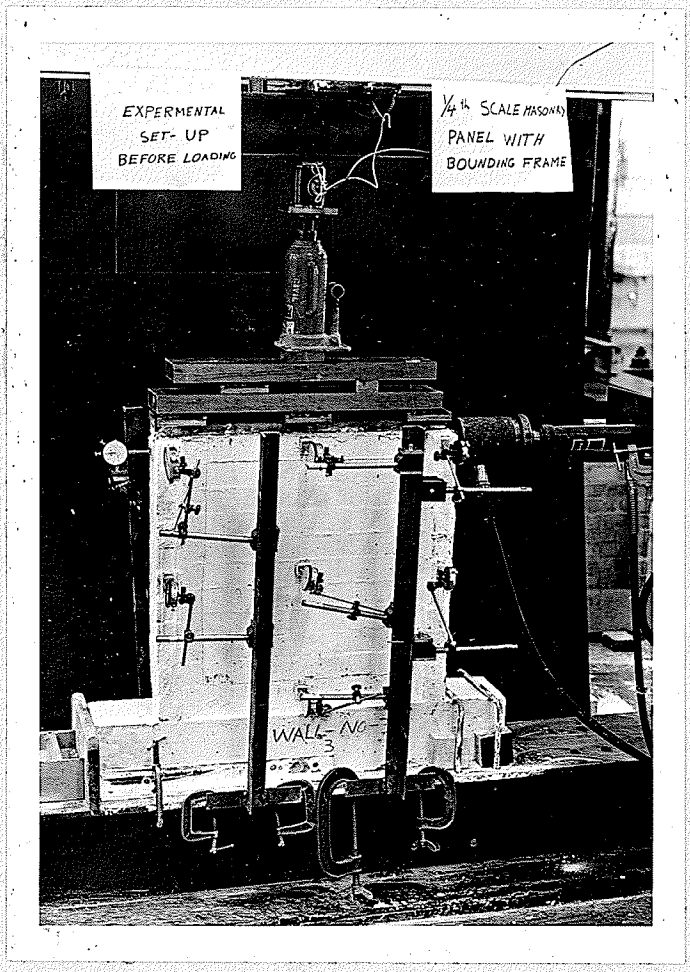


Figure 21 - Showing the Experimental Set-up Before Loading the Wall.

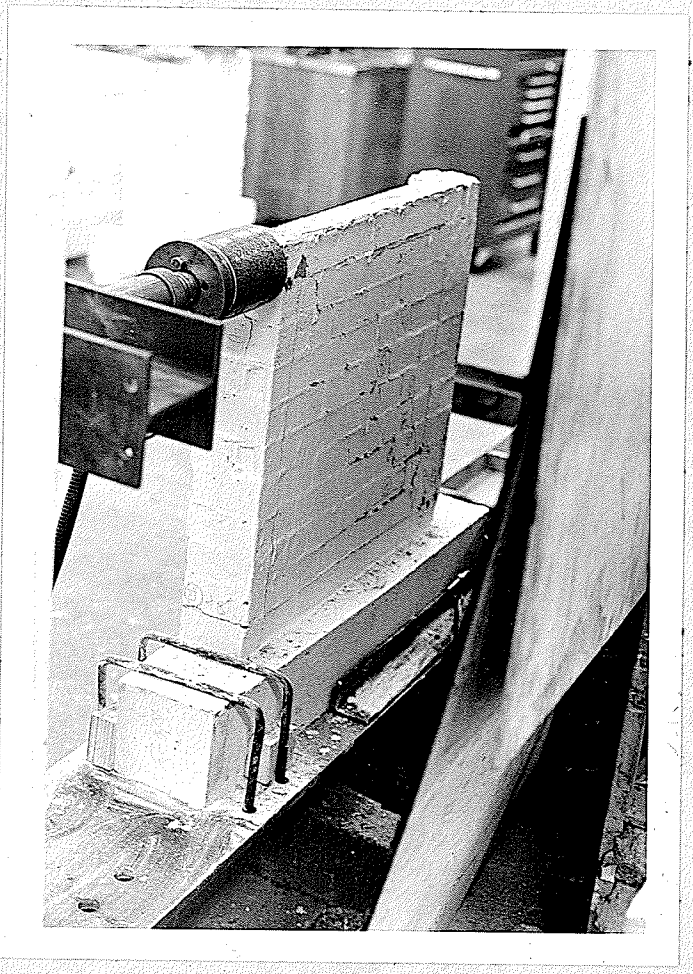


Figure 22 - Showing the Position of Racking Load.

CHAPTER V

TEST RESULTS

During the precompressing, considerable lateral deflection was observed to take place in Walls No. 1,2,3,4 and 5. At first it was difficult to establish the cause of the deflection, but later, just before testing Wall No. 6, it was found that the bottom face of the concrete base was deflected upward approximately 0.0625 inches at the toe and the deflection decreased toward the center of the concrete base. It was decided to put a shim of 0.0625-inch thick steel sheet in the gap between the concrete base and rigid steel base before loading. In this case it was noticed that the lateral deflection during the precompressing had been eliminated. The lack of flatness in concrete bases was due to curvature in the bottom of the steel forms. Also deflections in the flanges and webs of all walls were noticed during testing probably due to a slight off-center arrangement of the vertical load and the eccentricity in "Z" direction. (Figure 1 & 2). There was no cracking during the precompressing of the shear wall structures.

The first crack appeared after the static racking load was applied. The cracks which had just become visible to the naked eye were considered as "first" cracks. In general, the observed cracks were at vertical and horizontal mortar joints and in some walls many cracks appeared in the region of the applied racking load. In all

cases the cracks were almost identical in both faces of the walls. After failure it was observed that the concrete fill had a very poor bond with the precast concrete blocks, and also that the overlap-spliced perimeter steel bars had shifted outward at the corners and had caused splitting of the concrete fill. During testing no attempt had been made to restrain the wall from bending sideways, twisting or warping. The test data as obtained are presented in Table VII to XII. The curves for lateral racking load versus the deflection due to only lateral racking load for the walls without and with perimeter reinforcement are shown in Figures 29 and 30 respectively, also, for clarity the results are summarized with the help of load-deflection curves, diagrams of exaggerated deflected shapes of the walls, and photographs of some failures in Figures 23 to 36.

It should be noted that the walls were tested in order 1,2,3,4,5,6, but the tables are shown in order of groups (1,5,6) and (2,3,4) to show similar structural characteristics of the walls without and with perimeter reinforcement respectively.

Table VII

Test Data for Wall No. 1 Without Perimeter Reinforcement

Loading System		Deflection for only Precompressive Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
	0.0	0	0	0	0	0	0	0	0	
Precompressive Load (Kips)	1.0	29.5	22	-56	-35	-20	37	28	41	
	2.0	32.3	20	-69	-40	-21	45	30	46	
	3.0	27.1	16	-75	-42	-22	45	30	50	
	4.0	21.4	11	-81	-45	-22	46	30	53	
	5.0	17.8	5	-81	-49	-23	47	29	59	
	6.0	7.9	0	-93	-51	-24	48	29	62	
Lateral Racking Load (Kips)		Deflection for Combined Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
	0.4	1.6	- 4	-92	-49	-24	48	29	66	
	0.8	-9.2	-10	-90	-50	-24	49	31	73	
	1.2	-10.7	-12	-87	-52	-23	48	31	82	
	1.6	-29.0	-24	-83	-34	-22	46	31	90	
	2.0	-46.6	-34	-70	-38	-20	40	27	101	First Crack
	2.4	-70	-49	-49	-27	-15	25	18	118	
	2.8	-90	-66	-40	-19	-15	12	11	140	
	3.2	-133.5	-88	86	97	-10	0	4	160	
	3.6									ulti-mate
	4.0									
	4.4									
	4.6									

Note: G = Gauge As shown in Figures 23 - 28

Table VIII

Test Data for Wall No. 5 Without Perimeter Reinforcement

Loading System		Deflection for Only Precompressive Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
Precompressive Load (Kips)	0.0	0	0	0	0	0	0	0	0	
	1.0	7.5	-11	- 5	10	6	8	-4	40	
	2.0	26.7	0	-20	0	2	22	5	59	
	3.0	38.5	8	-33	-9	-2	30	11	69	
	4.0	38.8	8	-35	-10	-2	31	11	71	
	5.0	38.8	8	-36	-10	-2	31	12	73	
	6.0	38.8	8	-37	-10	-2	32	12	75	
Lateral Racking Load (Kips)		Deflection for Combined Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
	0.4	38.9	-8	-38	-11	-2	32	12	78	
	0.8	38.9	-8	-40	-11	-2	32	12	80	
	1.2	38.9	-8	-41	-12	-4	32	12	84	
	1.6	38.9	-8	-41	-12	-4	32	12	87	Fir. Cr.
	2.0	38.9	-6	-41	-12	-4	33	12	92	
	2.4	38.9	-6	-41	-13	-4	33	12	97	
	2.8	38	-0	-38	-11	-5	32	31	128	
	3.2	23.5	-5	-32	- 9	-5	23	0.7	154	
	3.6									ult-mate
	4.0									
	4.4									
	4.6									

Note : G = Gauge As shown in Figures 23 - 28

Table IX

Test Data for Wall No. 6 Without Perimeter Reinforcement

Loading System		Deflection for Only Precompressive Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
Precompressive Load (Kips)	0.0	0	0	0	0	0	0	0	0	
	1.0	0.7	1	-2	2	0	- 3	- 3	1	
	2.0	0.2	1	-1	4	2	- 8	- 5	2	
	3.0	-0.7	1	0	6	2	-13	- 7	4	
	4.0	-1.5	1	0	8	2	-18	-10	4	
	5.0	-2.5	1	0	+10	3	-21	-11	4	
	6.0	-3.0	1	12	11	3	-25	-13	4	
Lateral Racking Load (Kips)		Deflection For Combined Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
	0.4	-3.1	1	12	11	2	-25	-13	5	
	0.8	-3.0	2	0	10	3	-23	-11	12	
	1.2	-3.1	2	0	9	2	-18	- 9	12	
	1.6	-3.5	2	2	8	2	-14	- 7	12	First Crack
	2.0	-5.5	1	-2	7	1	-11	- 5	16	
	2.4	-6.5	-9	0	6	0	- 6	- 3	23	
	2.8	-10.8	-6	3	4	-1	1	1	35	
	3.2	-26.5	-19	+8	6	-1	5	3	53	
	3.6									Ulti-mate
	4.0									
	4.4									
	4.6									

Note : G = Gauge As shown in Figures 23 - 28

Table X

Test Data for Wall No. 2 With Perimeter Reinforcement

Loading System		Deflection for Only Precompressive Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
Precompressive Load (Kips)	0.0	0	0	0	0	0	0	0	0	
	1.0	-0.5	-1	8	4	2	-1	-4	4	
	2.0	-0.5	-1	11	5	2	-1	-5	8	
	3.0	-0.9	-2	12	7	2	-1	-5	10	
	4.0	-1.2	-2	13	7	2	-1	-5	13	
	5.0	17.8	-3	16	8	3	-1.3	-7	15	
	6.0	16.9	-6	18	10	3	-1.5	-8	18	
Lateral Racking Load (Kips)		Deflection for Combined Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
	0.4	16.5	- 4	13	7	2	-1	-5	24	
	0.8	16.1	- 6	3	0	0	0	3	37	
	1.2	16.0	- 4	- 5	- 5	- 3	0.011	8	48	
	1.6	14.4	- 4	-11	- 9	- 6	0.018	11	60	First Crack
	2.0	11	- 5	-17	-14	- 9	.023	16	71	
	2.4	9.1	- 6	-21	-17	-10	0.29	17	82	
	2.8	9.1	- 9	-22	-21	-12	0.04	23	101	
	3.2	9.0	-10	-33	-26	-12	.061	34	117	
	3.6	1.2	-36	-22	-26	-14	0.093	50	147	
	4.0									Ulti-mate
	4.4									
	4.6									

Note: G = Gauge As shown in Figures 23 - 28

Table XI

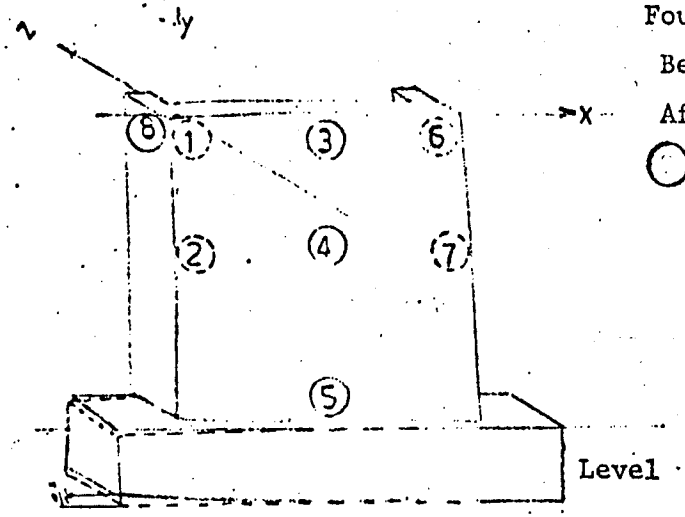
Test Data for Wall No. 3 With Perimeter Reinforcement

Loading System		Deflection for Only Precompressive Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
Precompressive Load (Kips)	0.0	0	0	0	0	0	0	- 0	0	
	1.0	7.7	-1	+8	4	+3	- 7	- 3	8	
	2.0	8.0	+1	9	6	+3	- 9	- 4	15	
	3.0	15.2	6	14	8	+4	-14	- 7	22	
	4.0	15.2	7	17	9	+4	-16	- 7	29	
	5.0	16.4	13	23	10	-5	-22	-11	35	
	6.0	16.4	8	27	15	-6	-25	-13	38	
Lateral Racking Load (Kips)		Deflection for Combined Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7G	G8	
	0.4	16.2	9	28	15	-6	-25	-13	42	
	0.8	15.7	9	28	16	-5	-24	-12	49	
	1.2	14.5	9	28	16	-5	-23	-10	58	
	1.6	18.7	11	28	16	-4	-21	- 8	66	
	2.0	17.6	12	28	15	-3	-20	- 7	75	First Crack
	2.4	13.4	10	28	13	-0	-18	- 5	85	
	2.8	-16.8	7	31	11	-4	-15	- 2	95	
	3.2	-45.0	2	46	14	-8	-15	- 2	141	
	3.6									Ulti-mate
	4.0									
	4.4.									
	4.6									
	Note: G = Gauge As shown in Figures 23 - 28									

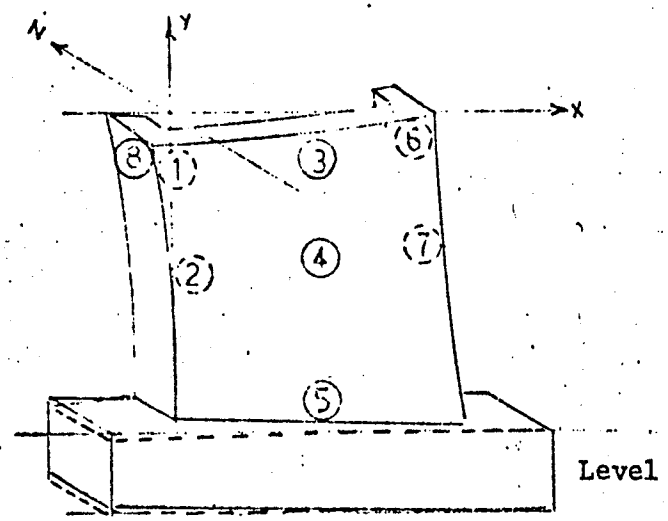
Table XII

Test Data for Wall No. 4 With Perimeter Reinforcement

Loading System		Deflection for Only Precompressive Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
Precompressive Load (Kips)	0.0	0	0	0	0	0	0	0	0	
	1.0	29.4	14	-18	-14	- 6	16	11	4	
	2.0	39.4	20	-29	-20	- 8	25	15	11	
	3.0	40.4	26	-40	-25	-12	32	20	18	
	4.0	67.5	39	-53	-35	-15	42	25	25	
	5.0	86.2	52	-69	-46	-20	54	32	33	
	6.0	94.7	65	-90	-60	-27	68	41	46	
Lateral Racking Load (Kips)		Deflection for Combined Load (inch x 10^{-3})								
		G1	G2	G3	G4	G5	G6	G7	G8	
	0.4	111.6	75	-110	-65	-34	81	48	58	
	0.8	129.8	82	-119	-79	-37	82	52	64	
	1.2	129.5	82	-123	-76	-38	91	53	70	
	1.6	129.5	82	-125	-86	-20	94	54	75	First Crack
	2.0	129.5	82	-128	-88	-40	95	54	81	
	2.4	127.5	82	-128	-88	-42	96	54	86	
	2.8	110.6	78	-130	-87	-41	98	54	96	
	3.2	103.2	73	-128	-88	-38	101	54	106	
	3.6	95.7	66	-126	-88	-41	102	53	122	
	4.0	70.5	58	-121	-86	-40	106	47	140	
	4.4	74.0	45	115	-83	-21	98	45	166	
	4.6									Ultimate
	Note: G = Gauge As shown in Figures 23 - 28									



(a) Under Precompressive Load Only



(b) Under Combined Load

Foundation Position

Before Distortion ——

After Distortion ------

○- Gage Number

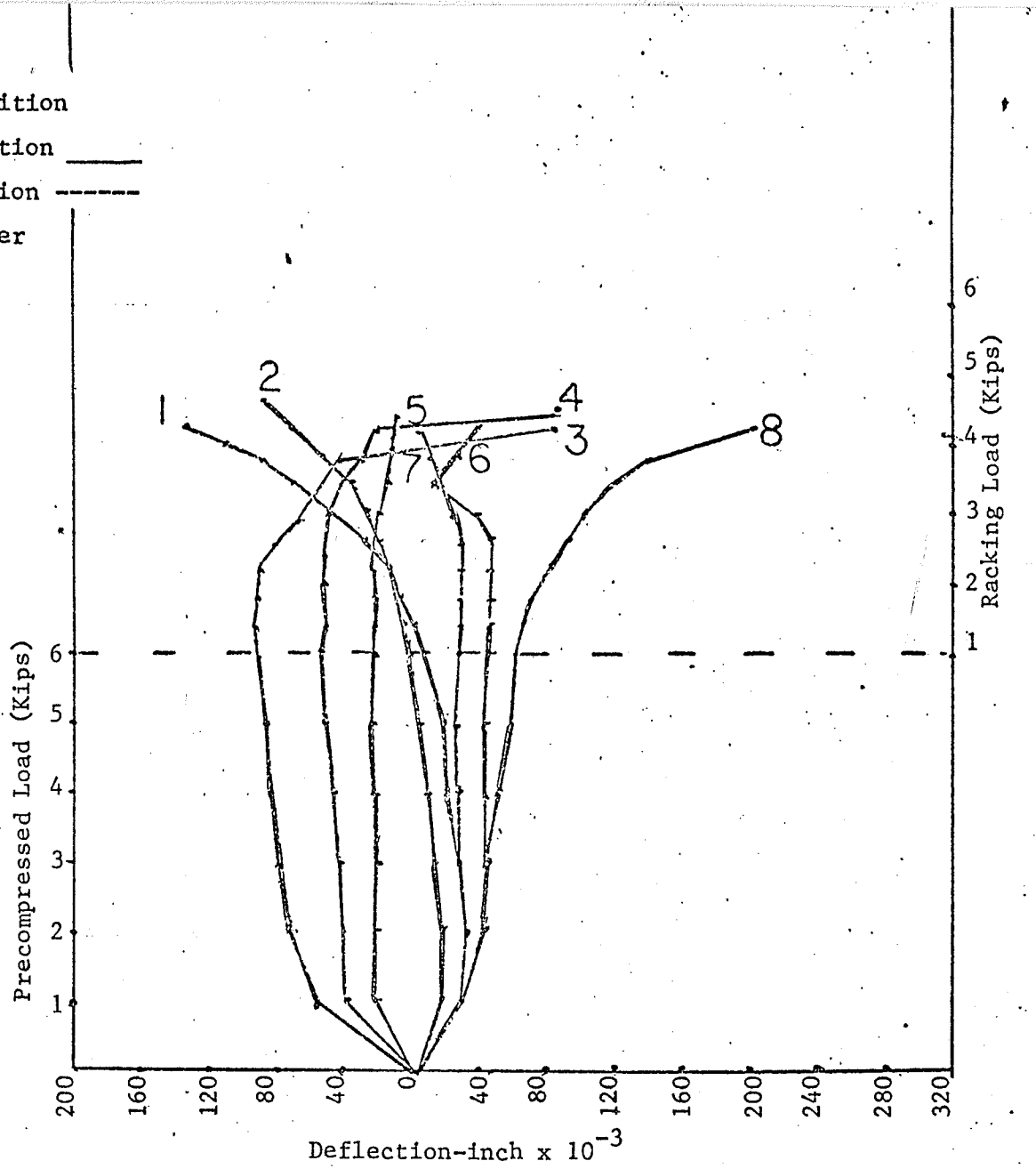


Figure 23 - Load-deflection Curves and Exaggerated Deflected and Twisted Shape (Wall No. 1)

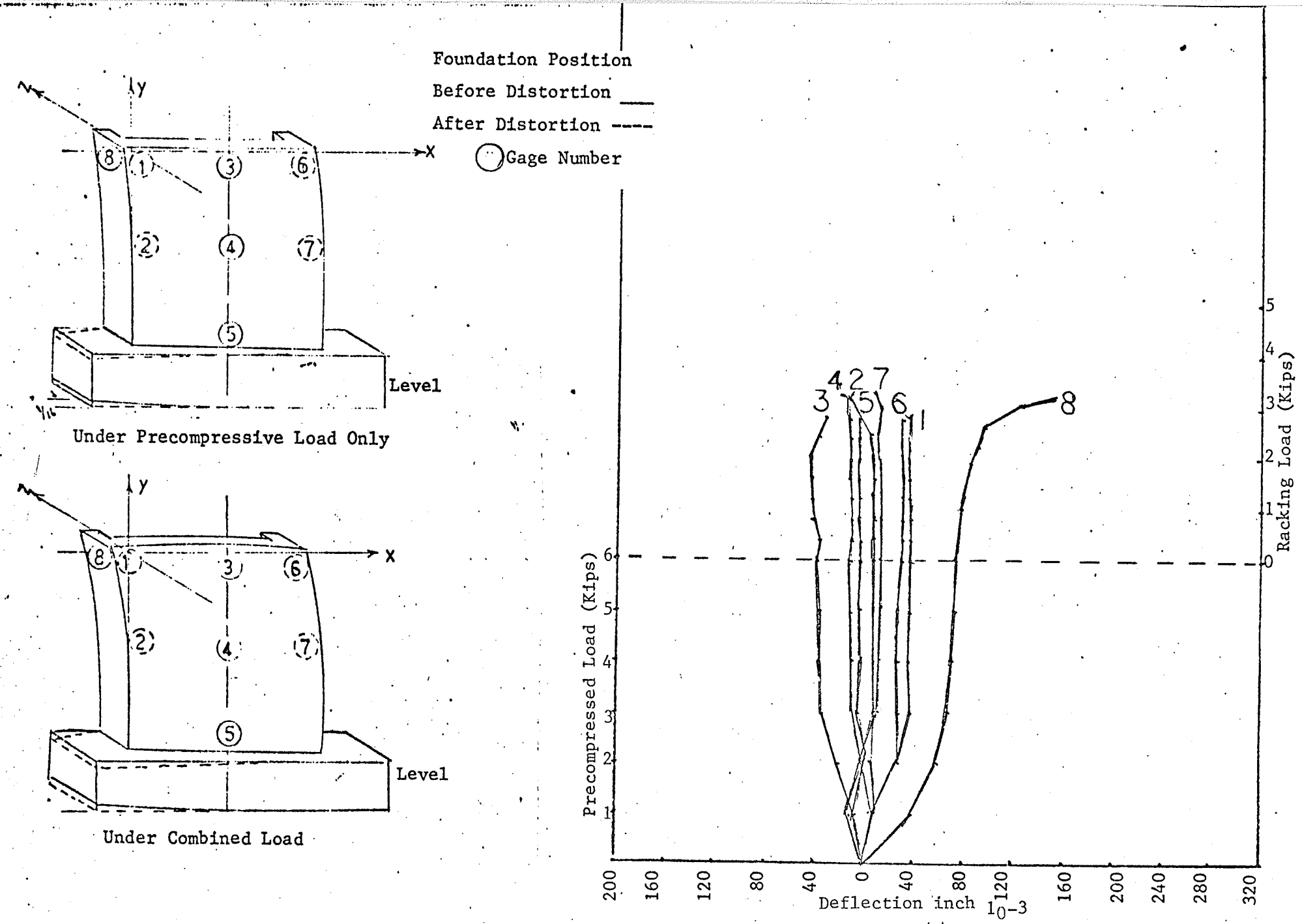
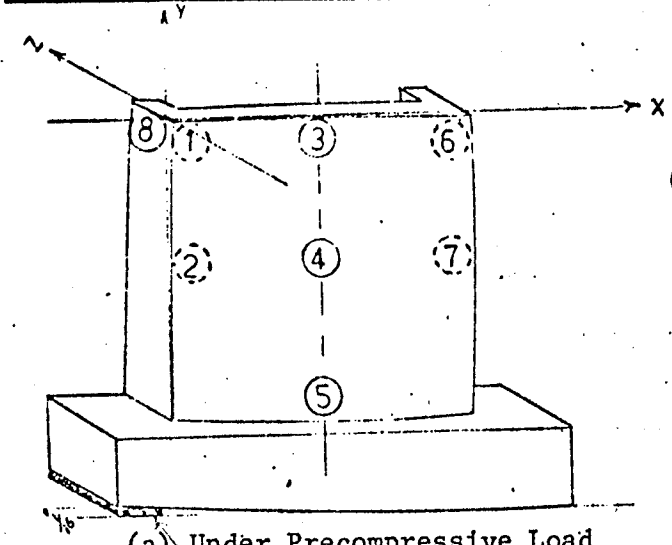
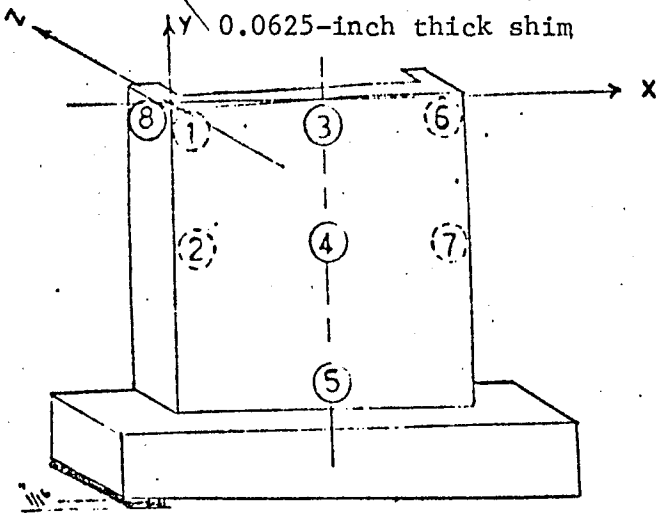


Figure 24 - Load-Deflection Curves and Exaggerated Deflection and Twisted Shapes (Wall No. 5)



(a) Under Precompressive Load

0.0625-inch thick shim

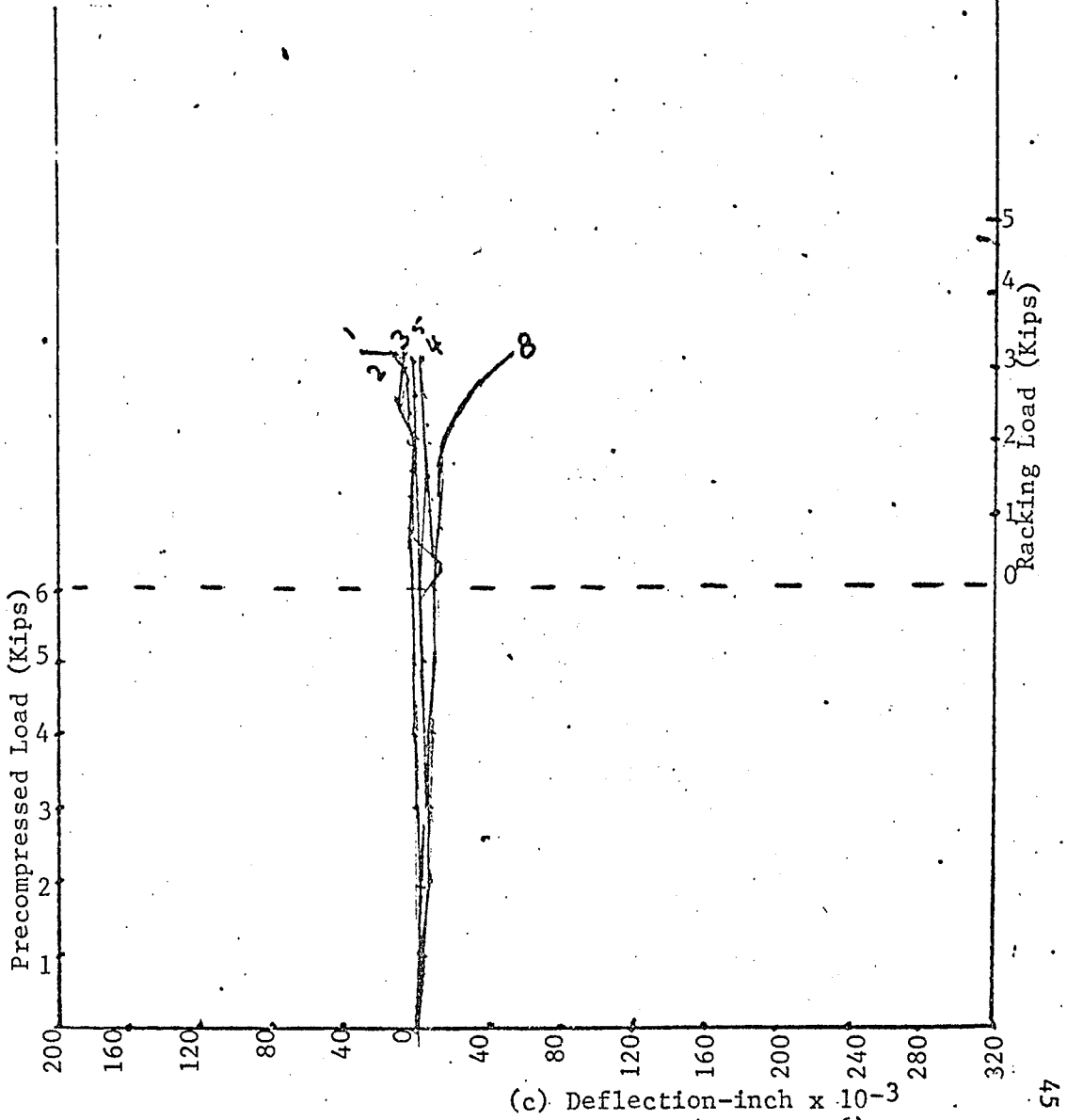


1/16" Steel Plate

(b) Under Combined Load

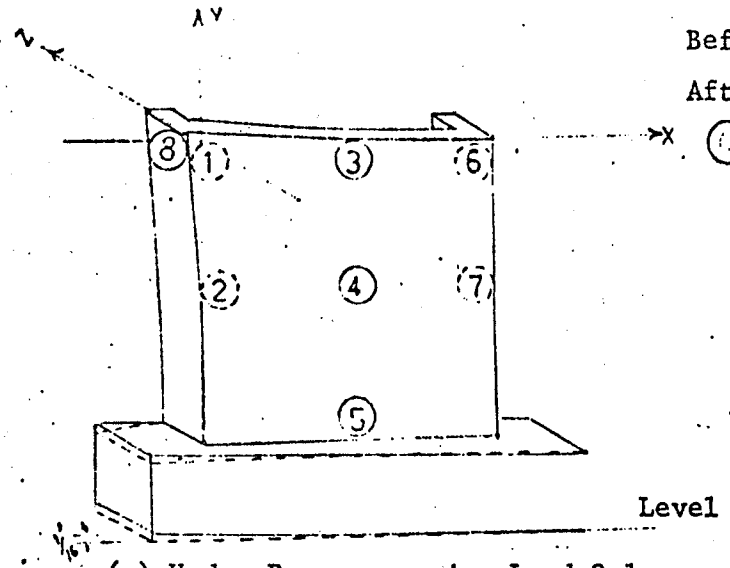
Foundation Position Before and After Loading ____

Gage Number

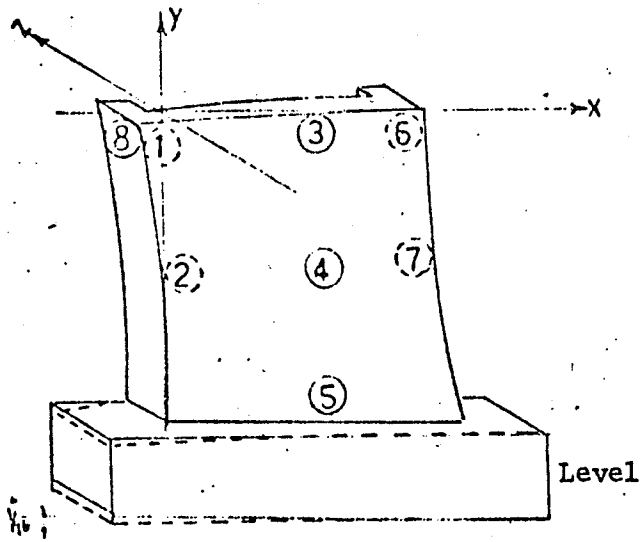


(c) Deflection-inch x 10^{-3}

Figure 25 - Load-Deflection Curves and Exaggerated Deflected and Twisted Shapes (Wall No. 6)



(a) Under Precompressive Load Only



(b) Under Combined Load

Foundation Position

Before Distortion ———

After Distortion - - - -

(n) - Gage Number

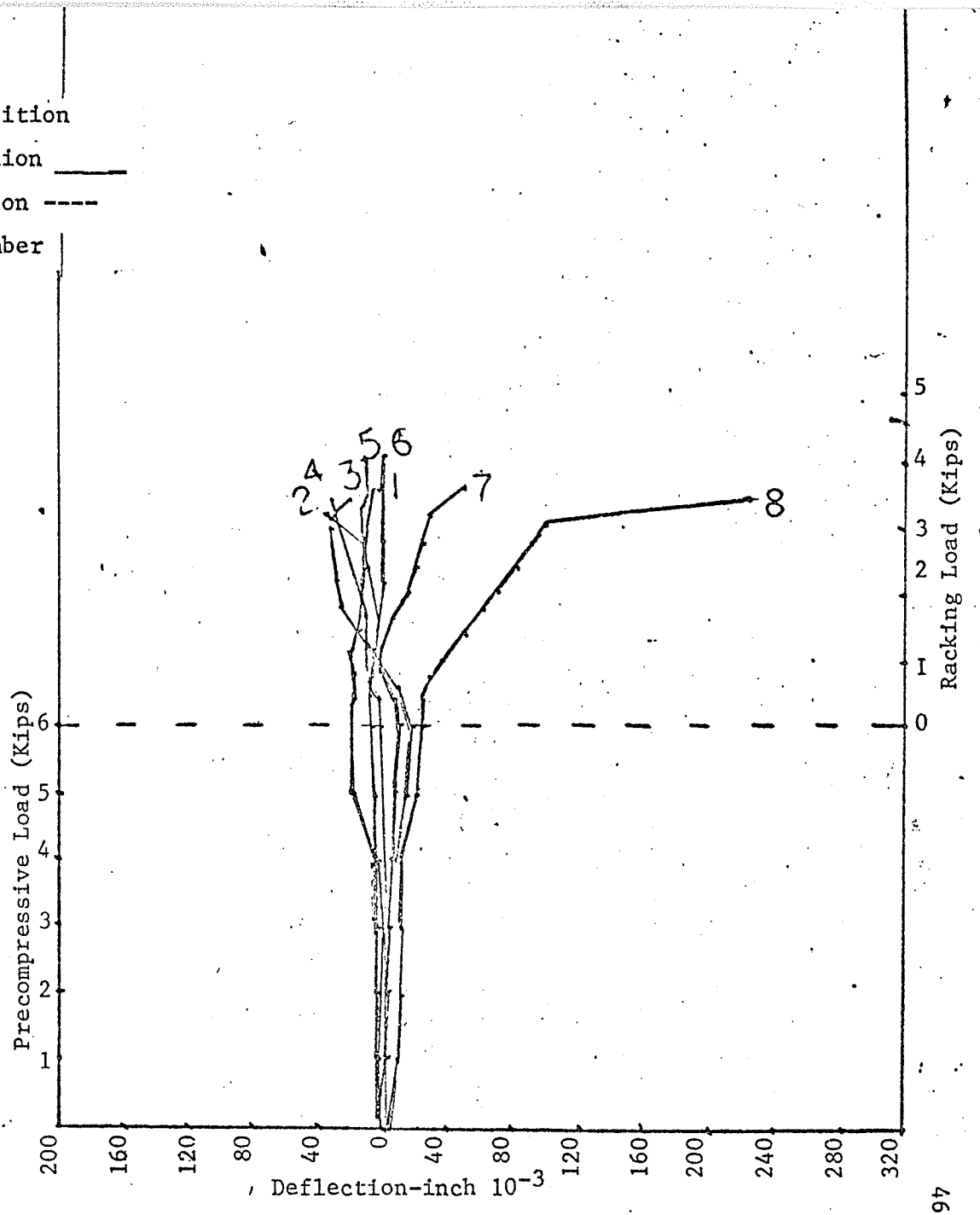
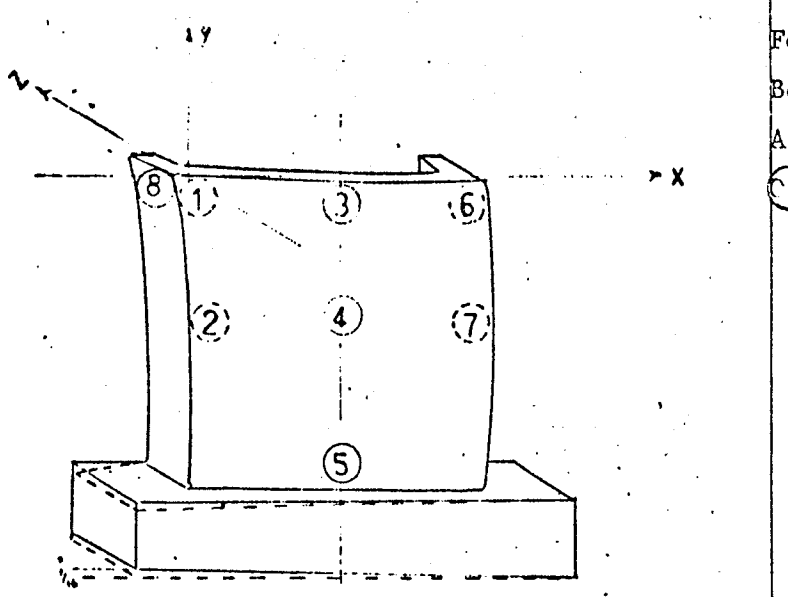
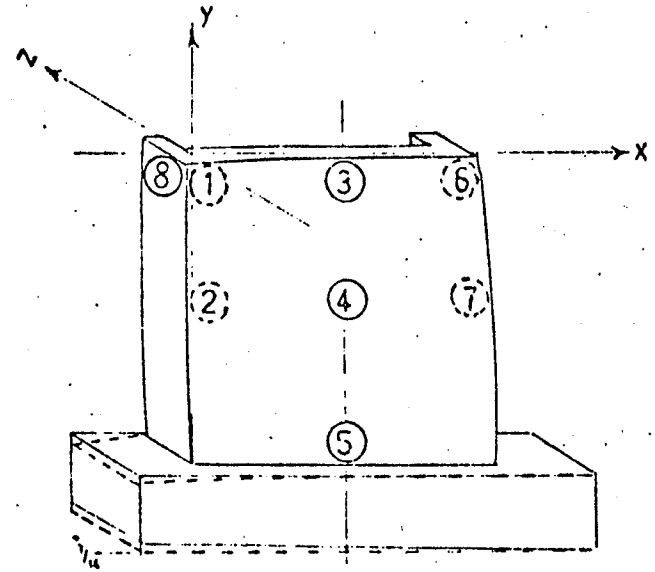


Figure 26 - Load-Deflection Curves and Exaggerated Deflected and Twisted Shape (Wall No. 2)



(a) Under Precompressive Load Only



(b) Under Combined Load

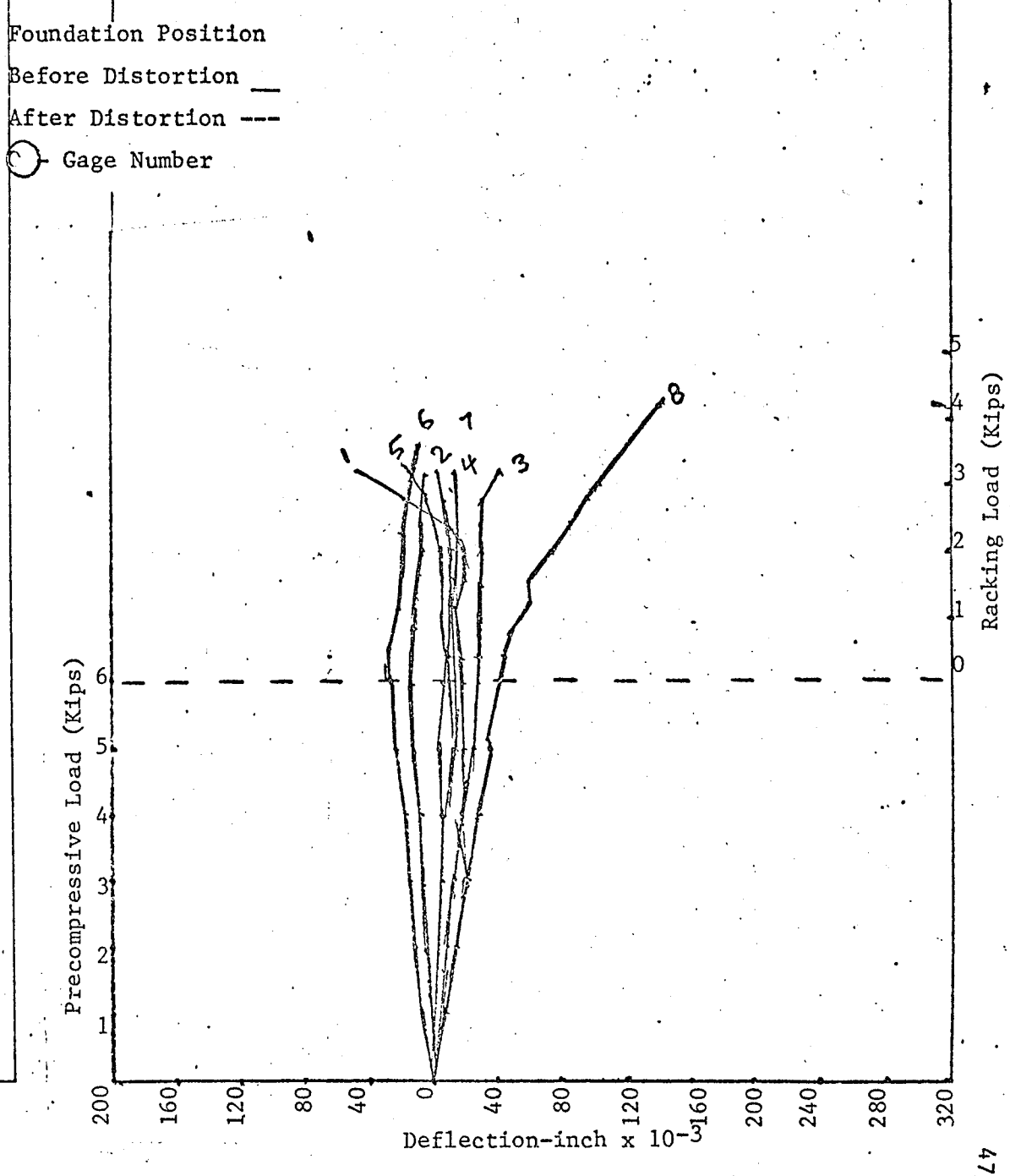


Figure 27 - Load-Deflection Curves and Exaggerated Deflected and Twisted Shapes (Wall No. 3)

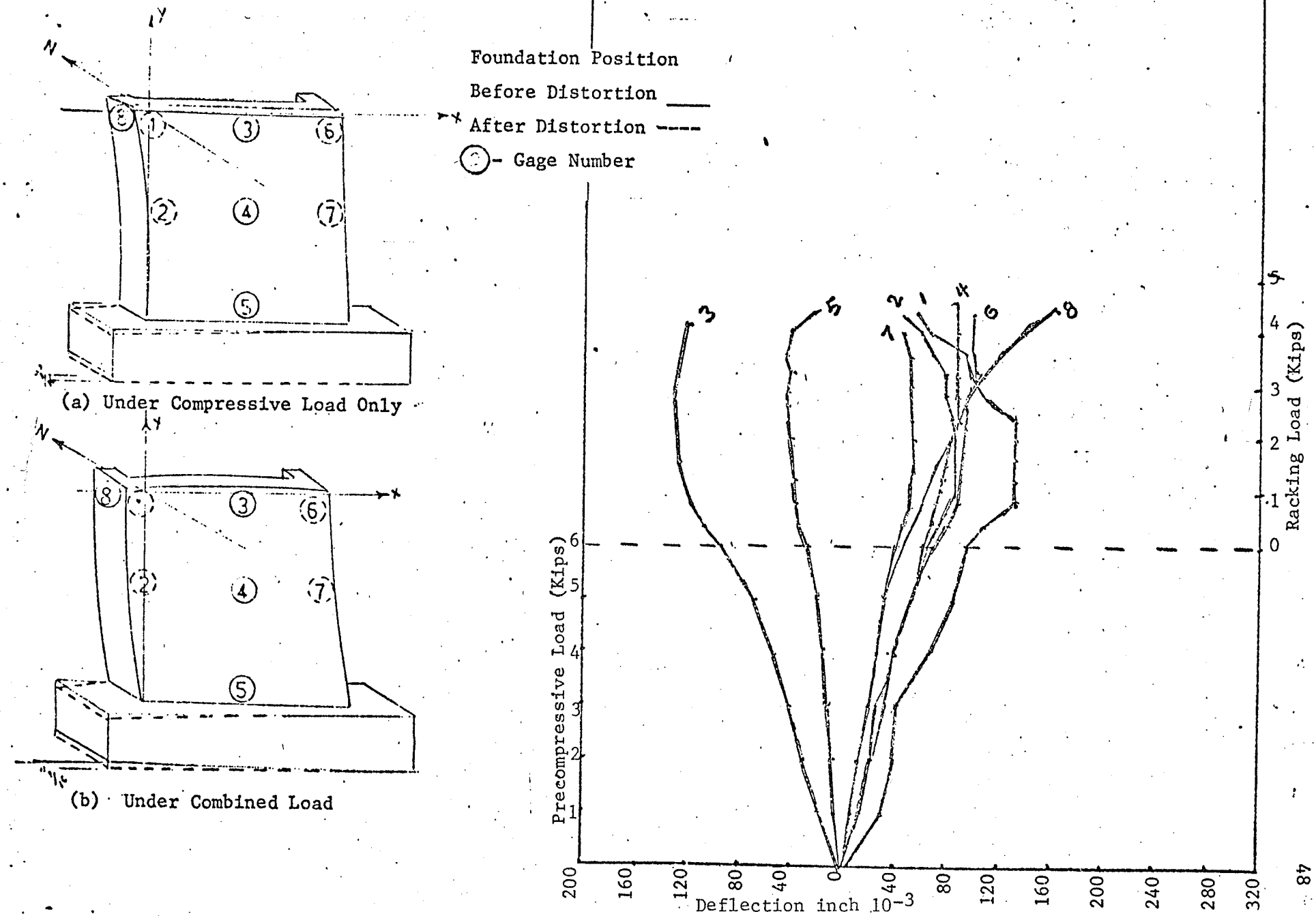


Figure 28 - Load-Deflection Curves and Exaggerated Deflected and Twisted Shapes (Wall No. 4)

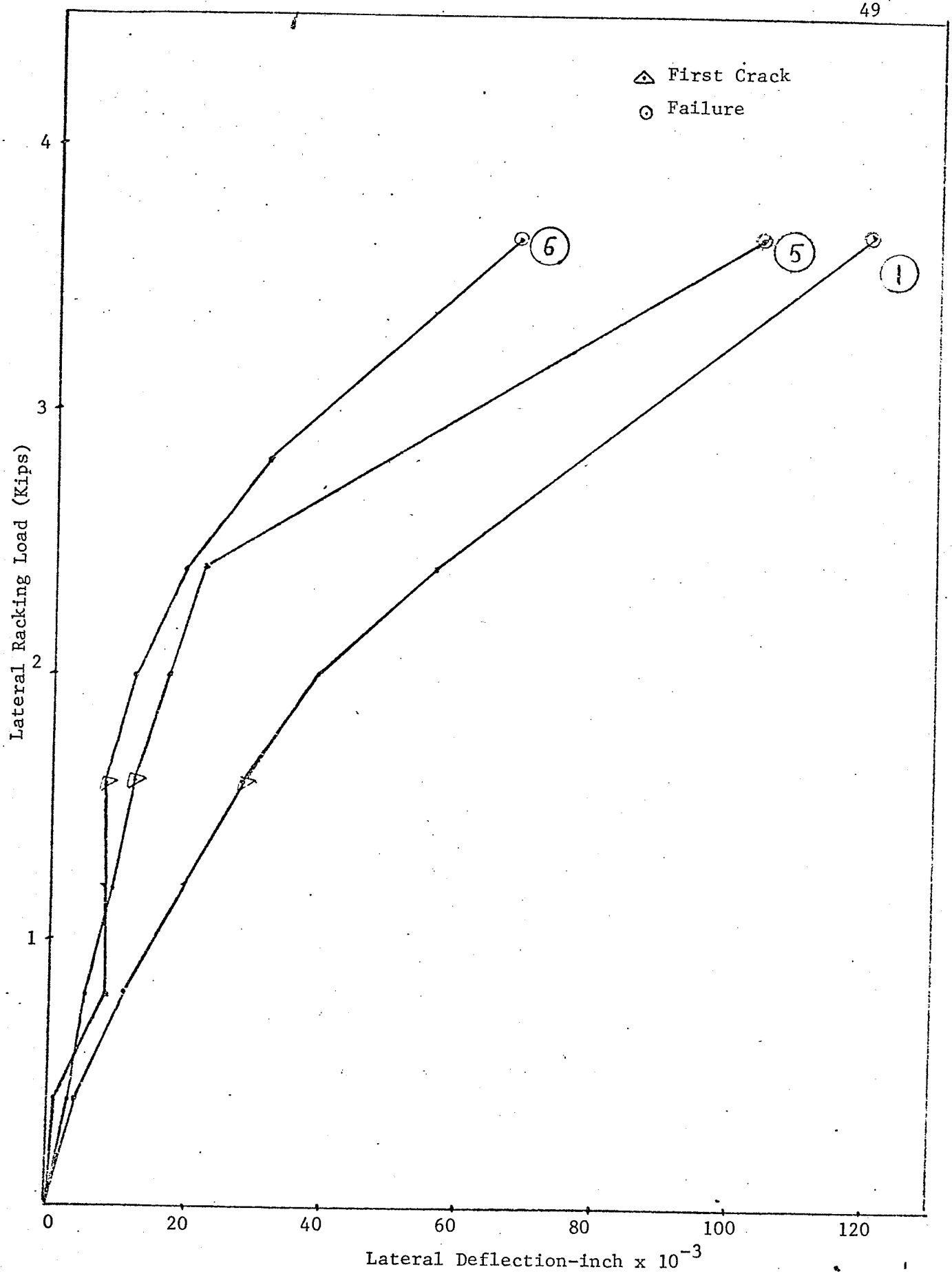


Figure 29 - Load Deflection curves for precompressed walls Nos. 1,5,6.
(Walls without perimeter reinforcement)

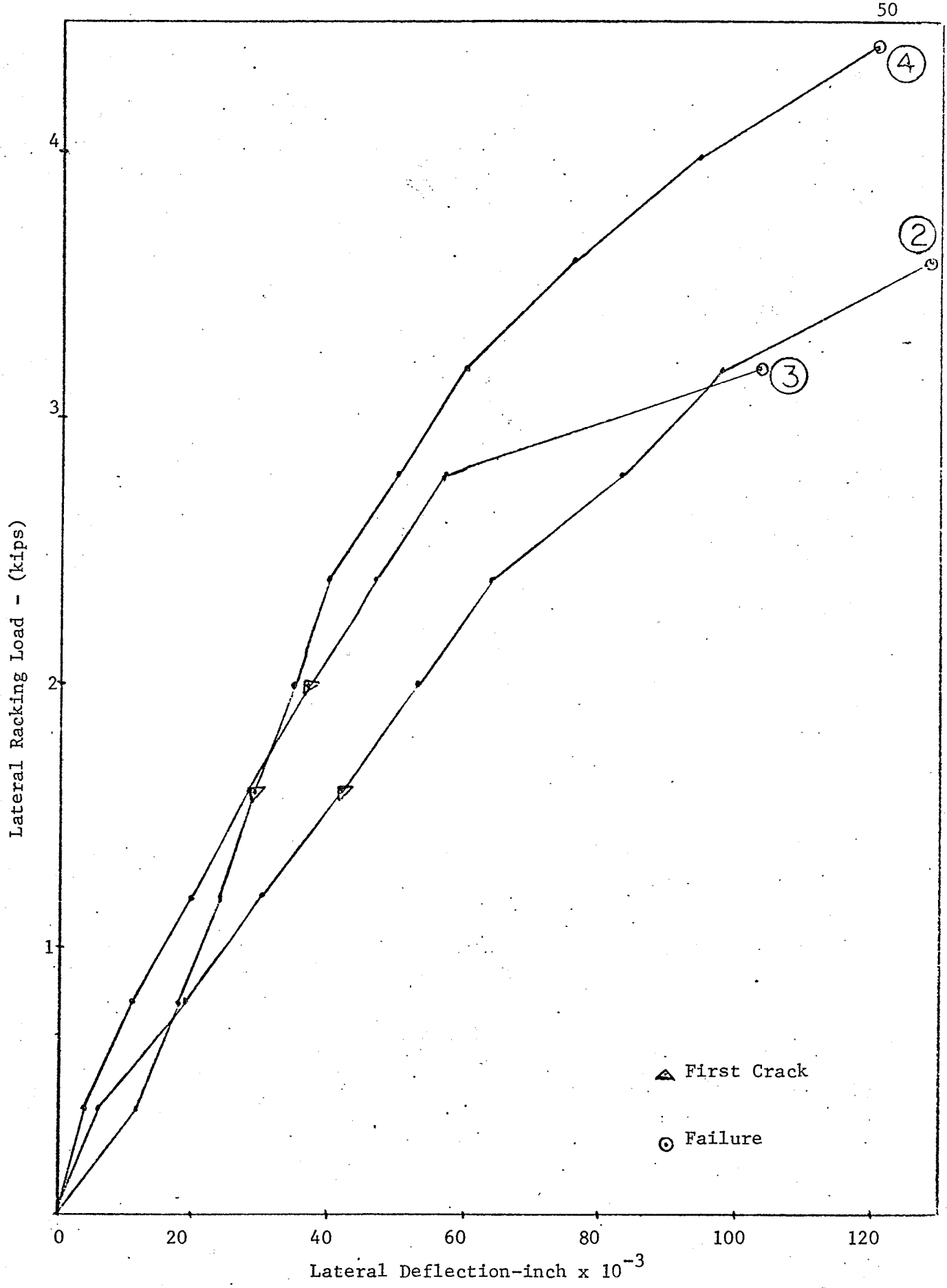


Figure 30 - Load deflection curves for precompressed walls Nos. 2,3,4
(Walls with perimeter reinforcement)

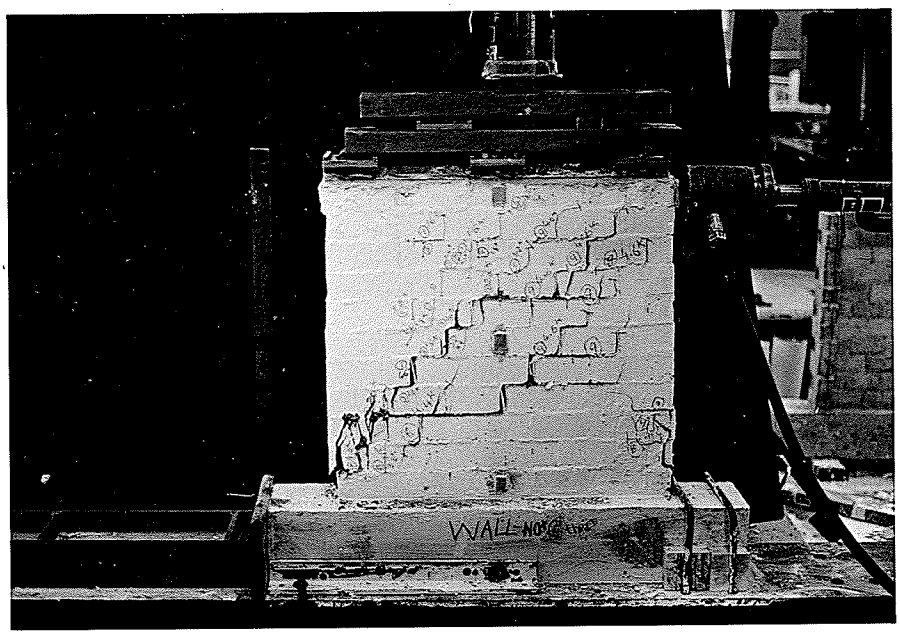


Figure 31 - Wall No. 4 Showing the Opening of the Cracks at Post Ultimate Load.

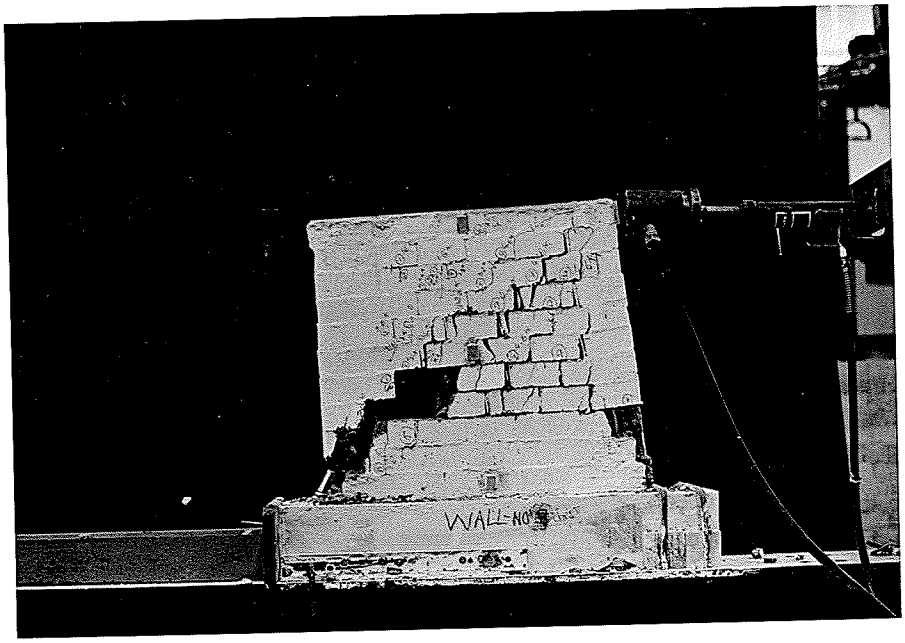


Figure 32 - Showing the "Ductility" of the Wall and Deflection of the Wall at Post Ultimate Load.

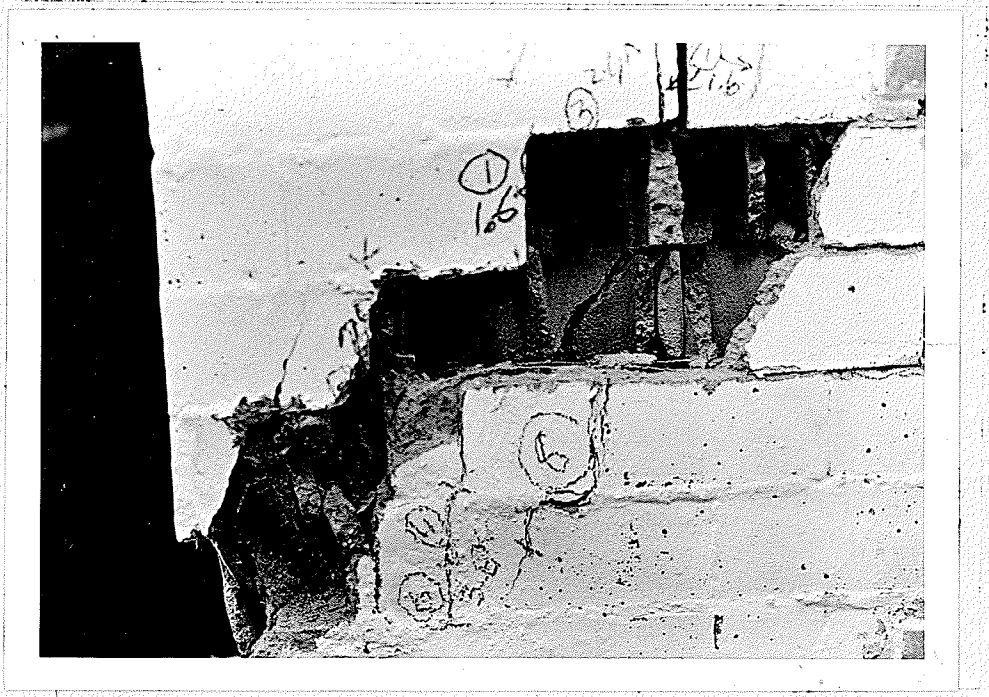


Figure 33 - Wall No. 4 - The Diagonal Cracks Through Blocks and the Bending of the Bars at the Toe

Figure 35 - Wall No. 4 Showing the Poor Bond of Concrete Fill

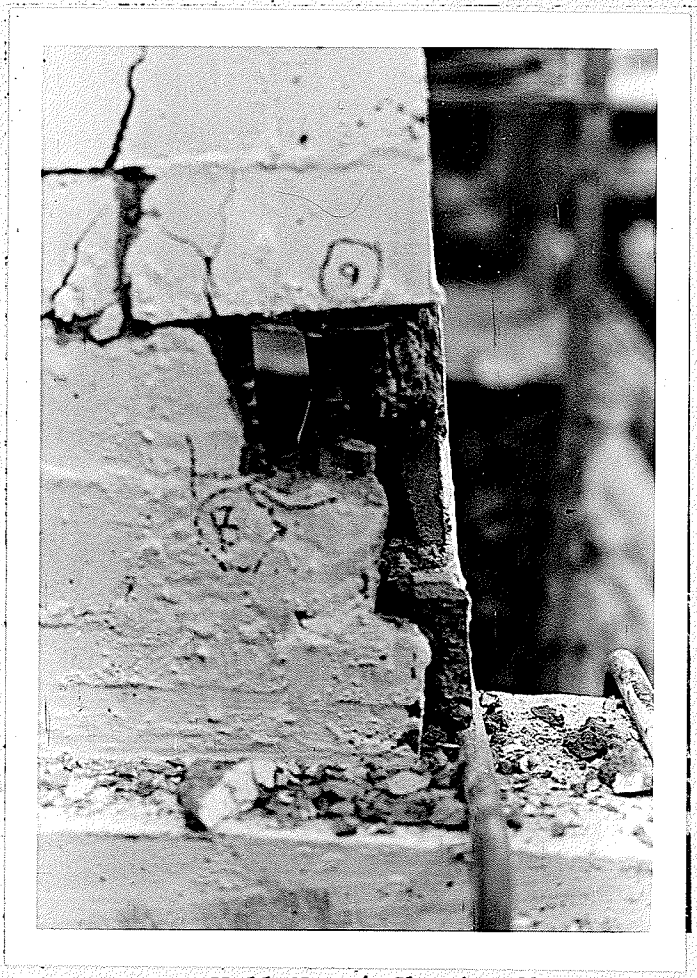


Figure 36 - Wall No. 4 Showing Movement and Separation of the overlapped Bars.

CHAPTER VI

MODES OF FAILURE

Figures 37 to 45 show the observed crack patterns. The possible modes of failure for shear walls with and without perimeter reinforcement are discussed below:

Walls Without Perimeter Reinforcement

The following modes of failure were observed during the tests of walls without perimeter reinforcement (Figures 37 - 40).

1. Local Failure

   This type of failure was caused by crushing of the masonry unit at the region of the applied racking load before the failure of the wall. The crack pattern is shown in Figures 37 - 39.

2. Shear and Tensile Bond Failure

   The second possible failure was the diagonal cracks at the masonry/mortar interface in the upper and lower tensile stress regions due to attaining the maximum bond shear and bond tensile strength at mortar joints. The crack patterns are shown in Figures 37 - 40.

3. Diagonal Tensile Failure

   The third or final mode of failure was due to the horizontal and vertical diagonal cracks at the mortar

joints and also diagonal cracks through some of the mortar-masonry units. This type of crack started from the center and suddenly extended to the point of racking load and toe. The crack patterns are shown in Figures 37, 39 and 40.

Walls With Perimeter Reinforcement (Figures 41 - 43)

In this type of wall the steel had no effect until after the first crack had formed. The perimeter reinforcing controlled the opening of the cracks and their propagation after their first appearance. The following modes of failure were observed:

1. Bond Shear and Tensile Failure

This type of failure involves the vertical and horizontal cracks at the masonry-mortar interface in the upper tension region and it was followed by horizontal local failure at the joint between the loading beam and panel. The crack patterns are shown in Figures 41 - 43.

2. Failure Due to Twisting

Vertical cracks were observed at the junction of the web and flange near the racking load in the tests of walls #2 and #3. The possibility is assumed that this type of failure is due to one or more of the following factors:

(a) Twisting of the wall during the loading;

(b) Discontinuity of the perimeter reinforcement;

(c) Restraint of the corner against the movement of the spliced reinforcing bars;

(d) Independent movement of the vertical and horizontal spliced reinforcing bars.

The crack patterns associated with twisting are shown in Figure 44, Wall #2 and #3.

3. Diagonal Tensile Failure

In this case the observed crack started near the center and was followed by diagonal cracks at the vertical and horizontal mortar joints and in some parts through the mortar-masonry units. Then cracks developed from the lower compression region, joined the central cracks, and extended to the upper compression region toward the racking load. The diagonal crack patterns are shown on all 3 walls tested, in Figures 41 - 43.

4. Tensile Failure

At approximately the same load as diagonal tensile failure occurred, tensile failures were observed at the heel of the wall which caused splitting of the concrete fill and further shifting of the bars out of the perimeter. The associated patterns are shown in Figure 44, Wall #2, #3 and #4.

5. Compressive or Final Failure of the Shear Wall

When a number of vertical and diagonal cracks appeared at the toe then the steel caused splitting of the concrete fill and the bars bent out. The leeward column deformed, causing the failure of the shear wall structure. The associated crack patterns are shown in Figure 45, Wall #2, #3 and #4

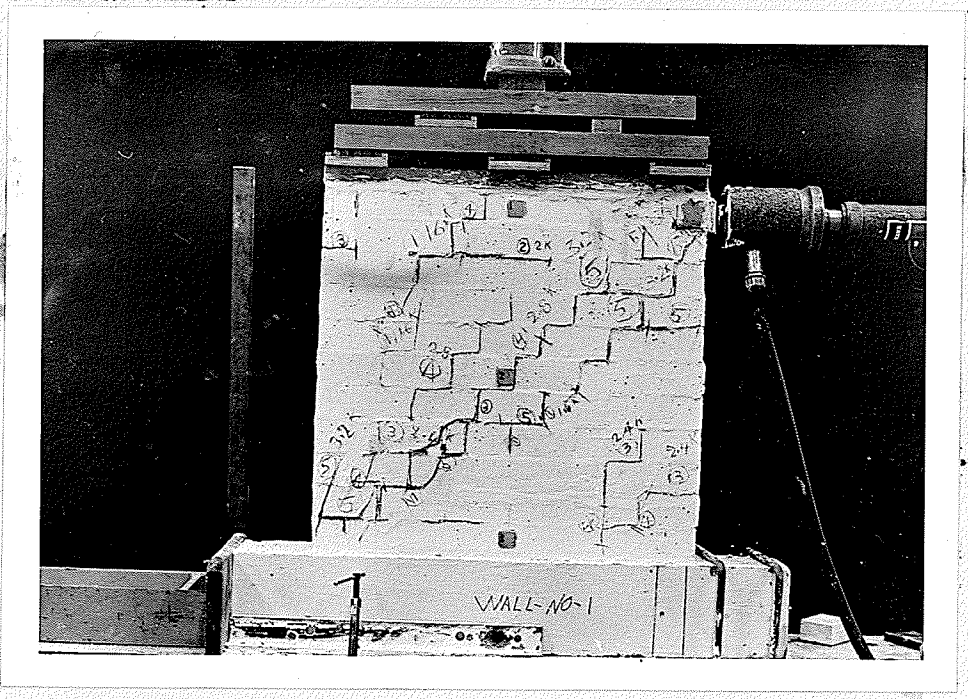


Figure 37 - Wall No. 1 Showing Local, Bond Shear, Bond Tensile and Diagonal Tensile Failures.

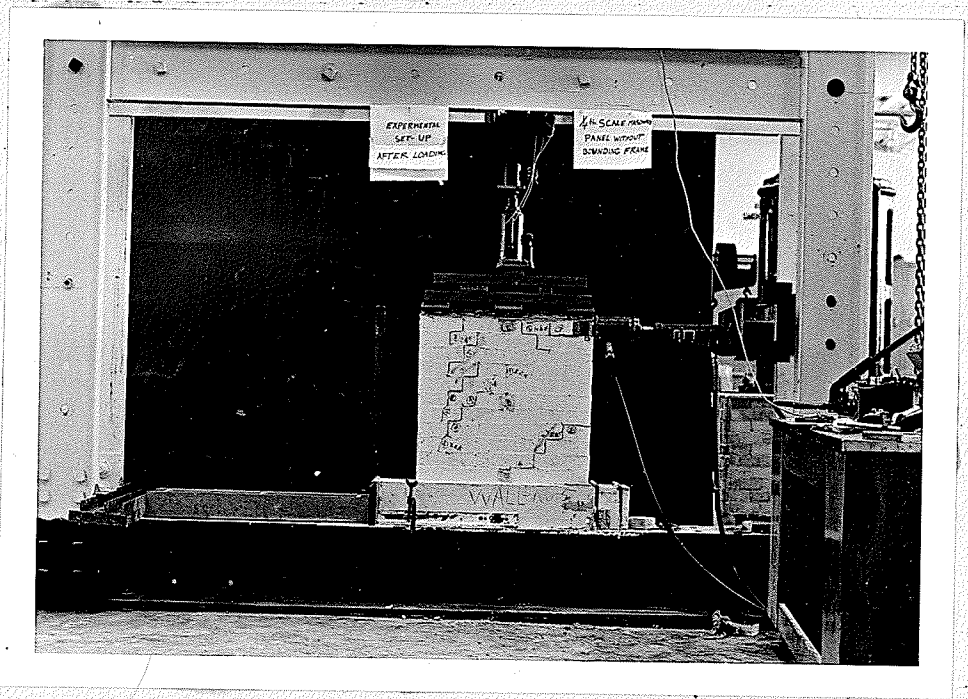


Figure 38 - Wall No. 5 Showing Local, Bond Shear and Bond Tensile Failures.

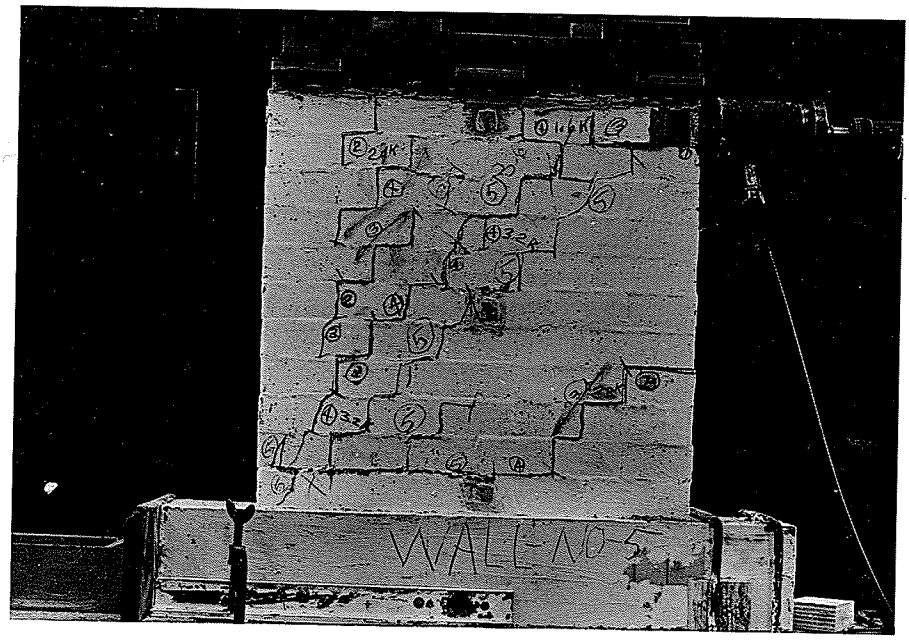


Figure 39 - Wall No. 5 Showing Local, Bond Shear, Bond Tensile and Diagonal Tensile Failure.

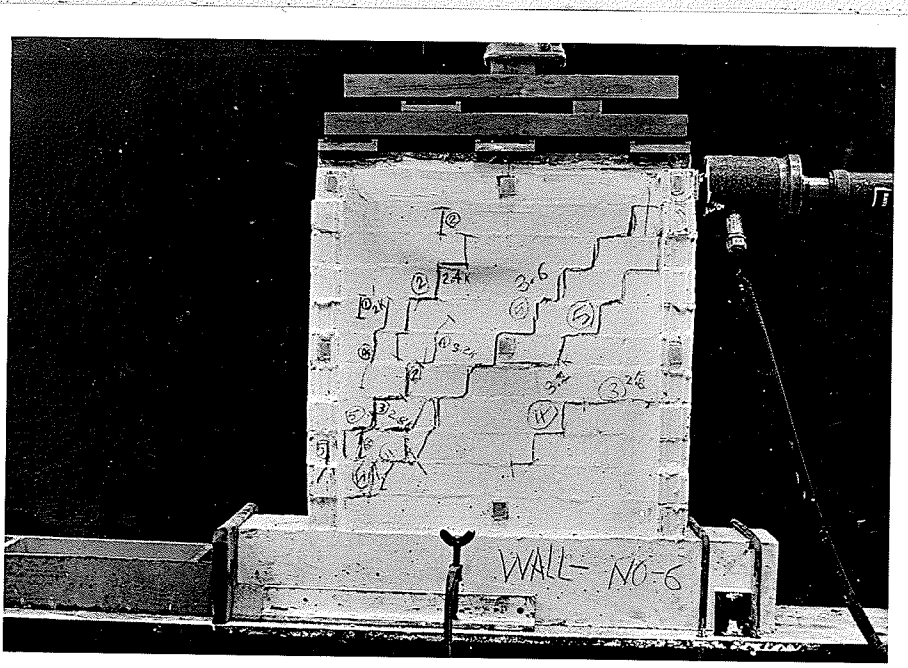


Figure 40 - Wall No. 6 Showing Bond Shear, Bond Tensile and Diagonal Tensile Failures.

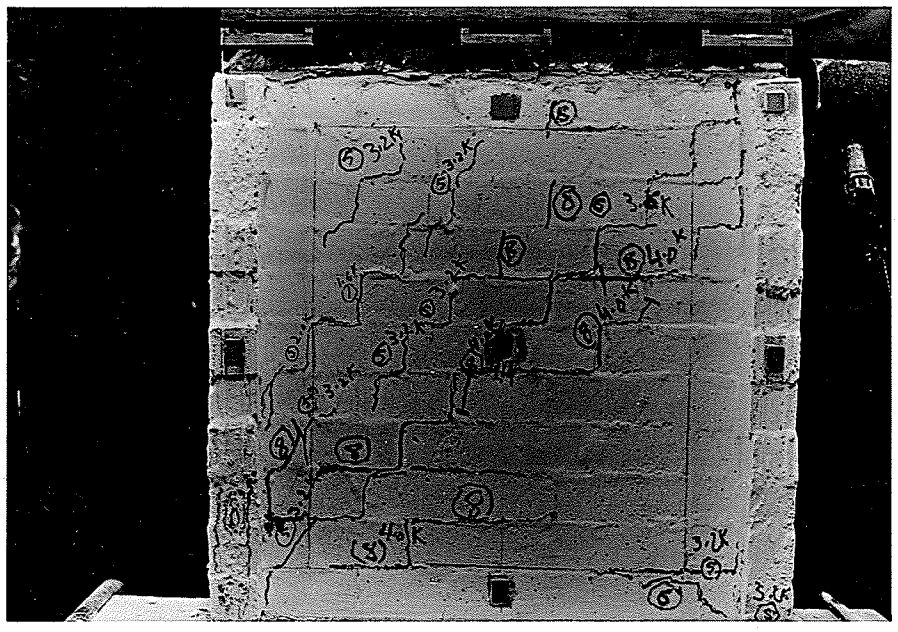


Figure 41 - Wall No. 2 Showing Bond Shear, Bond Tensile, Pure Tensile and Diagonal Tensile Failures.

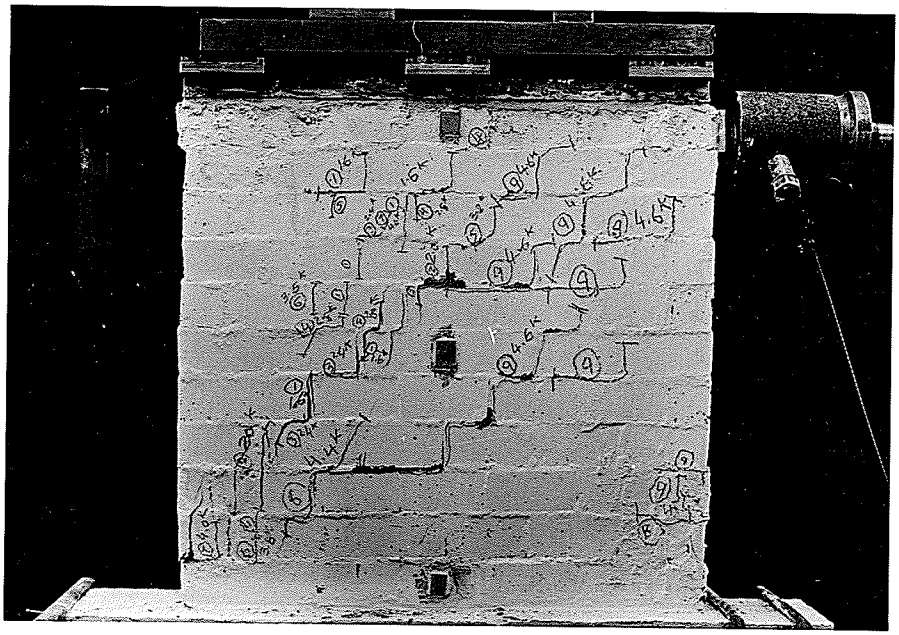


Figure 42 - Wall No. 4 Showing Bond Shear, Bond Tensile and Diagonal Tensile Failures.

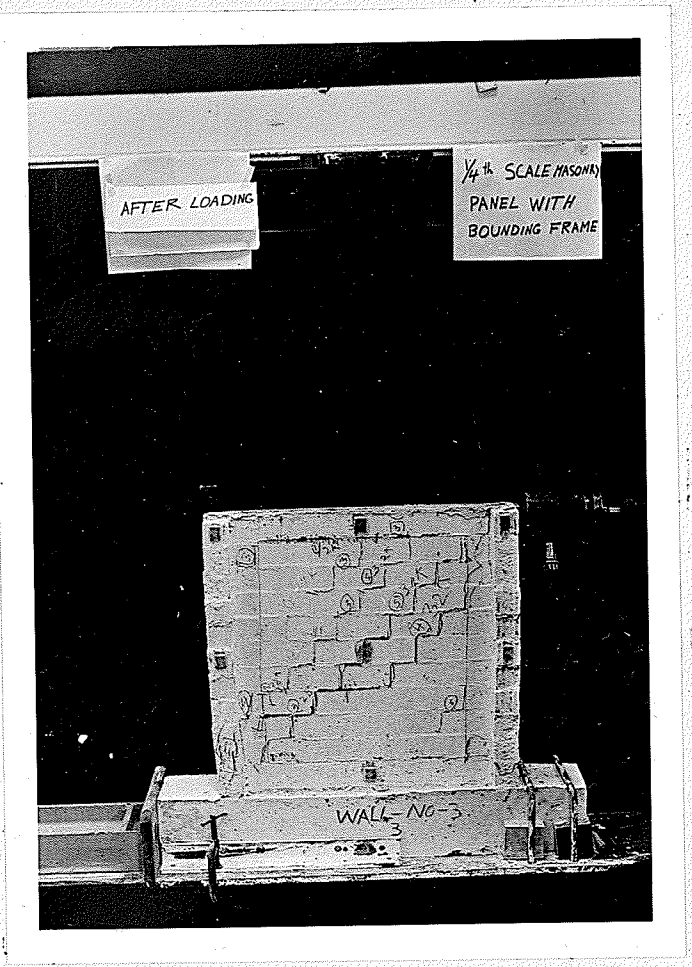


Figure 43- Wall No. 3 Showing the Bond Shear, Bond Tensile Diagonal Tensile Failures, and Crack along the Loading Beam.

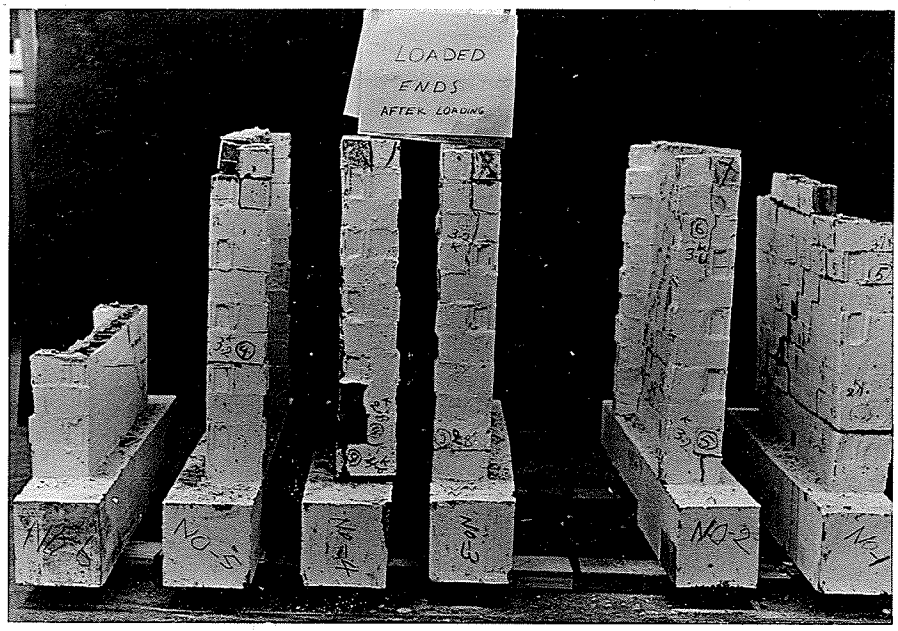


Figure 44 - Showing the Loaded Ends of All Six Walls After Failures.

Figure 45 - Showing the Free Ends of All Six Walls After Failures.

CHAPTER VII

ACCIDENTAL LOADING

In this investigation the following two different types of loading were accidentally applied before the application of the normal precompressive stress on the shear wall structure:

1. Static Racking Load (Wall No. 1)
2. Impact Load (Wall No. 6)

1. Static Racking Load

Wall No. 1 was accidentally loaded with a static racking load in the manner of a vertical cantilever beam. The wall was cracked at the bottom when the maximum tensile strength of the wall was reached at the heel of the structure. No indication of the load applied was obtained. The wall was then unloaded, white-washed and re-loaded with normal precompressive stress and static racking load. The crack pattern for the accidental static racking load only is shown in Figure 46, and is typical of the plain masonry shear wall structure, with no precompressive stress failing in tension at the bottom.

2. Impact Load

Wall No. 6 accidentally struck the horizontal loading rig while being placed against the loading cell inside

the loading frame. The wall was sheared off at the bottom of the last course, and as in the case of Wall No. 1, no indication of the load was obtained. The wall was repaired with plaster of paris, white-washed and tested within two hours. A very high bond strength at the repaired joint was observed in the wall, and the failure was sudden and brittle as compared to Walls No. 1 and No. 5. The crack pattern for the failure due to the impact load is shown in Figure 47, and it indicates that the plain masonry shear wall subjected to dynamic load can be expected to fail in pure shear, and that the failure is likely to be sudden and brittle. Clearly the plain masonry shear wall is a very poor structure for earthquake regions.

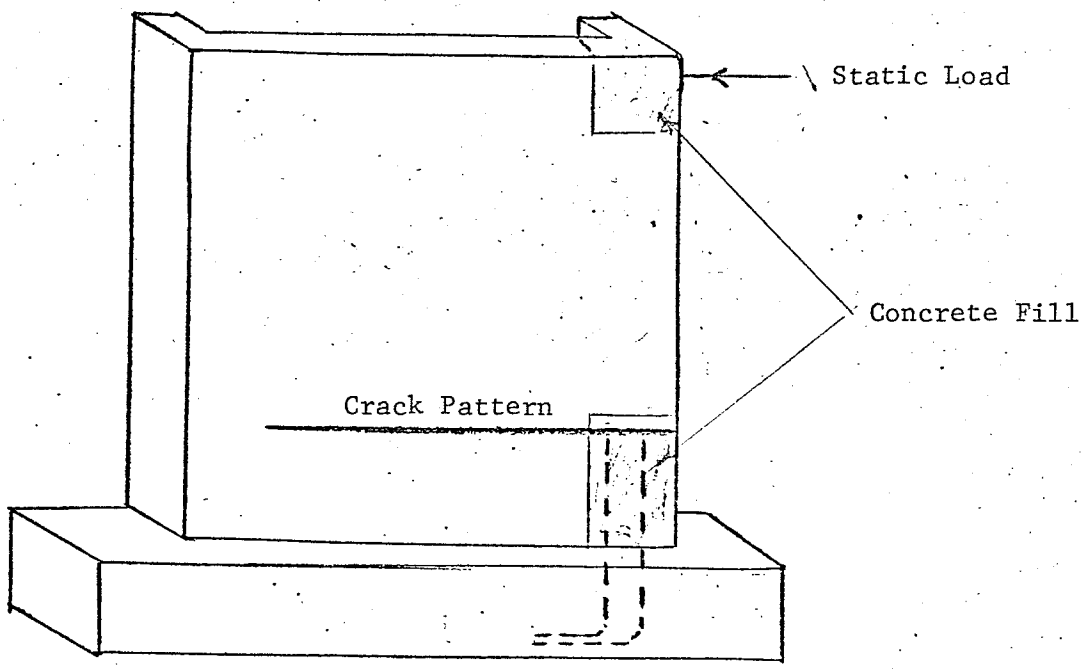


Figure 46 - Showing the Crack Pattern for Only Static Racking Load

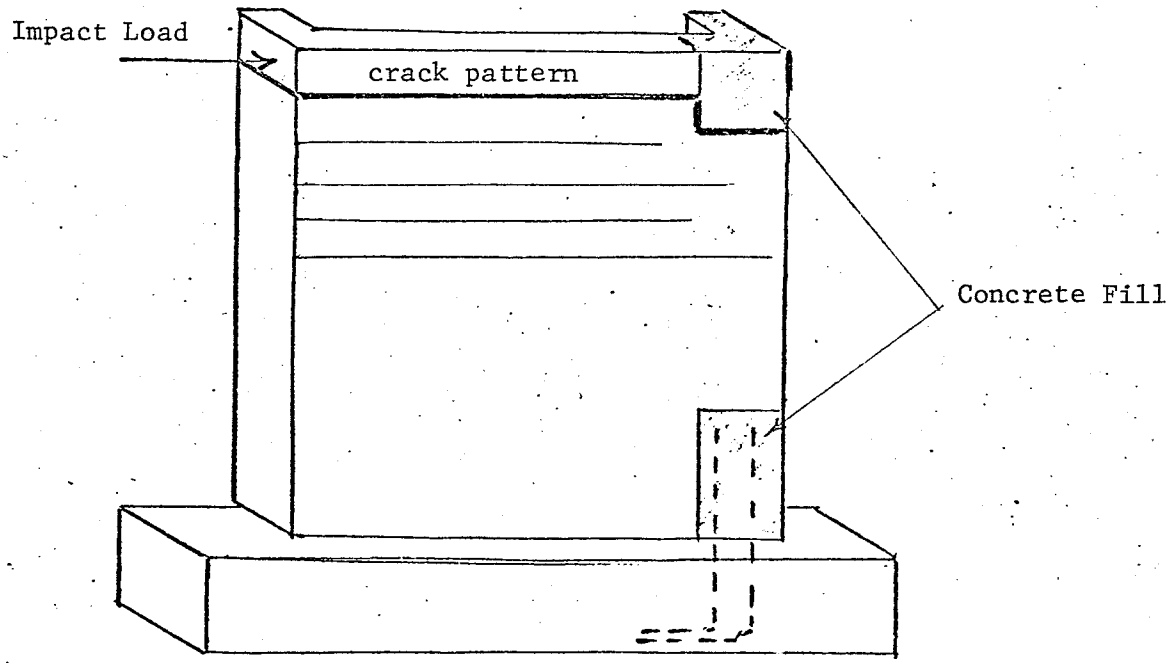


Figure 47 - Showing the Crack Pattern for Only Impact Load

CHAPTER VIII

PREDICTION OF STRENGTH

The strength and stiffness of the shear wall structure customarily has been calculated on the basis of prismatic shear walls with no flanges. It has further been assumed that the upper edge is restrained and the wall rests on a rigid foundation, but actually the shear wall structure is connected to face walls and may be restrained against horizontal rotation at the top by a rigid diaphragm.

In this investigation the shear walls are connected with flanges and subjected to precompressive stress and it is assumed that the thickness is small in comparison to the other dimensions, and that the walls are supported on fixed rigid foundations. The wall assemblies could be considered as a channel in cross section, as shown in Figure 48(a).

Stress Analogy

In general when a wall with flanges is subjected to a uniform normal stress "σ_n" along the center line of the web as shown in Figure 48(b), the stress on the flanges decrease linearly from "σ_n" at the web to zero at the outstanding point of the flange and the average stress at the bottom is less than that applied at the top. Since the length of the flange "b" is very much less than the length of the channel "L" therefore "σ_n" at the bottom of the wall section is approximately the same as "σ_n" at the top.

If the same wall is subjected to a lateral load at its upper edge, then the normal precompressive stress "σ_n", the bending stress "f", the combined stress distribution "($\sigma_n \pm f$)" at foundation level and the shear stress distribution at the junction of the web and flanges will be as shown in Figures 48 and 49. The angle of twist, and the deflection of the flanges and web in their respective planes will be discussed in the following paragraphs.

Considering the equilibrium of the flanges shown in Figures 49(b) and 49(d). The resultant vertical shear at the web connection for vertical equilibrium will be equal to:

$$V_L = \frac{bt}{2} (\sigma_n + 2f) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$V_w = \frac{bt}{2} (\sigma_n - 2f) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where

V_L - Vertical shear on leeward flange

V_w - Vertical shear on windward flange

σ_n - Normal precompressive stress

f - Normal bending stress

b - Length of flanges

t - Thickness of the wall section

For the moment equilibrium, horizontal shear forces must exist at the top and bottom of the flanges, equal to

$$R_L = b^2 t/H \, (1/6\sigma_n + 1/2f) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$R_w = b^2 t/H \, (1/6\sigma_n - 1/2f) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where

R_L - Horizontal shear force at the top of leeward flange

R_w - Horizontal shear force at the top of windward flange

H - Height of the wall

If it is assumed that the resultant vertical shear forces at the web connection are equal and opposite to the vertical shear forces of the corresponding flange at each junction, then from moment equilibrium horizontal shear force must exist at the top and bottom of the web as shown in Figure 49(c) equal to

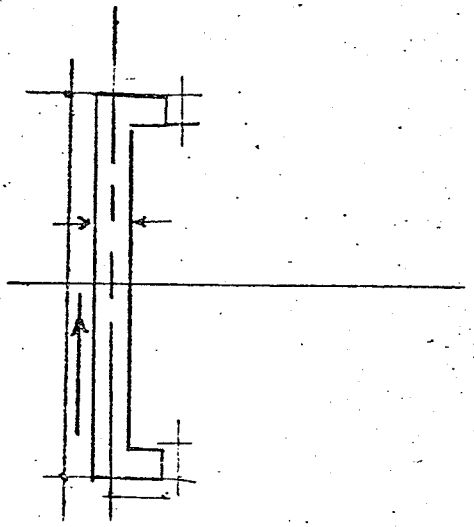
$$V = \frac{btL\sigma_n}{2H} + \frac{fL^2t}{6H} \quad \ldots\ldots\ldots\ldots\ldots \quad (5)$$

in which the first term represents the influence of the flanges and the precompressive stress and the second term the influence of the normal bending stress at the bottom of the shear wall.

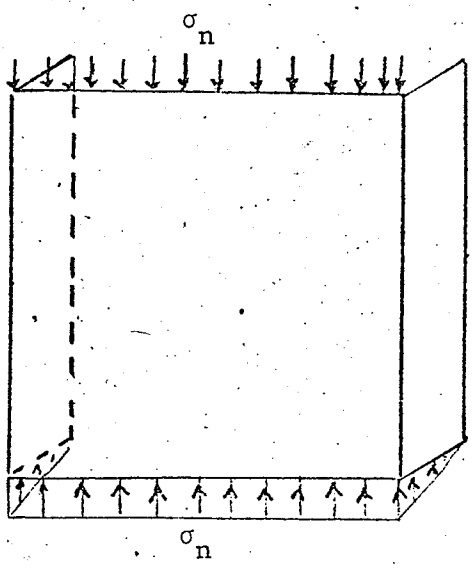
Also it indicates that the strength of the shear wall is increased only slightly with the existence of the flanges and normal precompressive stress, since the length of the flanges, b,is very small. Therefore, the effect of the flanges on the shear strength of the wall can be assumed to be negligible and in order to estimate the strength of the shear wall structures in this investigation it would be quite acceptable to consider the walls without flanges and estimate their strength and stiffness on this basis.

Strength of the Walls Without Perimeter Reinforcement

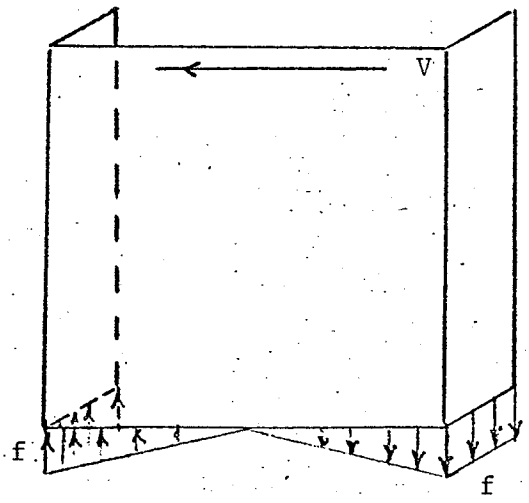
If it is assumed that the first crack occurs at the masonry mortar interface when it attains the maximum shear stress then the material resistance of the masonry wall to shearing stress will be equal to



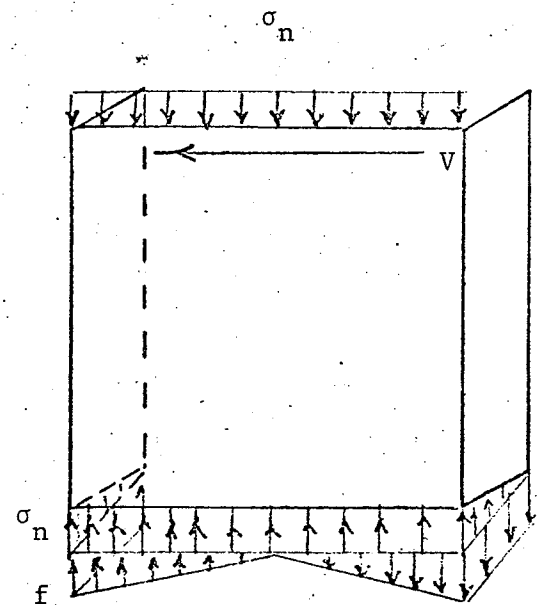
(a) Wall Cross-Section



(b) Precompressive Stress Distribution



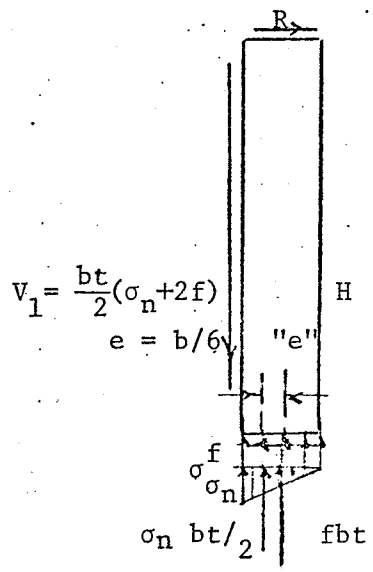
(c) Bending Stress Distribution-



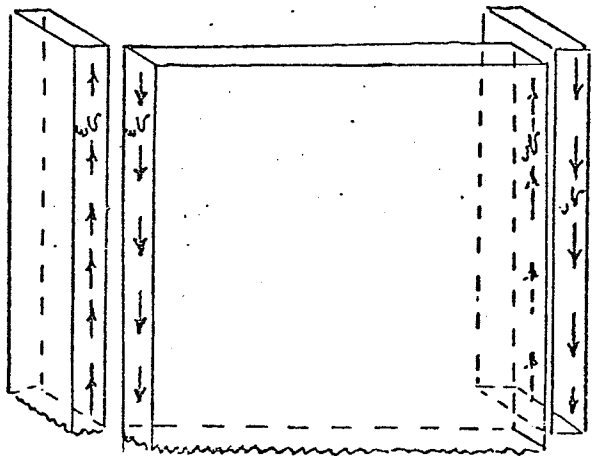
(d) Combined Stress Distribution

Figure 48 - Normal and Bending Stress Analogy

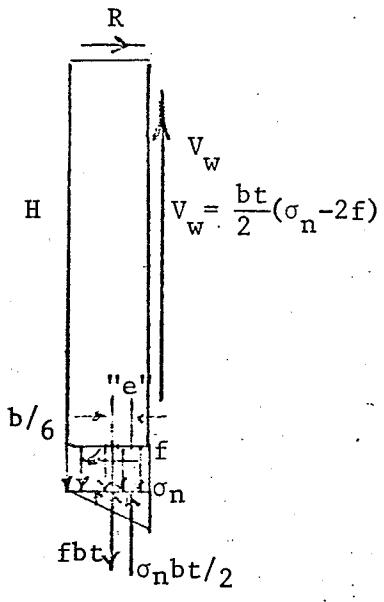
$$R = \frac{bt^2}{H}\left(\frac{1}{6}\sigma_n + \frac{1}{2}f\right)$$



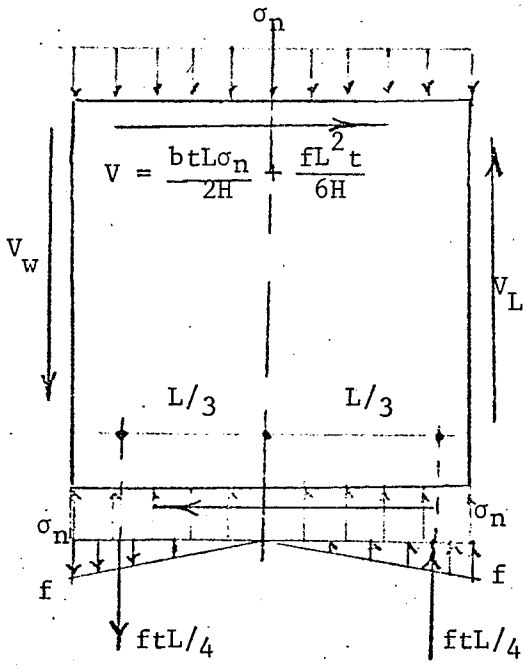
(b) Equilibrium of Leeward Flange



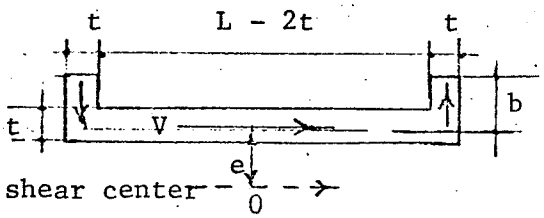
(a) Action Between Web and Flanges



(d) Equilibrium of Windward Flange



(c) Equilibrium of Web



(e) Resultants

Figure 49 - Shear Stress Analogy

$$v_{sc} = v_o + 0.3\sigma_n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)^*$$

and the total internal load at first crack is equal to:

$$V_c = A_n \, (V_o + 0.3\sigma_n) \quad \ldots\ldots\ldots\ldots\ldots \quad (7)$$

where

v_{sc} - Shearing stress at first crack;

v_o - The initial maximum shear strength of the composite masonry unit;

σ_n - The normal precompressive stress;

V_c - Total lateral load at first crack;

A_n - Net cross-sectional area of the wall

On the other hand if it is assumed that the failure of the masonry shear wall is due to the diagonal tensile cracks at the mortar and masonry joints, then it is necessary to know that:

1. When a vertical normal precompressive stress is applied to a composite masonry unit the internal shear resistance is increased by the increase of friction at the masonry-mortar interface. The following relationship has been suggested by POLYAKOV[10]

$$v_s = v_c + \mu\sigma_n \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

---

* Section 4.4.3.33 (3) CSDM[12] 1970 recommends vsw = (vm + 0.3 fcs) for estimating the maximum shear stress in masonry shear wall structures where $vm = \sqrt{f'm}$, vm = the allowable applicable shear stress and "fcs" is the compressive stress due to vertical loads.

where

v_s - The effective maximum shear stress of the masonry.

v_c - Initial maximum bond shear stress at the masonry-mortar interface.

σ_n - Normal precompressive stress;

μ - Coefficient of internal friction

2. When a wall with no precompressive stress is subjected to a lateral load the internal shear resistance at the masonry-mortar interface of the tension region is reduced as the normal tensile flexural stress increases and we have:

$$v_s = v_o - Cf_t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)^*$$

where

f_t - Tensile flexural stress normal to bed joints;

C - Effective coefficient of friction

v_o - Initial maximum bond shear stress at the masonry-mortar interface

v_s - The effective shear strength of the masonry

3. When a precompressed wall is subjected to a lateral load the combined normal stress distribution at the masonry-mortar interface of each course will be equal to:

$$\sigma = \sigma_n \pm f_t \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

---

* The effective shear strength of the masonry will be equal to zero when $Cf_t = v_o$ or $C = \frac{v_o}{ft}$ ...(1). In table 4.4.3.F section 4.4.3.13 CSDM[12] the maximum tensile flexural stress "ft" normal to bed joint for type N and M or S are given 28 and 36 psi respectively and the maximum shear stress "v_o" for type N and M or S are given 23 and 34 psi respectively substituting these values into (1) gives C = 0.83 for type N mortar and C = 0.95 for type M or S mortar. Which is very close to unity.

4. When a precompressed wall is subjected to lateral load the normal stress in the compressive region will be increased by increasing the compressive flexural stress. Therefore, the normal precompressive stress in Equation 8 will be equal to its effective normal stress Equation 10, with positive sign. Then the equation for estimating the ultimate internal shear resistance of the shear wall will be equal to

$$v_s = v_o + \mu(\sigma_n + f_t) \quad \ldots\ldots\ldots\ldots \quad (11)$$

and from Equation 9 the tensile flexural stress is equal to

$$f_t = \frac{v_o - v_s}{C} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

If it is assumed that C has a value of unity then from Equations 11 and 12 the ultimate shear stress and the ultimate lateral load are equal to

$$v_s = \frac{v_o(1+\mu) + \mu\sigma_n}{1 + \mu} \quad \ldots\ldots\ldots\ldots \quad (13)$$

and

$$V_{ult} = A_n \left(\frac{v_o(1+\mu)+\mu\sigma_n}{1 + \mu}\right) \quad \ldots\ldots\ldots\ldots \quad (14)$$

v_s - The ultimate shear strength of the masonry shear wall structure

V_{ult} - The ultimate lateral load

Strength of Shear Wall with Perimeter Reinforcement

The prediction of the strength of the precompressed shear walls with perimeter reinforcement is a difficult problem in the

opinion of the writer. The problem can be simplified by transforming the cross-sectional area of the vertical steel and the area of the concrete fill to the equivalent area of precast concrete - as shown in Figure 50, and use the sum of the transformed sections and the net plan area of the masonry work for estimating the strength of the precompressed shear wall structure with perimeter reinforcement.

If it is assumed that in shear wall structures with perimeter reinforcement the first crack occurs at the masonry-mortar interface when the maximum bond shear strength is reached and the steel has a negligible effect until the first crack appears, then the equation for estimating the cracking strength of the masonry shear wall with perimeter reinforcement will be equal to

$$V_a = A_T(v_o + 0.3\sigma_n) \quad \ldots\ldots\ldots\ldots\ (15)$$

where

$A_T = (n'Ag + Am)$

A_g - Area of concrete fill

A_m - Net area of the masonry work

n' - Ratio of the modulus of elasticity of the concrete fill and the modulus of elasticity of the masonry.

If it is assumed that the failure of the shear wall with perimeter reinforcement is due to the diagonal tensile cracks at the mortar and masonry joints, and after the first crack the steel controls the opening of the cracks and their propagation, and increases the ultimate strength of the wall, then the area of the steel must be considered in estimating the ultimate strength of the shear wall with

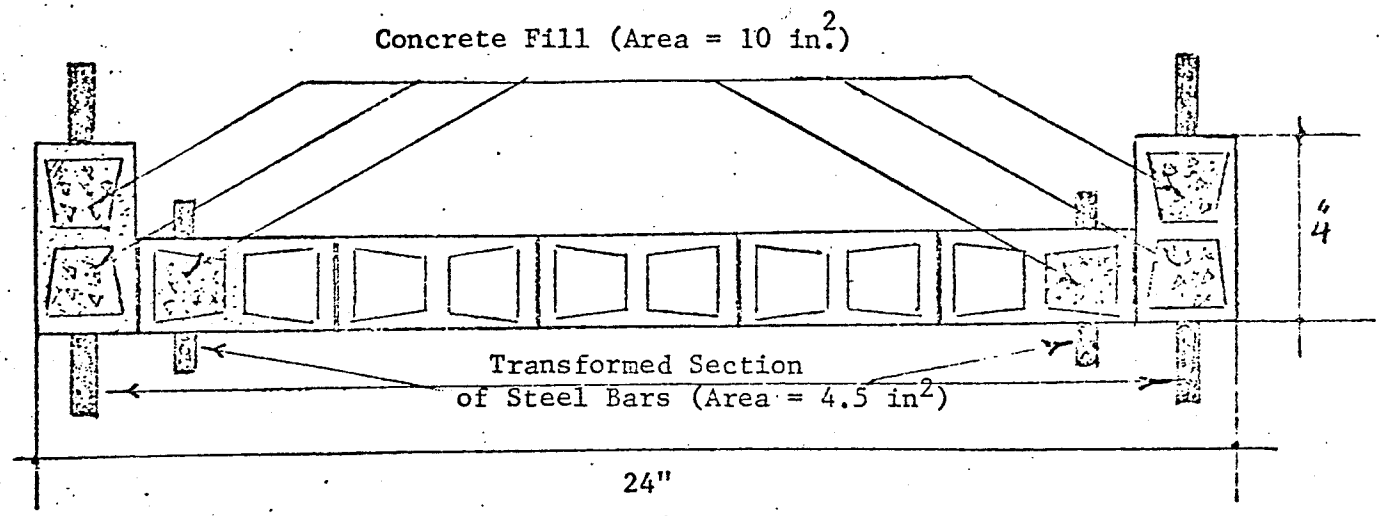


Figure 50 - Transformed Section

perimeter reinforcement. Based on equation No. 13, page 72, and the transformed section shown above, the equation for estimating the ultimate strength is:

$$V_{(ult)} = A_T\left(\frac{v_o(1+\mu) + \mu\sigma_n}{1+\mu}\right) \quad \ldots\ldots\ldots \quad (16)$$

where

$A_T = (nA_S + n'A_g + A_m)$

A_m - Net area of the masonry work

A_g - Area of the concrete fill

n' - Ratio of the modulus of elasticity of the concrete fill and precast concrete masonry units Eg/Em

n - Ratio of the modulus of elasticity of the steel and precast concrete masonry units Es/Em

A_s - Cross-sectional area of the vertical steel.

Since the strength of the masonry units and mortar composite in a shear wall structure normally will not be as great as that obtained in a laboratory couplet test because of factors of the workmanship, therefore, the ultimate strength and load at first crack should be expressed as coefficient K times equations (7,14, 15 and 16) where K is less than unity in parallel with the "Ø" factors of the ACI Building Code[14] and National Building Code of Canada[12] K might be assumed 0.85 for this investigation.

CHAPTER IX

PREDICTION OF STIFFNESS

The stiffness of a monolithic reinforced concrete shear wall structure is customarily expressed as the force required to produce a unit lateral deflection per unit of height. If the shear wall structure is of reinforced concrete and the base is fixed but the upper edge is unrestrained, the lateral deflection of the uncracked wall relative to its base when it is subjected to a lateral shear load is[11]

$$\Delta = \frac{V H^3}{3EcI} + \frac{6VH}{5AG} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (17)$$

where

V - lateral racking load

H - wall height from foundation to the lateral loading point

A - area of the wall panel cross-section

E_c - modulus of elasticity of concrete

I - moment of inertia of the cross-section

G - shear modulus of concrete

in which the first term represents the influence of bending and the second term the influence of shear. The shear modulus for concrete G assumed to be (0.4Ec). No rational expression for predicting the deflection of the plain masonry shear wall or masonry shear wall with perimeter reinforcement has been developed. There are many factors

that would have an effect on the stiffness of the shear walls of the type examined in this investigation such as the normal precompressive load, the influence of the flanges, the restraint against the rotation at the top, the base rotation, the amount of perimeter reinforcement and particularly the slippage of masonry units at the mortar joints. These factors can not all be accurately calculated, but from a knowledge of the overall properties of the masonry work and the results of the secondary tests one can estimate the values of the deflection at first crack and ultimate load.

In masonry shear wall structures the lateral deflection can be considered to make up the following five components:

1. Deflection due to shear
2. Deflection due to bending
3. Deflection due to the slippage of the masonry unit at the mortar joint
4. Twisting deflection due to the effect of flanges
5. Lateral displacement due to the base rotation.

1. Deflection Due to Bending and Shear

The deflection due to bending and shear can be estimated by the customary expression for predicting the deflection of monolithic reinforced concrete shear wall (see Equation (17) above).

2. Deflection Due to Slippage

When a composite masonry unit is subjected to direct shear or combined normal and shear forces, there will be a slippage at the masonry and mortar interface which may be assumed to be directly proportional to the applied

load and to be indirectly proportional to the bond strength at the masonry-mortar interface. If it is further assumed that the lateral deflections of the shear wall at first crack and failure are also each a function of the slippage per unit shear force of the composite masonry unit under the combined and direct shear forces respectively, then we have:

$$\Delta_{(cr)} = V_{(cr)} \times \delta_d \quad \ldots\ldots\ldots\ldots \quad (18)$$

$$\Delta_{(ult)} = V_{(ult)} \times \delta_s \quad \ldots\ldots\ldots\ldots \quad (19)$$

$V_{(cr)}$ - The load at first crack in kips;

$V_{(ult)}$ - The load at failure in kips;

δ_d - The deflection per unit load estimated from combined stress couplet test;

δ_s - The deflection per unit load estimated from direct shear test.

3. <u>The Lateral Displacements Due to Base Rotation</u>

The lateral displacement due to the base rotation can be very significant and it is usually large relative to the elastic wall deflection due to bending and shear. The following relationship can be used for predicting the deflection due to rotation, assuming rotation to take place about the heel and the angle of rotation is small.

$$\Delta_r = \delta_r \frac{H}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (20)$$

where

δ_r - The vertical movement of the base at the toe due to rotation

H - The height of the wall

L - The length of the wall

4. Twisting deflection due to the effect of flanges

If the shear force "V" is applied distance "e" from the center of the web in order to produce lateral deformation without twist, it is said to pass through the shear center of the section (Figure 51, page 80). Then

$$V\ e = RL$$

$$e = \frac{RL}{V}$$

Now assume the flanges as cantilever beams loaded at free end with R then the deflection of the flanges in Z direction at any point along the Y direction is

$$\Delta_z = \pm \frac{R\ Y^3}{3EI_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (21)$$

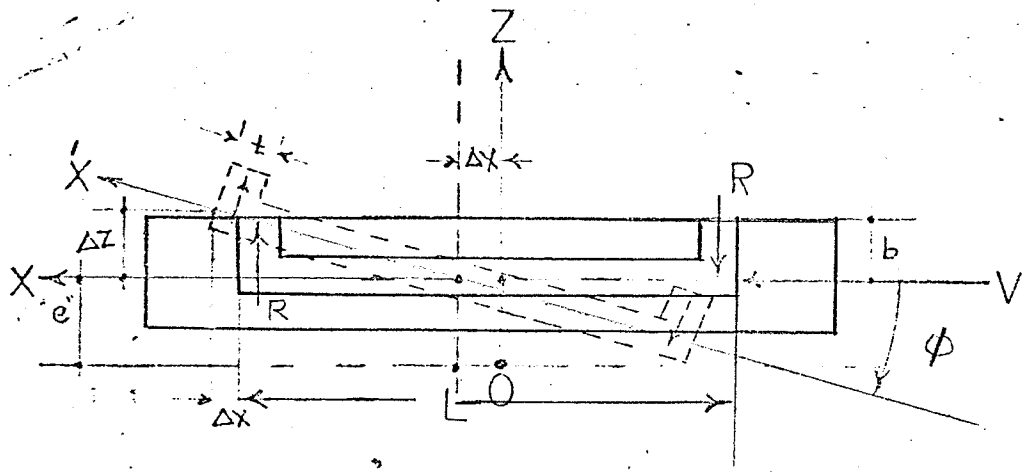
the angle of twist $\emptyset$ of the wall is

$$\emptyset = \frac{2\ \Delta\ z}{L} = \frac{2RY^3}{3EI} \quad \ldots\ldots\ldots\ldots\ldots \quad (22)$$

and the lateral deflection due to the flanges is

$$\Delta_x = \emptyset\ e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (23)$$

Since the deflections due to the flanges and bending are very small and may be neglected, then the expression for the total lateral deflection of the plain masonry shear wall structure at first crack and of ultimate load will be:



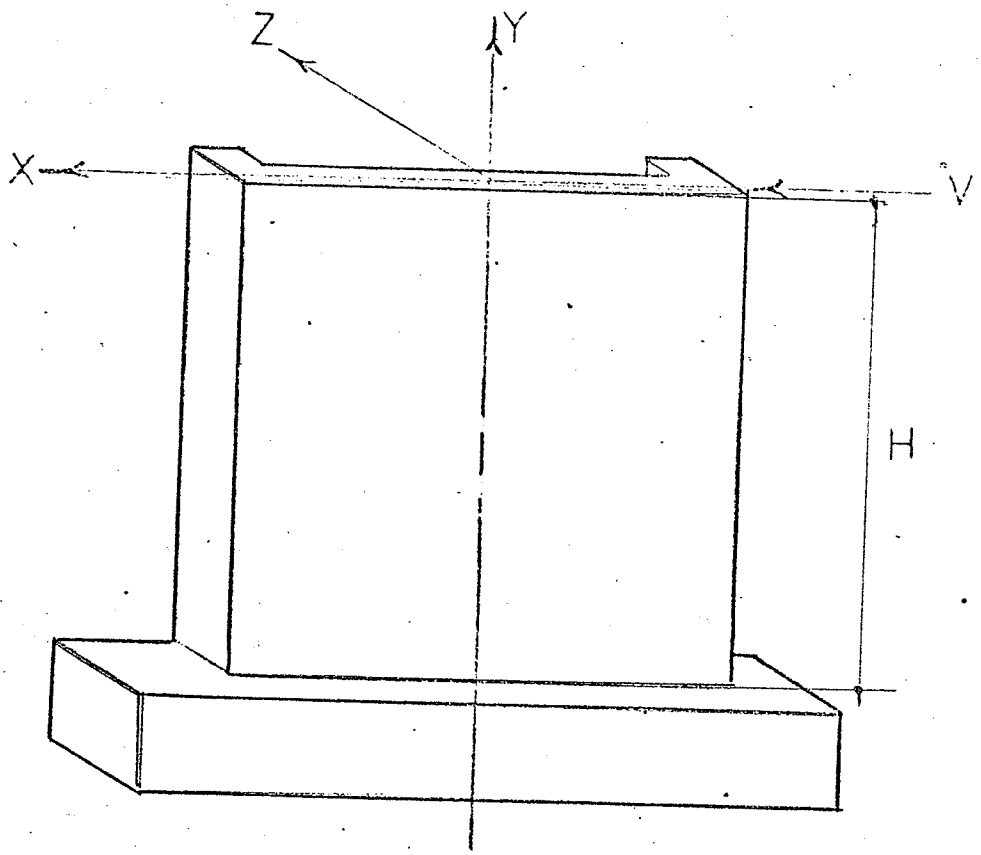


Figure 51 - Theoretical Twisted Shape

$$\Delta_{(cr)} = \frac{6V_{cr}H}{5AG} + V_{cr}\,\delta_d + \delta_r\,H/L \quad \ldots\ldots \quad (24)$$

$$\Delta_{(ult)} = \frac{6V_{ult}H}{5AG} + V_{ult}\,\delta_s + \frac{\delta r\,H}{L} \quad \ldots\ldots \quad (25)$$

If we further assume that the vertical bars are analogous to "dowel action"[13] in beams then deflection due to the slippage at first crack is restrained by the perimeter reinforcement in walls with perimeter reinforcement and the lateral deflection at first crack for the walls with perimeter reinforcement may be estimated by:

$$\Delta_{cr} = \frac{6V_{cr}H}{5AG} + \frac{\delta r\,H}{L} \quad \ldots\ldots\ldots\ldots\ldots \quad (26)$$

After the first crack appears vertical reinforcement will partially resist the lateral load and slippage of the masonry units at the mortar joint. When dowel cracking appears or steel yields, with increasing lateral load there will be a sudden slippage at the masonry mortar interface and then the slippage will increase with an increase of the load up to the wall failure. The total lateral deflection at failure may then be estimated from Equation 25.

CHAPTER X

DISCUSSION OF RESULTS

The discussion of this investigation is divided into the following three categories:

1. Discussion of the secondary tests;
2. Discussion of the proposed analytical expressions;
3. Discussion and evaluation of the test results.

1. Discussion of the Secondary Tests

The main purpose of performing the secondary tests was to develop basic data on which to predict the strength and stiffness of the precompressed masonry shear wall structure. Also investigated was the question of whether the bond strength at the masonry-mortar interface is influenced by a change in the properties of the mortar and if so to what extent.

Information on the first part of the question are the results listed in Tables XXII and XXIII Appendix A, which indicate that the bond strength at the masonry-mortar interface is influenced by the mortar properties. For example a comparison of specimens 4 and 5 (page 114) Appendix A) indicates that the bond shear strength and effective shear strength of the specimen No. 5 exceeded specimen No. 4 by 148% and 196% respectively. Also, the

slippage per unit force decreased by 60% and 65% for direct shear and combined stress couplet tests respectively.

An indication of the second part of the question will be found if the strength and stiffness of the two plain masonry walls having the same dimensions and loading system, but different mortar properties are compared. Table XIII (page 89) shows a theoretical comparison of the two masonry walls based on the properties of the same specimens (4 and 5) discussed above.

The table indicates that at first crack and failure respectively, the shear strengths of wall No. 2 are 66% and 30% greater than the strengths of wall No.1, and the deflections of wall No. 2 are 41.8% and 49% smaller than those of wall No.1. Also, the inelastic area under the idealized load deflection curve for wall No. 1 is 30% greater than the same area under the load deflection curve for wall No. 2; this area is of importance in judging energy absorption capacity of a structure under dynamic overload[11].

2. Discussion of the Proposed Analytical Expressions

The proposed formulae are based on the results of the secondary tests and the particular type of loading system discussed in this report. The theoretical results appear to agree reasonably well with the test results of the 6 walls.

Formula No. 14 - page 72 has also been compared to the results of 5 precompressed plain masonry walls tested by B.P. SINHA and A.W. HENDRY[10]. Table XIV presents a comparison of the results of 5 walls tested by SINHA[10] and the strength predicted by the proposed Formula No. 14. The ratio for experimental and theoretical results of 5 walls tested by SINHA are calculated and compared to the ratio of their experimental results and theoretical results based on the proposed Formula No. 14.

No other research is known to have been performed on the masonry precompressed shear walls with perimeter reinforcement. Therefore, the writer has been unable to develop a suitable comparison between the proposed formula and the test results of any other investigation.

The results of the secondary tests indicated that the behaviour of composite masonry units in direct shear and combined stress couplet tests are approximately linear up to failure. Since the proposed formulae are based on the secondary tests it would be feasible to assume that the proposed formulae are appropriate for predicting the ultimate strength and stiffness of the precompressed masonry shear walls with and without perimeter reinforcement in an elastic stress analysis in a range of precompressive stress between 35 - 200 psi, and 35 - 135 psi for walls without and with perimeter reinforcement.

Based on the above assumption and proposed formula, the relation between the normal precompressive stress

and the strength of the shear walls at first crack and failure, and the relation between the precompressive stress and shear strength of composite masonry units, are shown in Figure 58. Within the range of precompressive stress discussed earlier, the strength and stiffness of the precompressed masonry, shear walls built with the type N mortar could be estimated from graphs shown in Figure 58, page 100.

3. Discussion and Evaluation of the Test Results

The theoretical strength and stiffness of all 6 walls are calculated by the proposed formulae, based on the results from the secondary tests. The results of the secondary tests for the models are listed in Appendix A , Tables XVIII to XXI and Figures 60 and 61.

Tables XV and XVI - pages 91 and 92 show the summary of properties and theoretical and experimental results of the 6 walls tested. All 6 walls are subdivided into the following 4 groups:

Group	Description
A	Shear wall with foundation distortion and no perimeter reinforcement (wall No. 1 and 5).
B	Shear wall with no foundation distortion and no perimeter reinforcement (wall No. 6)
C	Shear walls with foundation distortion and spliced bars in perimeter (wall No. 2 and 3)

Group	Description
D	Shear wall with foundation distortion and continuous bars in perimeter (wall No. 4)

The average $\frac{V_{(Exp)}}{V_{(Theo)}}$ and $\frac{\Delta_{(Exp)}}{\Delta_{(Theo)}}$ ratios are calculated for each wall. $V_{(Exp)}$ and $V_{(Theo)}$ denote the Experimental and Theoretical lateral load. $\Delta_{(Exp)}$ and $\Delta_{(Theo)}$ denote the Experimental and Theoretical lateral deflections respectively.

The average $\frac{V_{(Exp)}}{V_{(Theo)}}$ ratio at failure for the walls in group A, B, C and D ranged from 1.06 to 1.32 while that for $\frac{\Delta_{(Exp)}}{\Delta_{(Theo)}}$ ranged from 0.836 to 1.34. Experimental and theoretical load deflection curves for all 6 walls under combined loading system are shown in Figure 52 and 53 and a comparison between the Experimental results and theory is illustrated in Table XVI. It is observed that there is an acceptable agreement between the analytical results and the limited number of experiments. The theoretical and experimental load deflection curves for all 6 walls, with the effect of foundation distortion removed, have been shown in Figure 54 and 55.

The experimental results for walls in group C and D could not be compared with walls in group A because they are based upon different normal precompressive stresses. For a more significant comparison, both compressive stresses should be equal. To compare these two types of structures the precompressive stress of 200 psi for walls in group A and B can be reduced to precompressive stress of walls in group C and D(135 psi).

Using the reduced precompressive stress in Formula No.14 page 72 the theoretical ultimate shear strength of the walls in group A and B can be proportionally reduced to a common precompressive stress. This provides a rational theoretical comparison between the two types of shear wall structures.

A theoretical comparison of the walls in group C and A is illustrated in Table XVII. The comparison indicates that the average $\frac{V_{(ult)C}}{V_{(ult)A}}$ ratio equals 1.533 and $\frac{\Delta_{(ult)C}}{V_{(ult)A}}$ ratio is equal to 1.46, which indicates that with 1.46% reinforcement and 27% concrete fill, the strength and lateral deflection of the walls with spliced bars in perimeter increased by 53.3% and 46.0% compared with the strength and lateral deflection of the walls without perimeter reinforcement.

In Table XVII the ratio of area under the load deflection curves for both types of structures have been calculated. The calculations show that $\frac{A_C}{A_{A^*}}$ ratio is equal to 2.40. This ratio is also very important in judging energy absorption capacity of the two types of structures under dynamic overload[11].

The test results are also compared with previous investigations. Figure 56 shows the comparison of the results of this investigation with the following type of shear walls tested by BENJAMIN[1,2,3] and adopted by BLUME[11]

1. Unreinforced masonry walls;

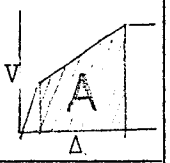
2. Reinforced concrete frames with unreinforced masonry walls;

3. Reinforced concrete walls with reinforced concrete frame;

4. Plain concrete walls with reinforced concrete frame.

In the opinion of the writer this comparison is also not relevant, for it is based on a different type of loading system. However, from the comparison of the load deflection curves one could get an idea of the "ductility" of the masonry walls with perimeter reinforcement as compared to reinforced concrete frames with unreinforced masonry walls. Also, the effect of precompressive stress on the strength and stiffness of the plain masonry shear walls can be clearly observed in Figure 57.

Table XII

A Comparison of Two Plain Masonry Walls With Different Mortar Properties										Area Under v-Δ Curve
Wall No.	Mortar Properties	Bond Shear Strength psi	V_{cr}	$V_{(ult)}$	δ_s	δ_d	Slippage at First crack Δ_d in.x10-3 $V_{cr} x \delta_d$	Slippage at Failure Δ_s in.x 10^{-3} $V_{ult} x \delta_s$	Coefficients of Friction μ	
1	6 Coarse Sand 1 Masonry Cement 1 Portland Cement 1 Lime	23.6	1.370	2.66	.0279	.00684	9.35	74.5	1.48	131×10^{-3}
2	6 Coarse Sand 2 Portland Cement	58.6	2.270	3.46	.0112	0.0024	5.45	37.6	1.45	92.1×10^{-3}
Wall 2 / Wall 1	Ratio	2.48	1.66	1.30	0.4	0.35	0.582	0.505	0.98	0.700

Wall Cross-Sectional Area - A = 30 in.^{2}

Precompressive Stress - 130 psi

K = 0.85

$$V_{cr} = KA(v_o + 0.3\sigma_n)$$

$$V_{ult} = AK\frac{(1 + \mu)v_o + \mu\sigma_n)}{1+\mu}$$

Table XIV

A Comparison of the Proposed Formulae No. 14 (Chapter IX) With Data From Reference No.10

Reference - No. 10								Formulae - No. 14	
Test No.	Coefficient of Friction μ	Initial Bond Shear Strength V_o Psi	Normal Precompressive Stress σ_n Psi	Total Area in^2	Experimental - Ultimate Racking Load V Kips	Theoretical Ultimate Racking Load - V Kips	Ratio of $\frac{V_{Exp}}{V_{Theo}}$	Ultimate Racking Load - by $V=K(\frac{V_o(1+\mu)+\mu\sigma_n)}{1+\mu}$ kips	Ratio of $\frac{V_{Exp}}{V_{Theo}}$
1	0.74	40	55	31	2.250	1.640	1.37	1.77	1.27
2	0.74	40	78	31	2.80	2.064	1.36	2.00	1.40
3	0.74	40	109	30.6	2.95	2.404	1.22	2.36	1.25
4	0.74	40	151	30.6	2.80	2.170	1.29	2.84	0.986
5	0.74	40	147.5	30.6	3.80	2.492	1.52	2.78	1.36

Table XV

Summary of the Theoretical Lateral Deflection and Displacement due to Three Different Factors

Type of Wall	Wall No.	Deflection - Inch x 10^{-3}									
		First Crack					Failure				
		Movement Due to Foundation Distortion in. x 10^{-3}	Deflection Due to Shear in. x 10^{-3}	Deflection Due to Slippage in. x 10^{-3}	Total Deflection in. x 10^{-3}	Total Experimental Deflection in. x 10^{-3}	Movement Due to Foundation Distortion in. x 10^{-3}	Deflection Due to Shear in. x 10^{-3}	Deflection Due to Slippage in. x 10^{-3}	Total Theoretical Deflection in. x 10^{-3}	Total Experimental Deflection in. x 10^{-3}
* Plain	1	60	2.05	15.8	77.53	101	60	3.68	58.6	122.28	160
	5	60	2.05	15.8	77.53	87	60	3.68	58.6	122.28	154
	Average	60	2.05	15.8	77.53	94	60	3.68	58.6	122.28	157
	6	0.0	2.05	15.8	17.53	12	0.0	3.68	58.6	62.28	53
** Reinforced	2	60	2.22	0.0	62.22	60	60	3.68	60.0	123.68	147
	3	60	2.22	0.0	62.22	75	60	3.86	60.0	123.68	141
	Average	60	2.22	00.0	62.22	67.5	60	3.86	60.0	123.68	144
	4	60	2.22	0.0	62.22	75	60	3.86	60.0	123.68	166

* Without Perimeter Reinforcement

** With Perimeter Reinforcement

Table XVI

Summary of Properties and Theoretical and Experimental Results

Type of Wall and Group	Wall No.	Cross Sectional Area A in^2	Precompressive Stress P/A lb/in^2	Experimental				Theoretical				Ratio at First Crack		Ratio at Failure	
				First Crack		Failure		First Crack		Failure					
				$V_{c\gamma}$ Kips	$\Delta_{c\gamma}$ in.x 10^{-3}	V_{ult} Kips	Δ_{ult} in.x 10^{-3}	V_{cr} Kips	Δ_{cr} in.x10^{-3}	V_{ult} Kips	Δ_{ult} in.x 10^{-3}	$\frac{V_{Exp}}{V_{Theo}}$	$\frac{\Delta_{Exp}}{\Delta_{Theo}}$	$\frac{V_{Exp}}{V_{Theo}}$	$\frac{\Delta_{Exp}}{\Delta_{Theo}}$
Type G Walls Without Perimeter Reinforce. A***	1	30	200	2.00	.101	3.6	160	1.84	77.53	3.30	122.28	1.09	1.30	1.09	1.31
	5	30	200	1.60	87	3.6	154	1.84	77.53	3.30	122.28	0.87	1.12	1.09	1.26
	Average	30	200	1.80	94	3.6	157	1.84	77.53	3.30	122.28	0.98	1.21	1.09	1.28
Walls Without Perimeter Reinforce. B**	6	30	200	1.60	12	3.6	53	1.84	17.53	3.30	62.28	0.87	0.68	1.09	0.836
Walls With Perimeter Re-Inforce. C*** Cont. Spliced Bars	2	44.5*	135	1.60	60	4	147	2.0	62.22	3.48	123.68	0.80	0.965	1.15	1.19
	3	44.5*	135	2.0	75	3.4	141	2.0	62.22	3.48	123.68	1.0	1.20	0.98	1.15
	Average	44.5*	135	1.80	67.5	3.7	144	2.0	62.22	3.48	123.68	0.90	1.08	1.06	1.16
Walls With Perimeter Re-Inforce. D*** Continuous	4	44.5*	135	1.60	75	4.6	166	2.0	62.22	3.48	123.68	0.80	1.20	1.32	1.34

*Transformed Cross-Sectional Area

** B - Fixed End

*** A - With Foundation Distortion

$\mu = 1.37$

$V_o = 13.7\ lb/in^2$

TABLE XVII

Theoretical Results of the Walls in Group A, C and D and an Average Theoretical Results for Walls in Group A Based on the Same Precompressive Stress as Walls in Group C and D

Type of Wall	Wall Group	Plain Cross Sectional Area in^2	Precompressive Stress psi	Theoretical				Area Under V-Δ Curve
				First Crack		Failure		
				V_{cr} Kips	Δ_{cr} in.x 10^{-3}	V_{ult} Kips	Δ_{ult} in.x 10^{-3}	
Wall Without Perimeter Reinforcement	A	30	200	1.84	77.53	3.30	122.28	115.2
	A*	30	135	1.37*	60*	2.27*	84.5*	56.4
Walls With Perimeter Reinforcement	C	44.5	135	2.0	62.22	3.48	123.68	135.9
	D	44.5	135	2.0	62.22	3.48	123.68	135.9

* Based on theory and reduced precompressive stress.

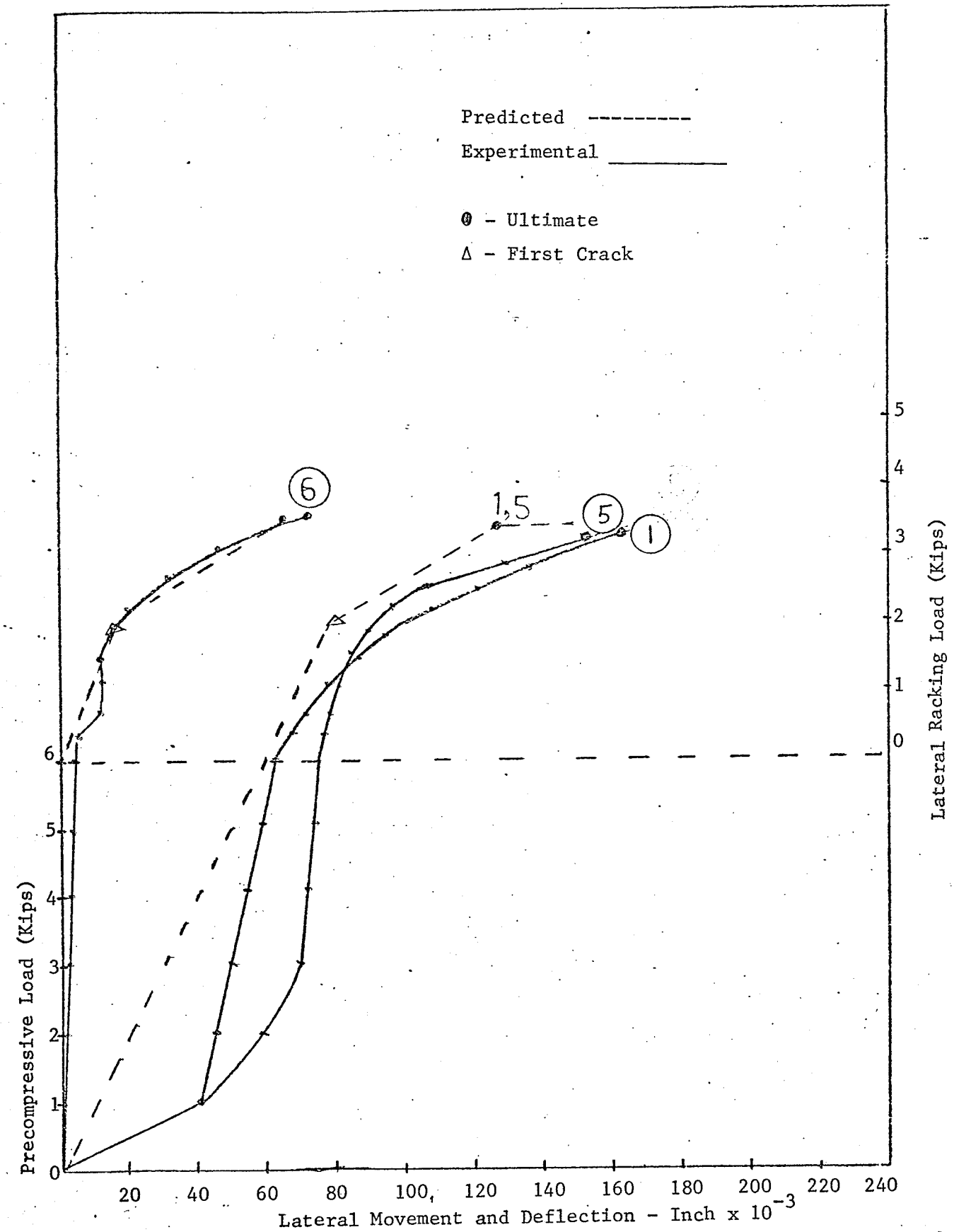


Figure 52 - Load Deflection Curves for Wall No. 1, 5,6, under Combined Loading.

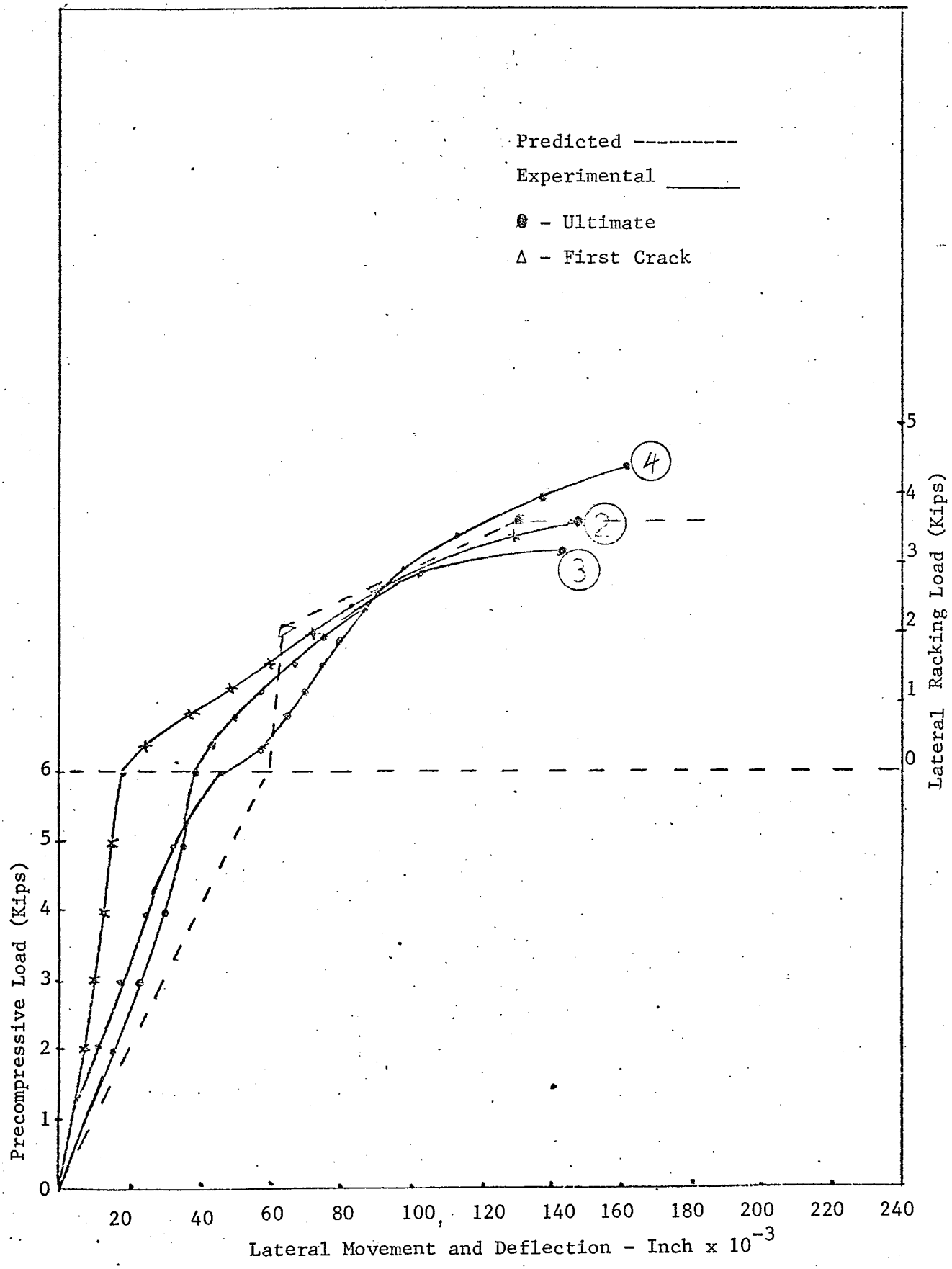


Figure 53 - Load Deflection Curves for Wall No. 2,3,4 Under Combined Loading

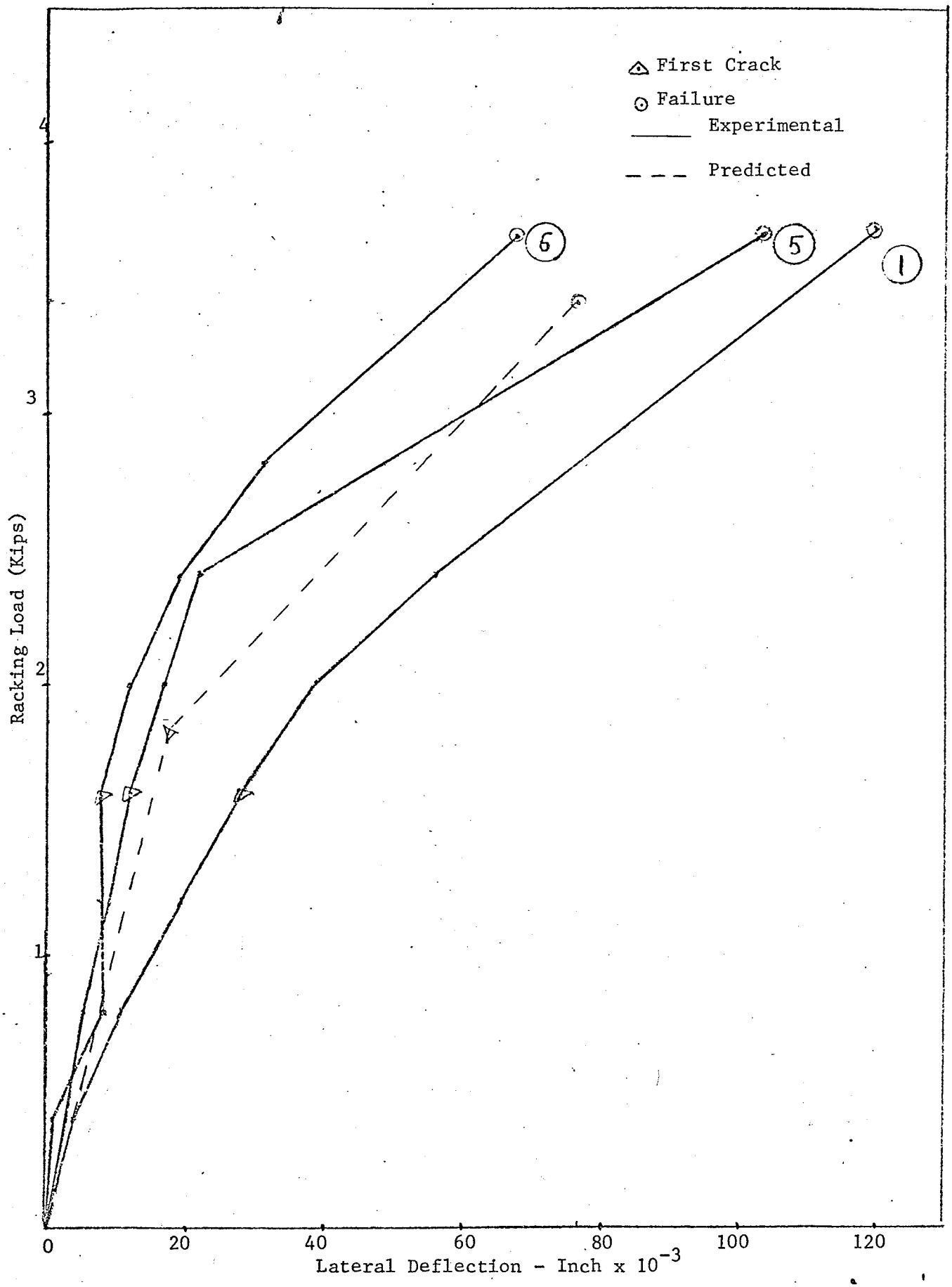


Figure 54 - Load-Deflection Curves for Walls No.1, 5, 6 After Precompressing

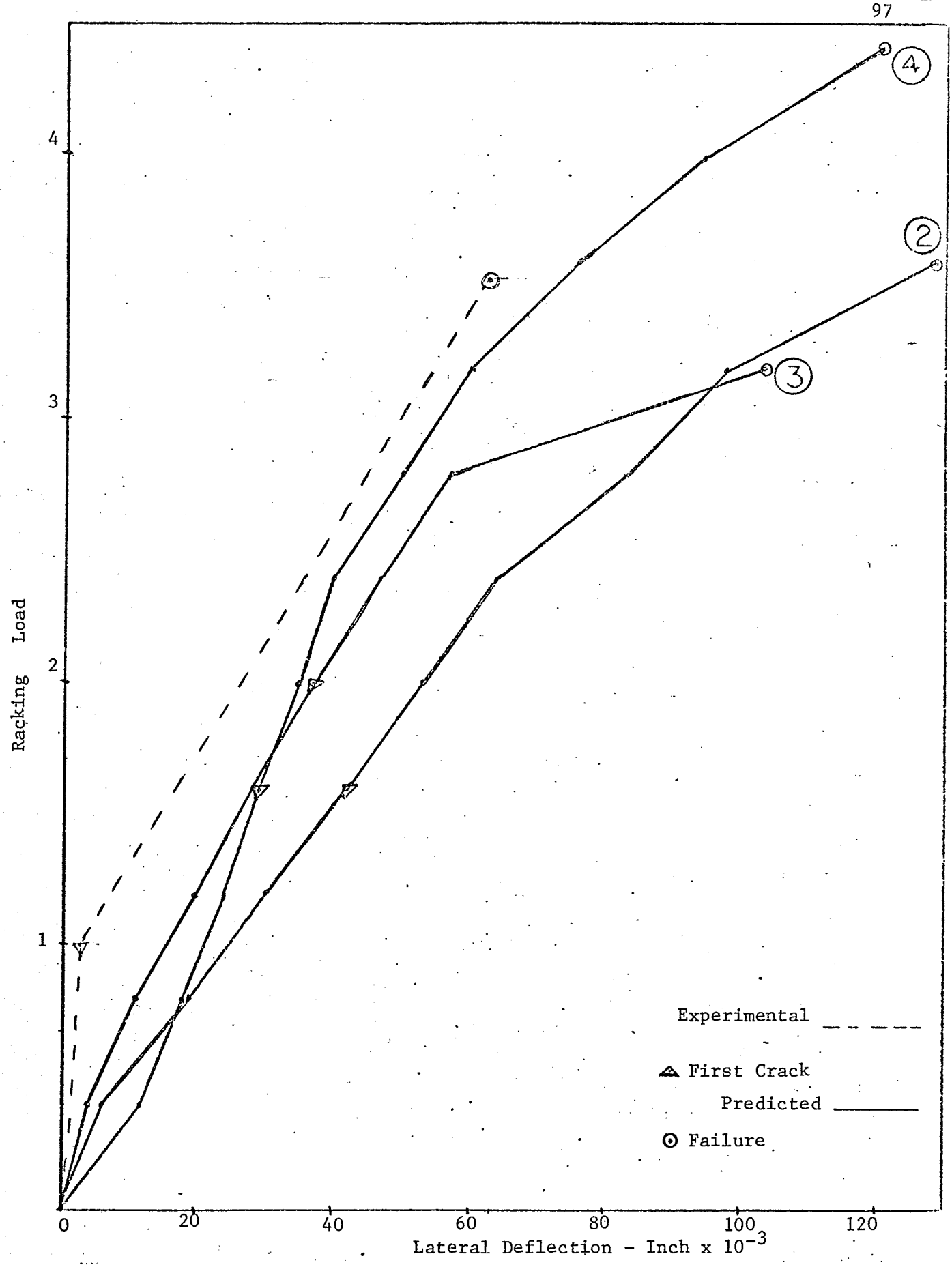


Figure 55 - Lateral Load-Deflection Curves for Wall No. 2,3 & 4 After Precompressing

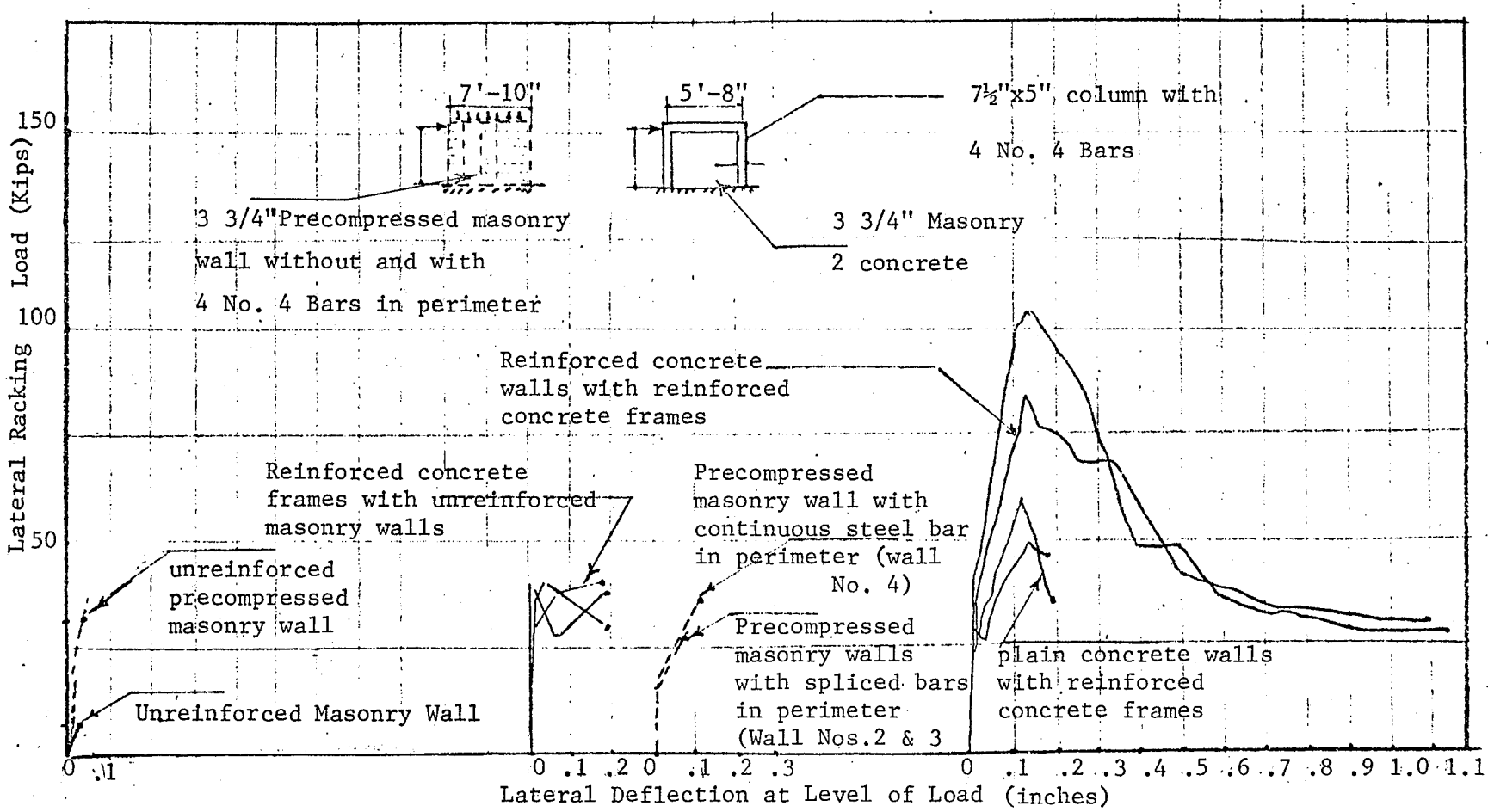


Figure 56 - Load Deflection Curves for One-Storey Shear Wall Structures

——— Taken from Reference No. 11 Page 132

------- Adapted from Test Data Table XXI on Basis of an Equivalent cross-sectional area.

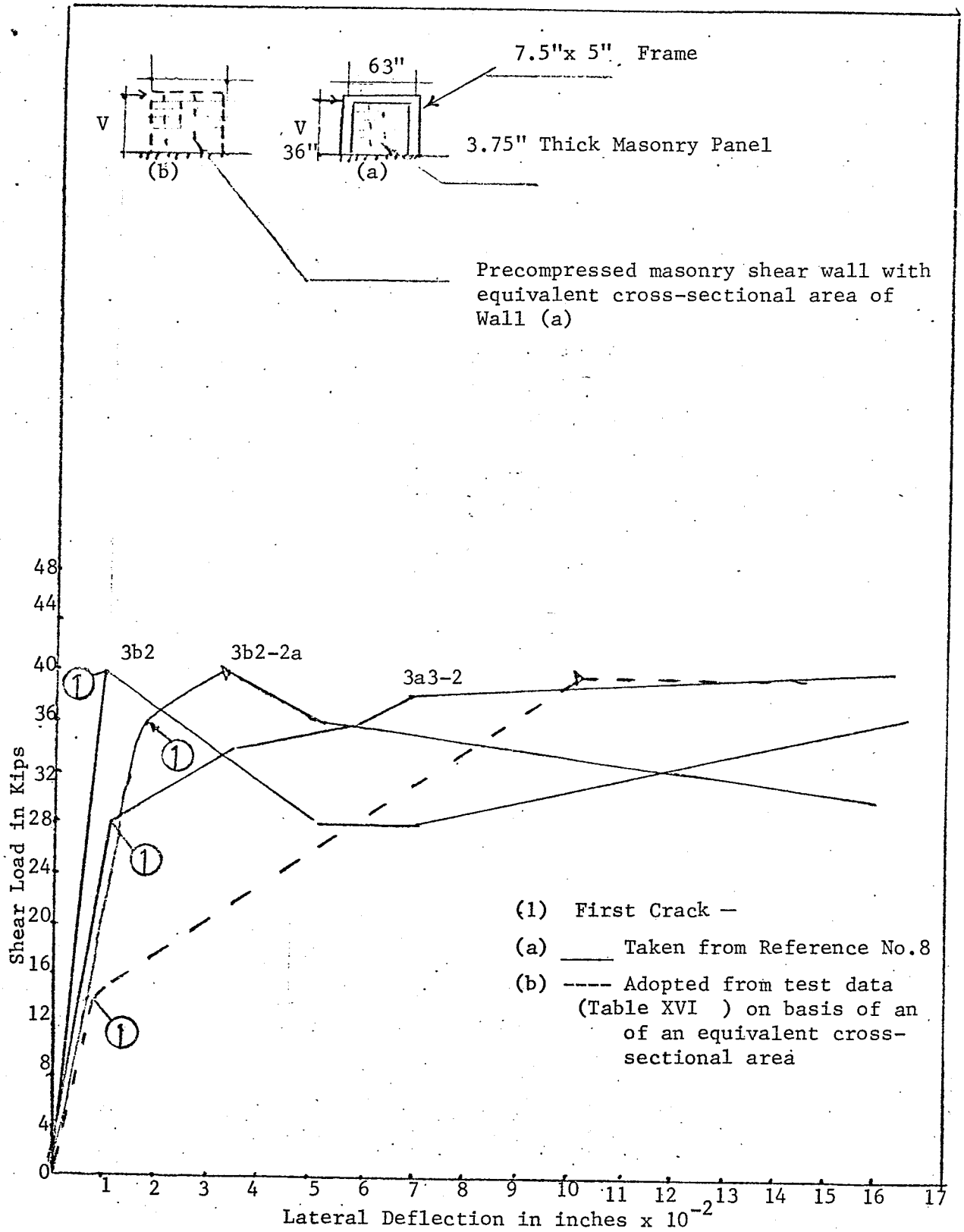


Figure - 57 - Load-Deflection Curves for One-storey Shear Wall Structures

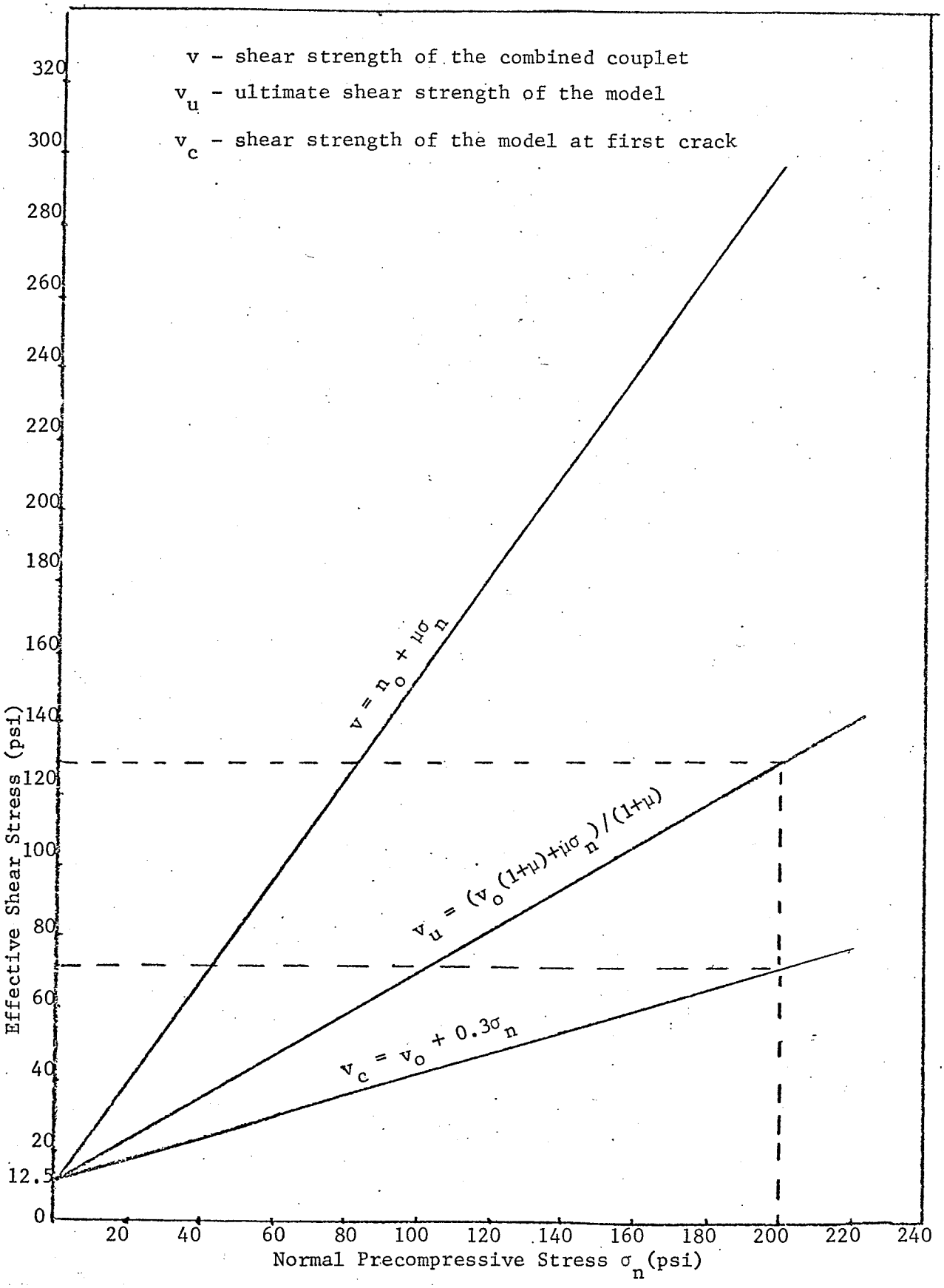


Figure 58 - Theoretical Relation Between Normal Compressive Stress and Shear Stress at First Crack, and Failure of the Model and the Difference Between Combined Couplet Test and Model Strength.

CHAPTER XI

CONCLUDING REMARKS

The results of the preliminary investigation of the precompressed masonry square shear walls for a one-storey building suggest that:

1. Unity of action exists between the reinforced perimeter and the inside panel;
2. The stiffness of the masonry shear wall structure is inversely proportional to the bond shear strength at the masonry and mortar interface;
3. Within the limit of the discussed precompressive stress the ultimate bond shear strength increases with an increase in precompressive stress;
4. Under the same loading system and dimensions the load at first crack is almost the same for walls with and without perimeter reinforcement;
5. Over 85% of the lateral deflection of the masonry shear wall structure is due to the slippage at the masonry-mortar interface;
6. The bond strength between the precast concrete and concrete fill can be very low;
7. The failure of a masonry shear wall without perimeter reinforcement and precompressive stress subjected to static lateral load is likely to be through a tensile failure at the bottom;

8. The failure of a masonry shear wall without perimeter reinforcement and precompressive stress subjected to dynamic load may be through a shear failure at the masonry-mortar interface;

9. The failure of the plain masonry shear walls with high bond strength at the masonry-mortar interface will be more sudden than that of the plain masonry shear walls with low bond strength at the masonry-mortar interface;

10 The proposed formulae and the design curves shown in Figure 58 appear to give a reasonable estimate of the strength and stiffness of the precompressed masonry shear wall structure;

11. The strength and the capacity of energy dissipation of the shear wall with 1.46% spliced steel bars and 27% concrete fill in the perimeter increased by 53.3% and 240% respectively, when compared to the strength and the energy dissipation capacity of the shear walls without perimeter reinforcement.

12. The strength of the shear wall with 1.46% continuous steel bars and 27% concrete fill in the perimeter was 28% greater than the strength of the shear wall with 1.46% spliced bars and 27% concrete fill.

CHAPTER XII

SUGGESTED FUTURE RESEARCH

Based on the limited experimental investigation contained in this thesis, the writer concludes that a great deal of research is necessary on the masonry shear walls with perimeter reinforcement subjected to static and dynamic lateral load. In particular he suggests work in the following areas:

1. Masonry shear walls for one-storey buildings under different ranges of precompressive stress;
2. Masonry shear walls for two-storey buildings;
3. Masonry shear walls with a diaphragm for one-storey and two-storey buildings.

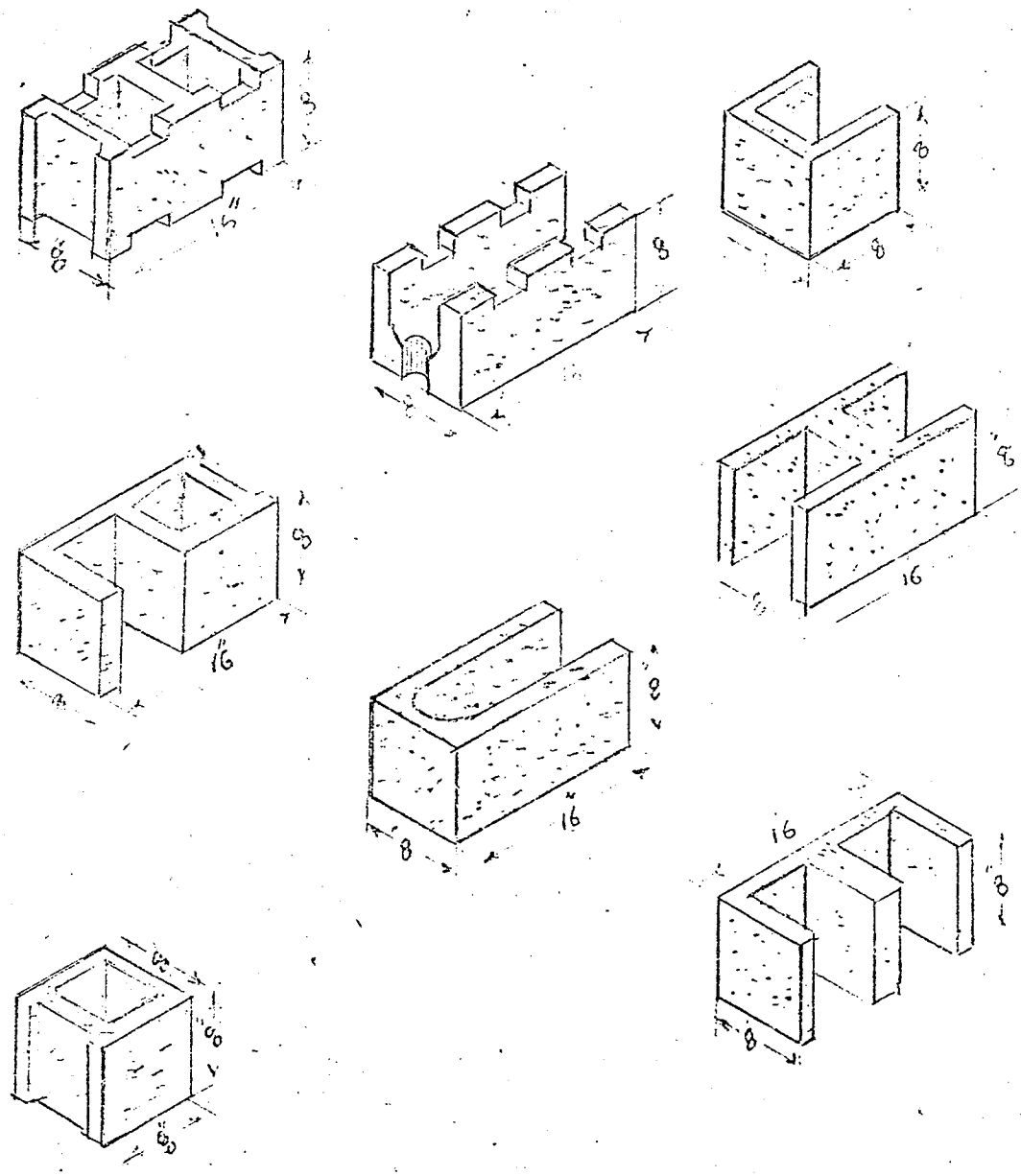
In order to develop rational mathematical expressions for predicting the strength and stiffness of the shear walls, it will be necessary to perform a great number of further tests on masonry composite units with common types of hollow and solid blocks and on the types of blocks proposed and shown in Figure 59. For example the writer suggests experimental work on:

1. Bond shear strength;
2. Diagonal tensile strength;
3. Tensile strength;
4. Slippage/unit force at the masonry-mortar interface in the direct shear test;

5. Slippage/unit force at the masonry-mortar interface in the diagonal tensile test.

Also, the behaviour of the reinforcing bars inside the perimeter the force and slippage at each joint of the shear wall structure have yet to be investigated.

In addition to the particular areas suggested above, a great deal of experimental work must be done to establish the manner of failure of practical shear walls under various types of loading, and the effect of workmanship on the strength and failure characteristics.



Proposed Type of Blocks for Reinforced and Plain Masonry Shear Wall Structures

Figure - 59

APPENDIX A

SECONDARY TESTS

In the main part of the thesis, it was assumed that the strength and stiffness of the masonry shear wall structures were a function of the bond shear strength, bond tensile strength, diagonal tensile strength, the prism compressive strength and the slippage of the masonry units at the masonry-mortar interface of the 1/4th scale blocks.

To establish strength and stiffness relationships between a composite masonry unit and the actual shear wall structure it was decided to investigate the following characteristic of the composite masonry unit.

1. Bond shear strength;
2. Effective shear strength by means of a combined stress couplet test at a 120^{o} angle with the horizontal;
3. Bond tensile strength;
4. Total slippage at the masonry-mortar interface in a direct shear test;
5. Slippage per unit shear force in a direct shear test;
6. Total slippage at the masonry-mortar interface in combined stress couplet test;
7. Slippage per unit shear force in a combined stress couplet test;

Three specimens were tested for each type of investigation, and all specimens were tested after two weeks of their mortaring. The loading rate of the specimens was kept low to avoid any dynamic effects. The load - slippage curves were drawn automatically on a 60,000 lb. hydraulic Riehle testing machine as shown in Figure 73. The average results obtained are listed in Tables XVIII - XIX and in Figure 60, and the relationship obtained between the shear stress and the normal stress is shown in Figure 61. Test specimens and loading procedures are shown in Figures 62 to 66.

Later, the same procedures were used by another graduate student (Mr. D.A. Baydack) in conjunction with the writer, for testing ten different types of composite masonry units, from which the following results were obtained:

1. Average test data, Table XXII;
2. Composite masonry unit properties, Table XXIII;
3. Average ultimate shear force, slippage per unit shear force and coefficient of internal friction, Table XXIV
4. Load - slippage curves for direct shear test and combined stress couplet test Figure 67;
5. Relationship between prism* compressive strength and bond shear strength, Figure 68.
6. Relationship between prism* compressive strength and effective shear strength by means of combined stress couplet tests, Figure 69;

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* Prism - A specimen which is built of the same type of mortar, under the same conditions if possible, and of the same thickness and boundary arrangement as for the structure.

7. Relationship between mortar compressive strength and bond shear strength, Figure 70;
8. Relationship between mortar and prism unit compressive strength, Figure 69;
9. Relationship between mortar compressive strength and effective shear strength by means of combined stress couplet tests, Figure (70);

Conclusion of the Secondary Tests

The study of the characteristics of the composite masonry units indicated that:

1. The internal shear resistance is increased by the increase of friction at the masonry unit-mortar interface;
2. The internal slippage is decreased by increasing the bond at the masonry unit-mortar interface;
3. There appears to be no direct relationship between compressive strength of the prism and compressive strength of the mortar cubes;
4. There appears to be no direct relationship between prism compressive strength and bond shear and effective shear strength;
5. The relationship between the load and slippage at the masonry unit-mortar interface is approximately linear;
6. There appears to be little direct relationship between mortar strength and bond shear strength and between mortar strength and effective shear strength of the composite masonry unit;

7. The bond tensile strength is approximately 40% to 45% of the bond shear strength at the masonry unit and mortar interface;
8. Unit compressive strength of the single blocks is greater than that of the prism - (2 block prism) unit;
9. Because plaster of paris sets very quickly, its use would not be parctical in masonry structures unless the time of setting could be retarded.

The test results also indicated that the bond shear strength, diagonal tensile strength and resistance to slippage of composite masonry units could be improved by changing the properties of the mortar. For example the bond strength of the portland cement mortar was more than 100% greater than masonry cement mortar; and the slippage in the masonry mortar unit with portland cement mortar was much less than that made with masonry cement mortar. Also the bond strength of the composite masonry unit with plaster of paris was much greater and the slippage was much less than the composite masonry unit of any other type of mortar. The significance of the bond strength and the slippage in shear wall structures is discussed in Chapter X.

Time was too limited to run a great many tests, therefore, the data presented must be interpreted as tentative only. It is recommended that many of these tests be run each with sufficient numbers of specimens as to make the results statistically significant.

TABLE XVIII

Bond Shear Strength at the Masonry - Mortar Interface

Test No.	Masonry Cement Mortar 1:3		Plaster of Paris		Ratio V^p_u/V^m_u
	V^m_u lbs.	v^p_u psi	V^p_u lbs.	v^p_u psi	
1	80	10	415	104	10.4
2	150	18.7	495	123	5.74
3	100	12.5	435	109	8.74
Average	110	13.7	448.33	112	8.283

Note
Test after 14 days
A = 8 $in.^2$ (contact area)

Note
Test after 24 hours
A = 8 $in.^2$ (contact area)

Table XIX

Bond Tensile Strength at the Masonry-Mortar Interface

Test No.	Masonry Cement Mortar 1:3		Plaster of Paris		Ratio f^p_t/f^m_t
	P_u lbs.	f^m_t psi	P_u lbs	f^p_t psi	
1	25	6.25	130	32.5	5.2
2	25	6.25	220	55	8.8
3	25	6.25	180	45	7.2
Average	25	6.25	176.6	44.16	7.07

Note
Test after 14 days
Contact area = 4 $in.^2$

Note
Test after 14 days
Contact area = 4 $in.^2$

Table XX

Test Data for Combined Stress Couplet Test (See Figure 5)

Test No.	Masonry Cement Mortar			Plaster of Paris		
	Total Force P_u lbs	Total Shear Stress $P_u \sin 120^\circ/A$ Psi	Total Normal Stress $P_u \cos 120^\circ/A$ Psi	Total Force P_u lbs.	Total Shear Stress $P_u \sin 120^\circ/A$ Psi	Total Normal Stress $P_u \cos 120^\circ/A$ Psi
1	350	75.5	44.0	1240	267	155
2	205	44.5	25.6	980	213	122
3	370	80	46.3	970	210	121
Average	308.23	66.8	38.7	1063	230	133

Note:
Test After 14 days
Contact Area = 4 $in.^2$

Table XXI

Ultimate Shear Force, Slippage/Kip and Coefficient of Internal Friction
Composite Masonry Units

	Direct Shear Test			Combined Stress Couplet Test			
Test No.	Shear Force Kips V_u	Total Slippage inch Δ_s	Slippage Per Unit Force Δ_s/V_u $(\frac{inch}{Kip})$	Shear Force Kips $v=P_u$ Sin 120°	Total Slippage inch Δ_s	Slippage Per Unit Force $\frac{\Delta_s}{V}$ $(\frac{inch}{Kips})$	Coefficient of Internal Friction μ
1	0.080	0.00145	0.0178	0.302	0.0260	0.0086	
2	0.150	0.00268	0.0178	0.177	0.01525	0.0086	
3	0.100	0.00178	0.0178	0.320	0.0276	0.0086	
Average	0.110	0.00197	0.0178	0.266	0.0230	0.0086	1.37

Note

1. Type of Block - hollow block
2. Size of Block - 2 in.x4 in.x2in.
3. Mortar Thickness - 3/16 in.
4. Mortar Properties - 6 Silica e sand and 2 masonry cement

Table XXII

Average Test Data for Ten Different Composite Masonry Units

Test No.	No. of Tests	Mortar Properties	Direct Shear lbs	Compressive Strength of Prism (lbs)	Compressive Strength of Mortar Cube (lbs)	Diagonal Load lbs. (couplet test)
1	3	6-Course Sand 2-Portland Cement 1-Lime	1687	79000	34150	8280
2	3	6-Fine Sand 2-Portland Cement 1-Lime	1440	65000	37275	8840
3	3	6-Course Sand 2-Masonry Cement 1-Lime	775	60000	16400	5150
4	3	6-Course Sand 1-Masonry Cement 1-Portland Cement 1-Lime	1507	70000	25725	6033
5	3	6-Course Sand 2-Portland Cement	3760	75000	65500	17650
6	3	6-Course Sand 2-Portland Cement 2-Lime	1618	63000	31025	8783
7	3	6-Course Sand 2-Portland Cement 2-Lime	1567	66000	18400	5900
8	3	6-Course Sand 2-Portland Cement 1-Lime	1203	74000	36550	8683
9	3	6-Silicate Sand 2-Masonry Cement	768	85000	16166	2400
		Plaster of Paris	3328	83000	27666	12660

Note: 1. The Tests for Specimen 1-8 were conducted along with Mr. D.A. Baydack. For details see thesis by D.A. Baydack.(15)

2. Brick Size - 2" Thick x 4" wide x 8" Length (nominal size).

Table XXIII

Composite Masonry UnitsProperties

Test No.	Precast* Concrete Solid Brick Size	Mortar Joint Thickness in.	Size of Mortar Cube in.	Prism Compressive Strength lb/in^2	Bond Shear Strength lb/in^2	Effective Shear Strength lb/in^2 (couplet test)	Mortar Compressive Strength lb/in^2
1	2"x4"x8"	3/8	4"x4"x4"	2470	26.4	258	2130
2	"	"	"	2030	22.6	276	2660
3	"	"	"	1870	12.1	161	1025
4	"	"	"	2190	23.6	187	1610
5	"	"	"	2340	58.6	552	4100
6	"	"	"	1965	25.3	274	1940
7	"	"	"	2060	24.5	184	1150
8	"	"	"	2310	18.8	271	2280
9	"	"	"	2660	12.5	68	1010
10	"	"	"	2600	5.2	260	1730

* Actual size $2\frac{1}{4}$ x $3\frac{5}{8}$ x $7\frac{5}{8}$. Stresses based on nominal size to be conservative.

Table XXIV

Average Ultimate Shear Force, Slippage/Kip Force and Coefficient of Internal Friction

Test No.	Direct Shear Test			Combined Stress Couplet Test			Coefficient of internal friction
	Shear Force Kips V_u	Total Slippage inch Δ_s	Slippage Per Unit Force $\delta_s = \frac{\Delta_s}{V_u}$ inch/kip	Shear Force kips $P_u \sin 120^o$	Total Slippage inch Δ_s	Slippage Per Unit Force $\delta = \frac{\Delta_s}{P \sin 120^o}$ inch/kips	μ
1	1.68666	0.05	0.0297	7.16	0.0583	0.00815	1.52
2	1.24166	0.05	0.0402	7.64	0.0483	.00635	1.59
3	0.775	0.046	0.0538	4.45	0.05	0.011	1.46
4	1.50666	0.042	0.0279	5.20	0.036	0.00684	1.48
5	3.760	0.042	0.0112	15.2	0.036	0.0024	1.45
6	1.61833	0.0444	0.0274	7.60	0.0333	.0044	1.54
7	1.56666	0.0386	0.0246	5.10	0.05	0.0098	1.465
8	1.20333	0.0275	0.0228	7.5	0.0416	0.00555	1.59
9	0.800	0.0146	0.0183	1.90	0.015	0.0079	1.10
10	3.328	0.048	0.0145	10.90	0.03	0.00275	1.46

1. Type of Block - Solid Precast Concrete Bricks
2. Size of Brick - 2 in. thick x 4 in. wide x 8 in. long (nominal size)
3. Mortar thickness - 3/8 in. (for mortar properties see table)

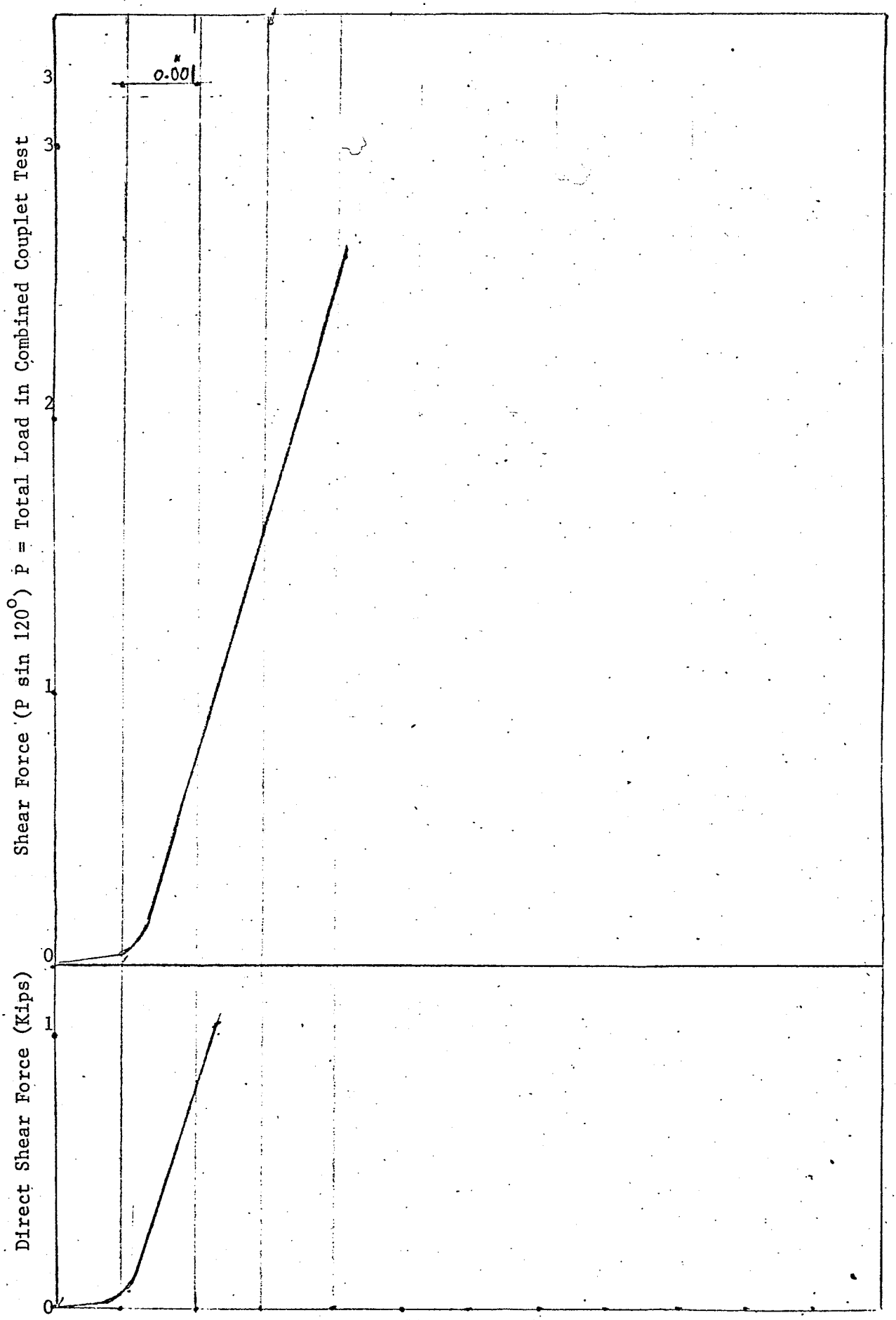


Figure - 60 - Load - Slippage Curves from Direct Shear Test and Combined Couplet Tests of 1/4 Scale Blocks

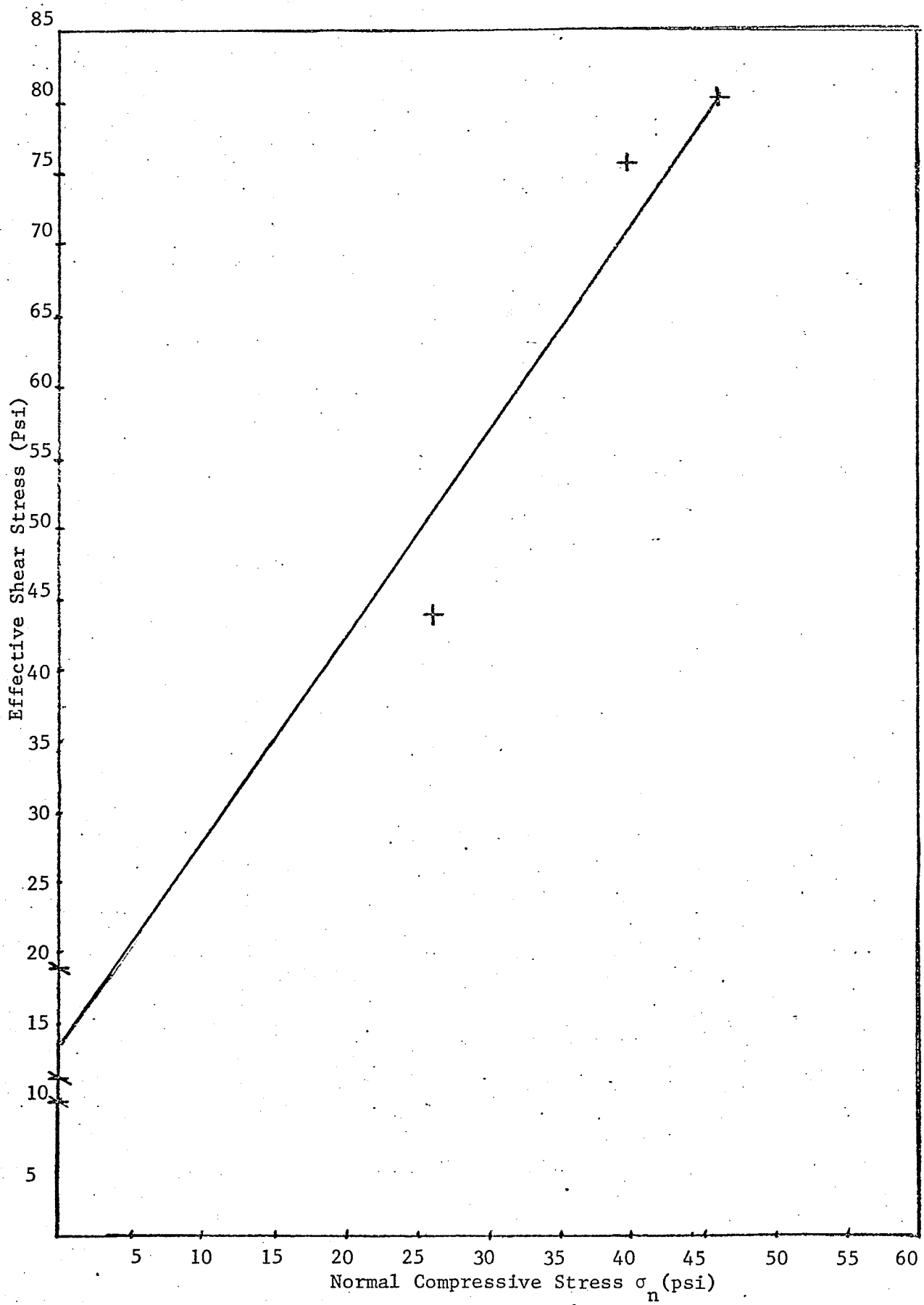


Figure 61 - Relation Between Effective Shear and Normal Compressive Stress
(For Test Procedure See Figure 64)

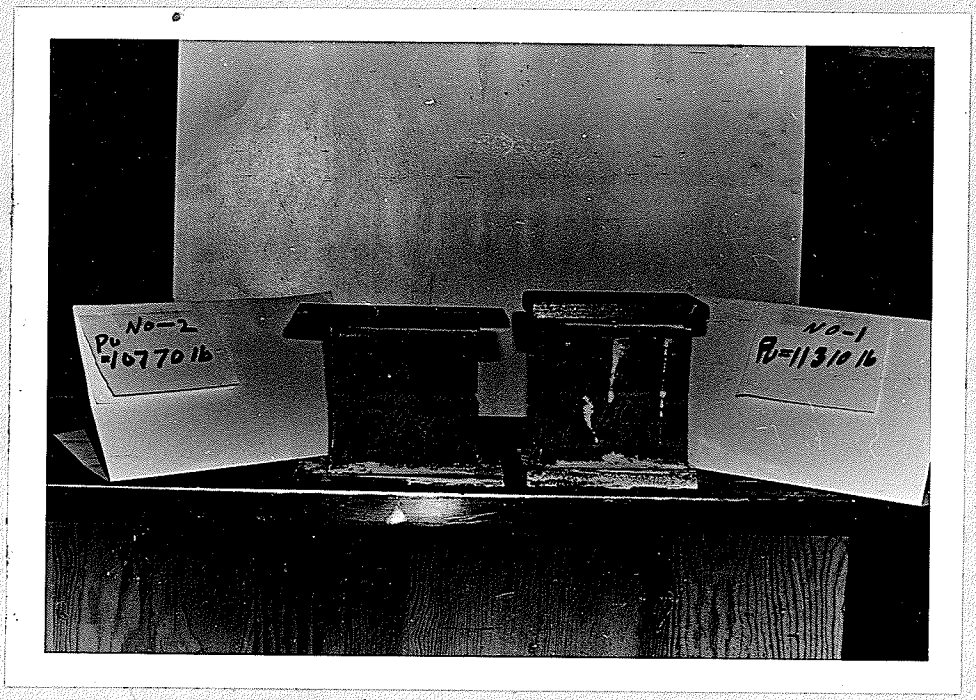


Figure 62 - Prisms After Compression Tests.

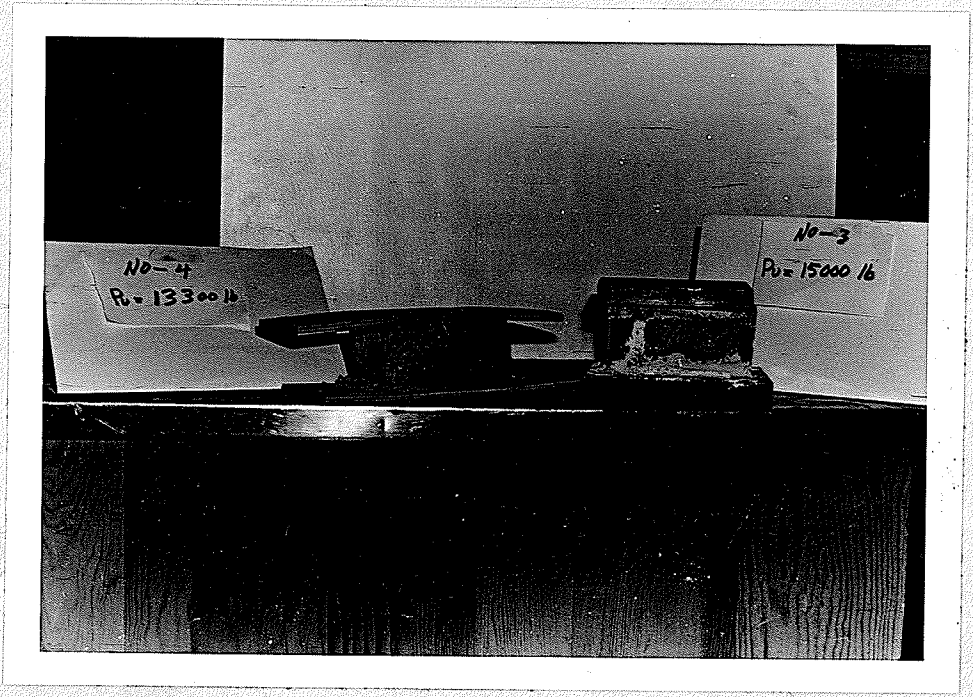


Figure 63 - Single Blocks After Compression Test.

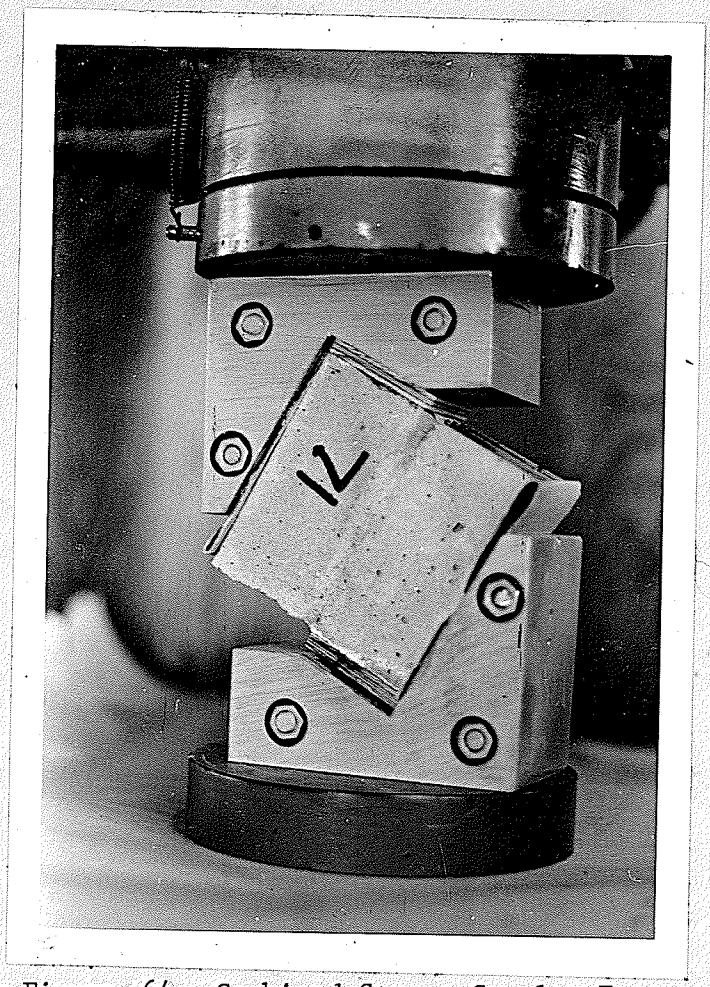


Figure 64 - Combined Stress Couplet Test.

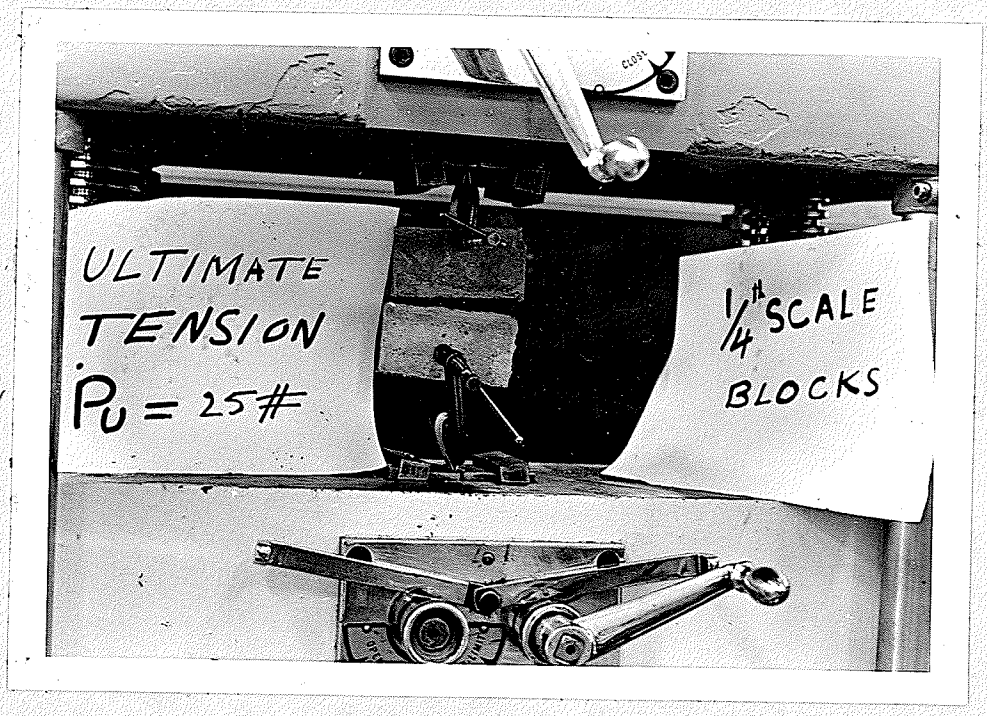


Figure 65 - Direct Tension Test.

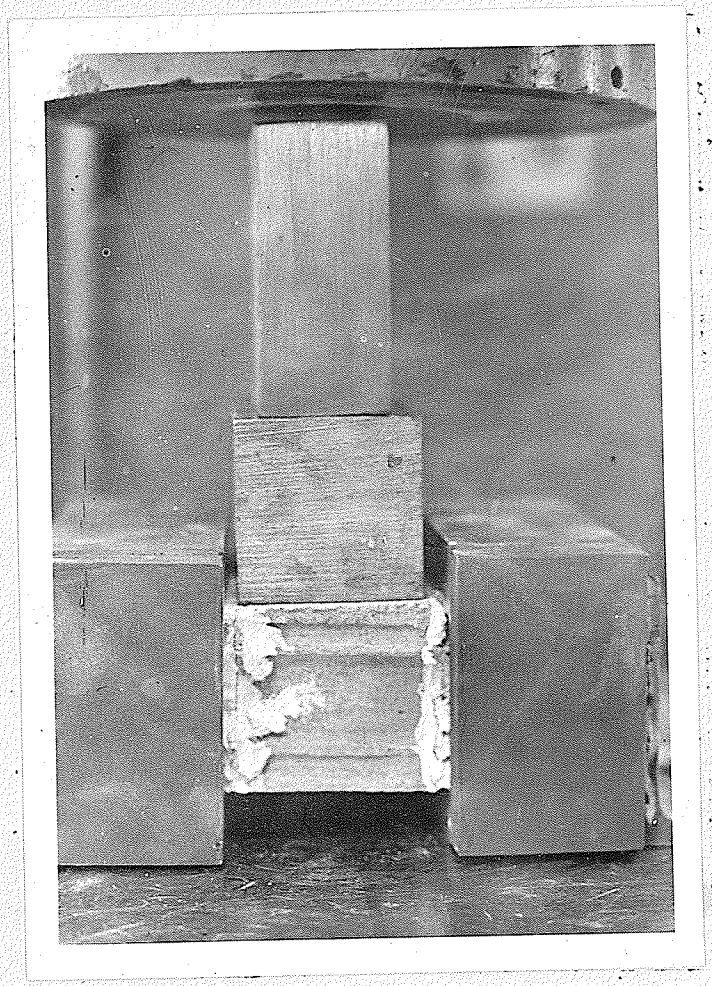


Figure 66 - Direct Shear Test.

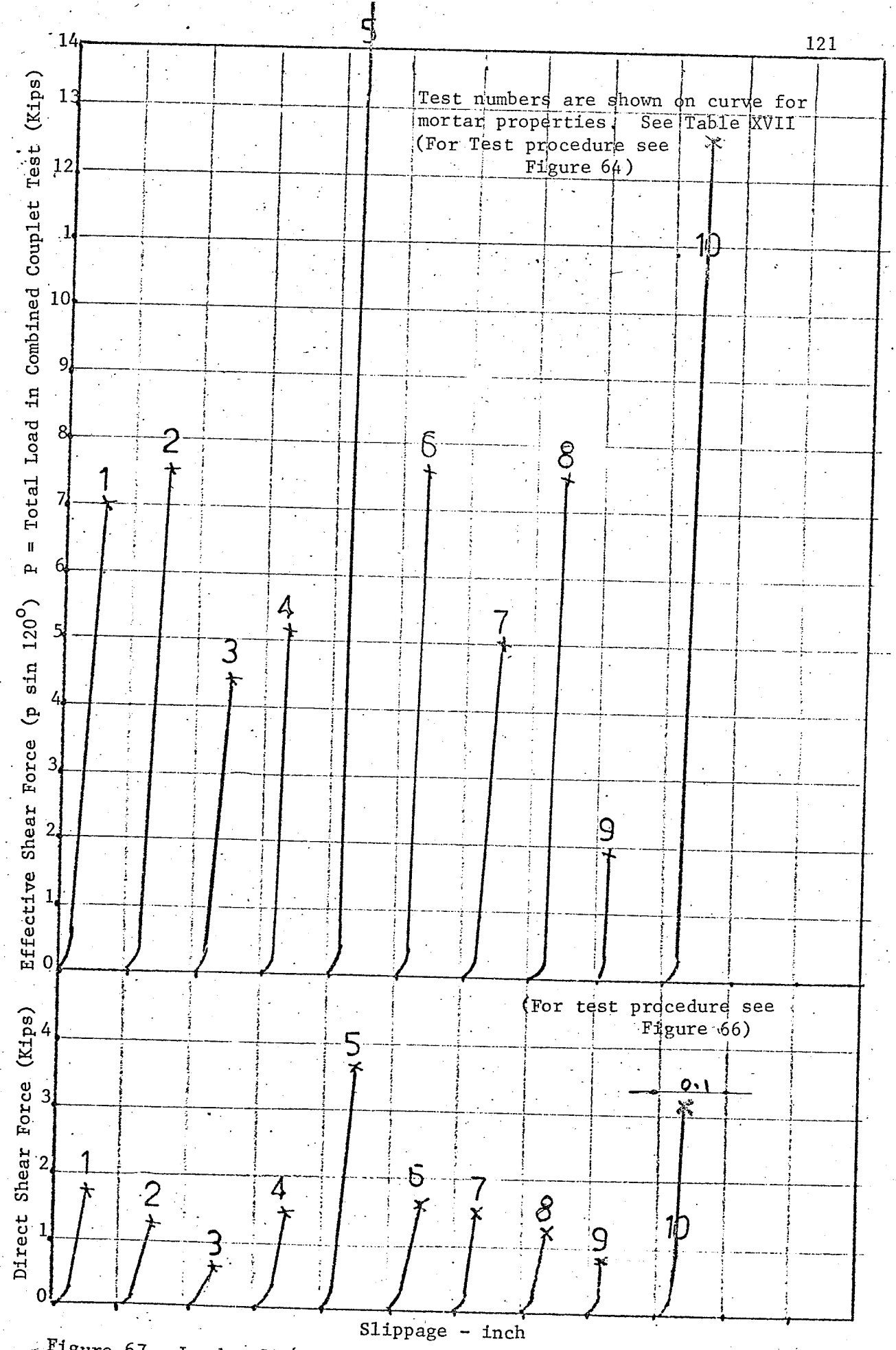


Figure 67 - Load - Slippage Curves from Direct Shear Test and Combined Couplet Tests on Ten Composite Masonry Units with Different Mortar Properties.

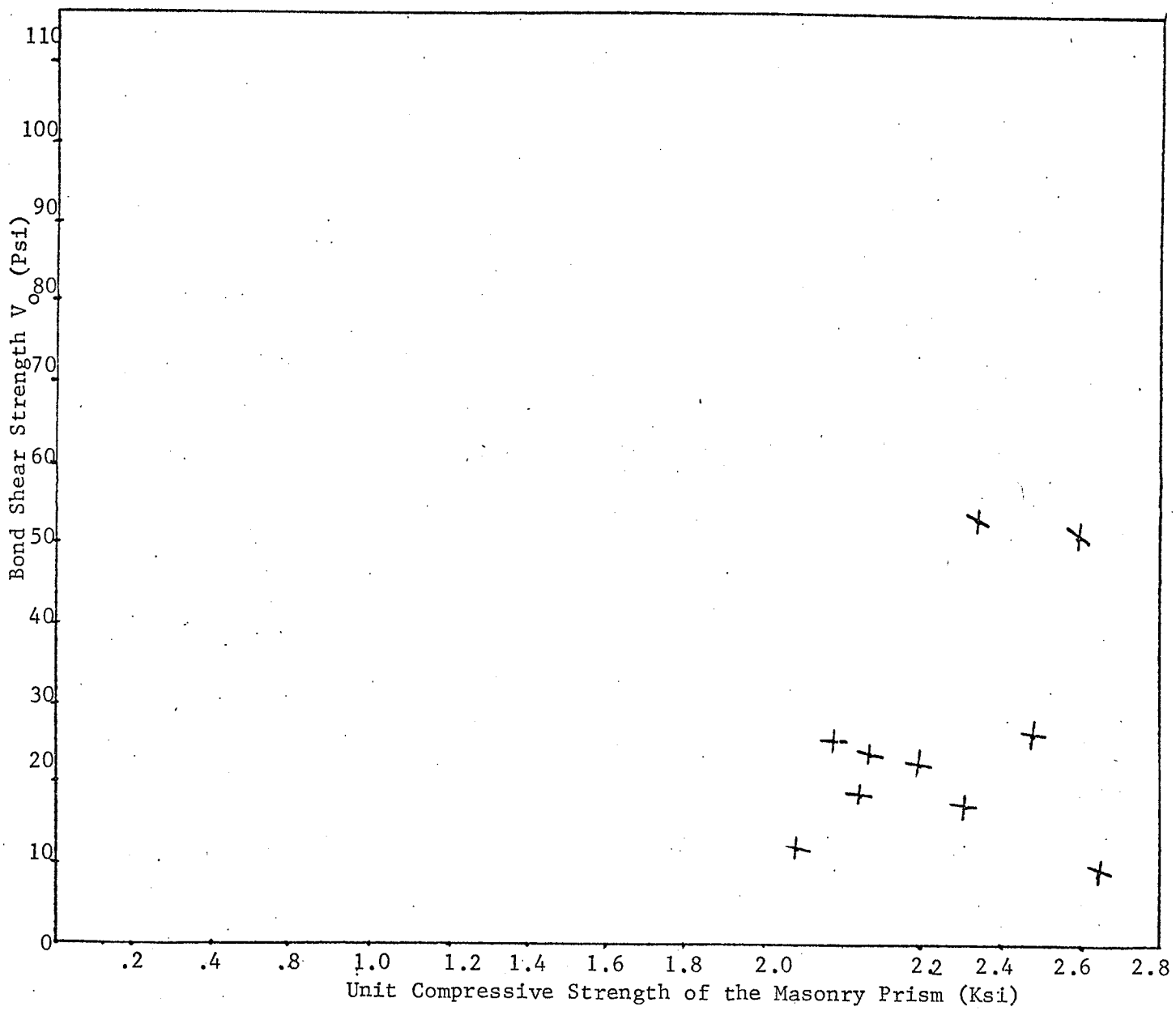


Figure 68 - Relation Between Prism Compressive Strength and Bond Shear Strength
(For data see Table VI)

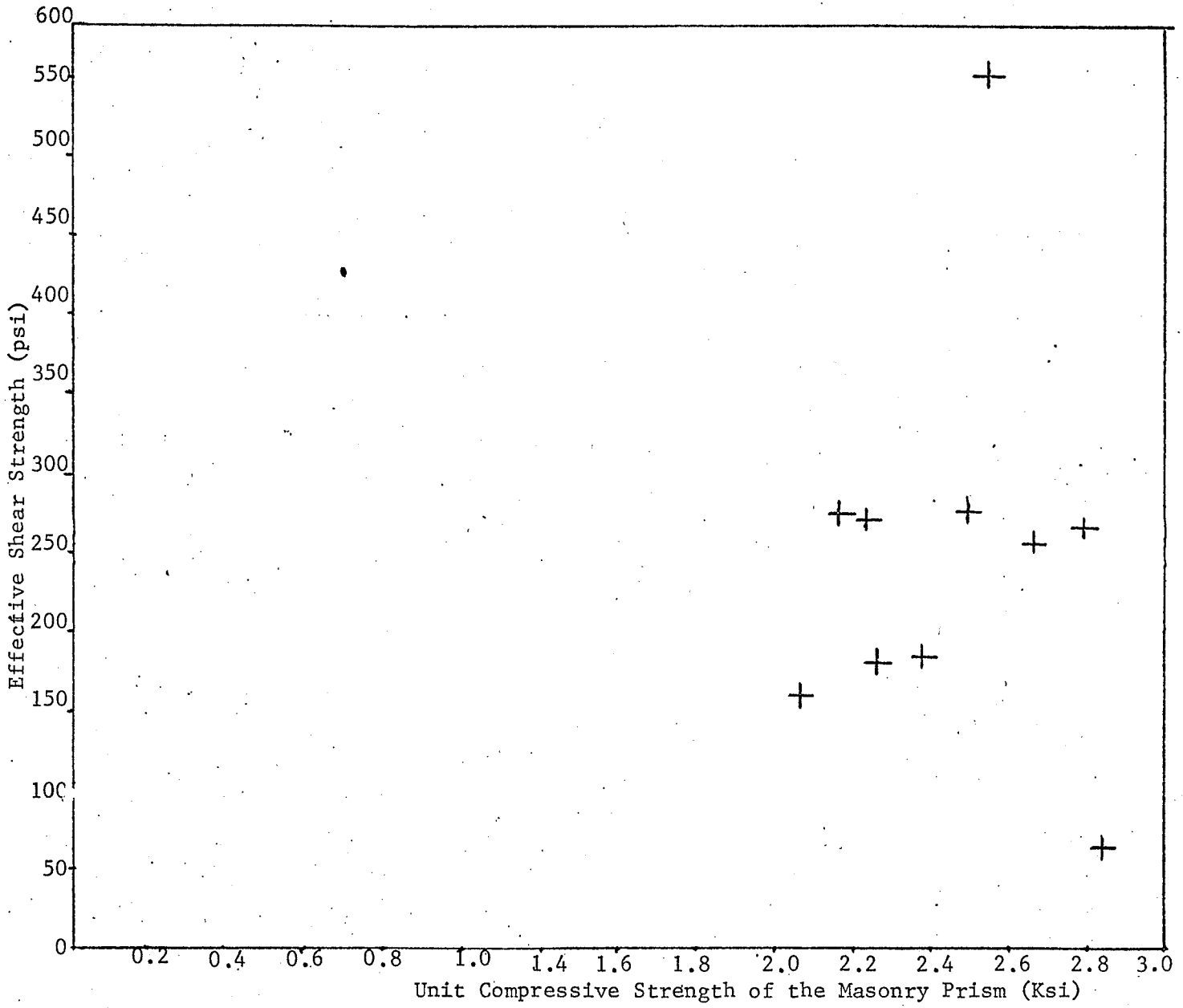


Figure 69 - Relation Between Prism Compressive Strength and Effective Shear Strength
(For data see Table VI)

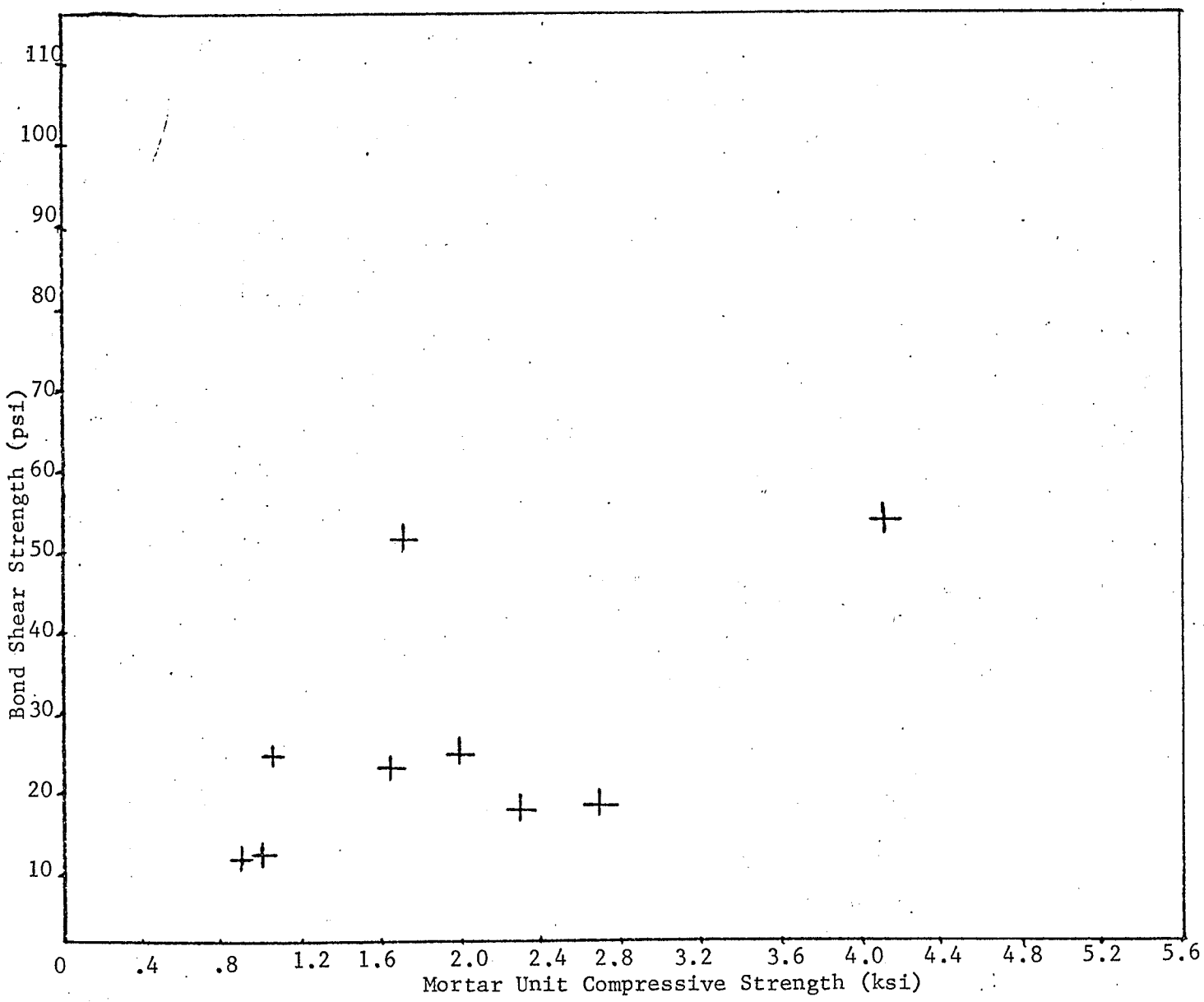


Figure 70 - Relation Between Mortar Compressive Strength and Bond Shear Strength

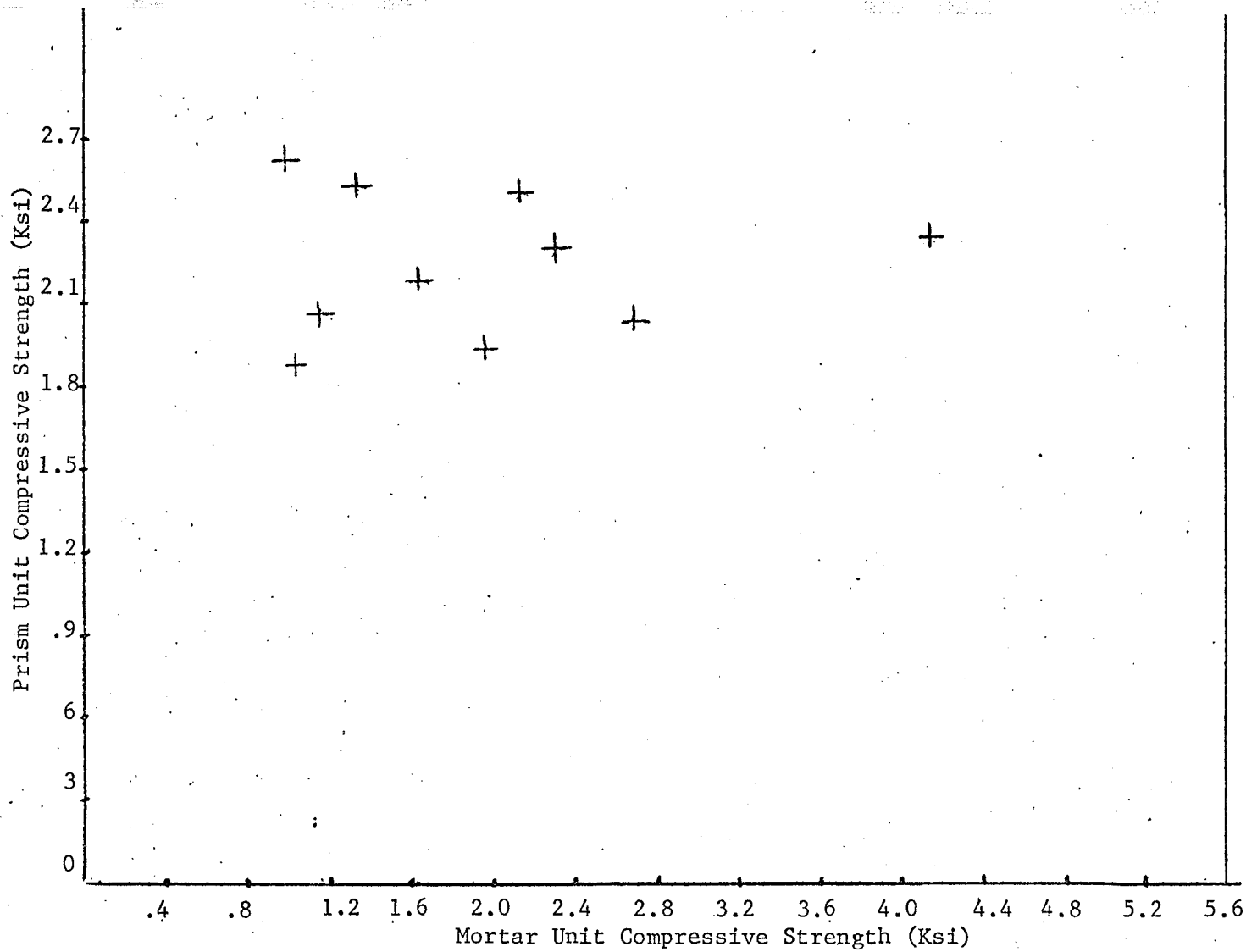


Figure 71 - Relation Between Mortar and Prism Unit Compressive Strength
(For data see Table VI)

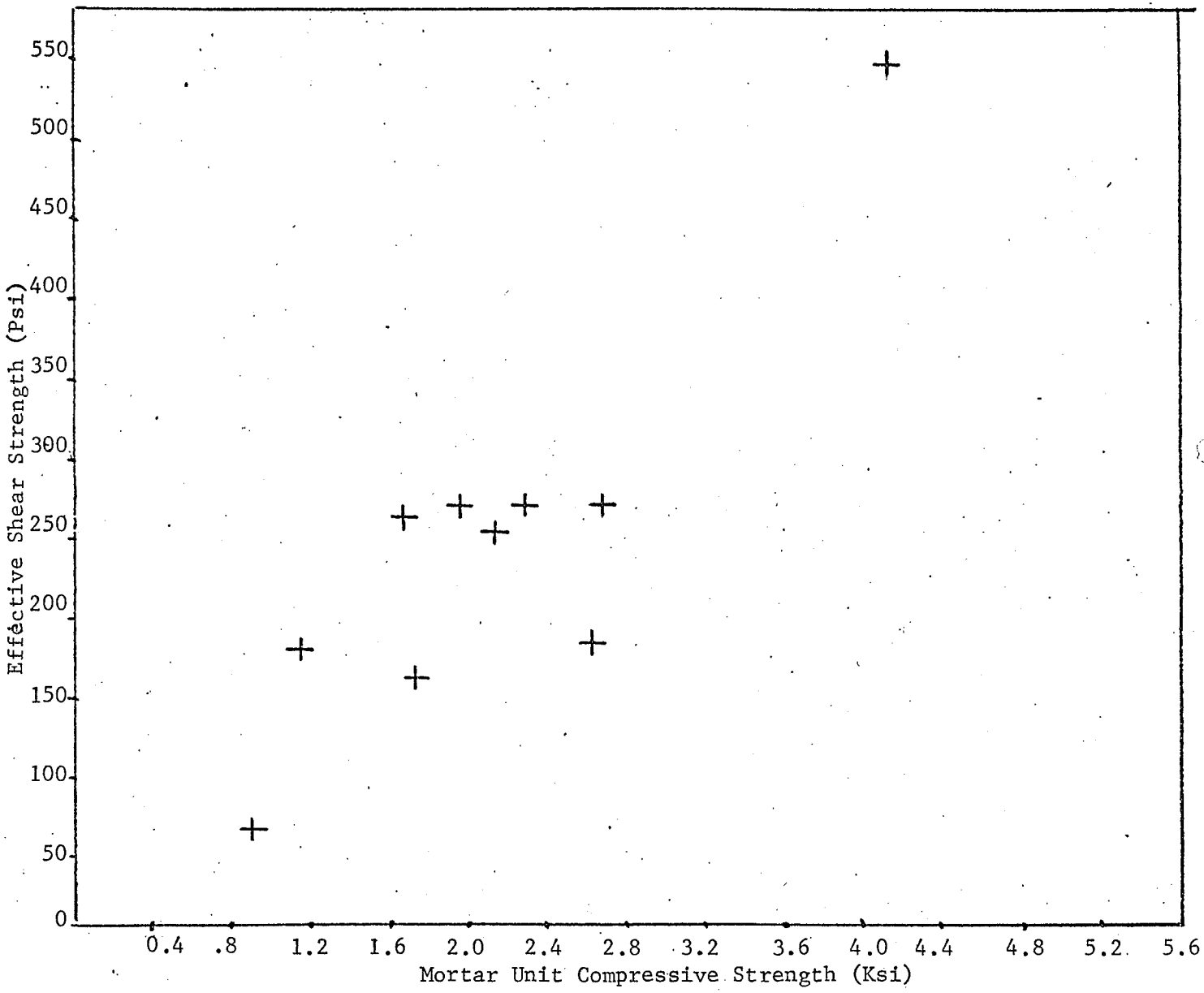
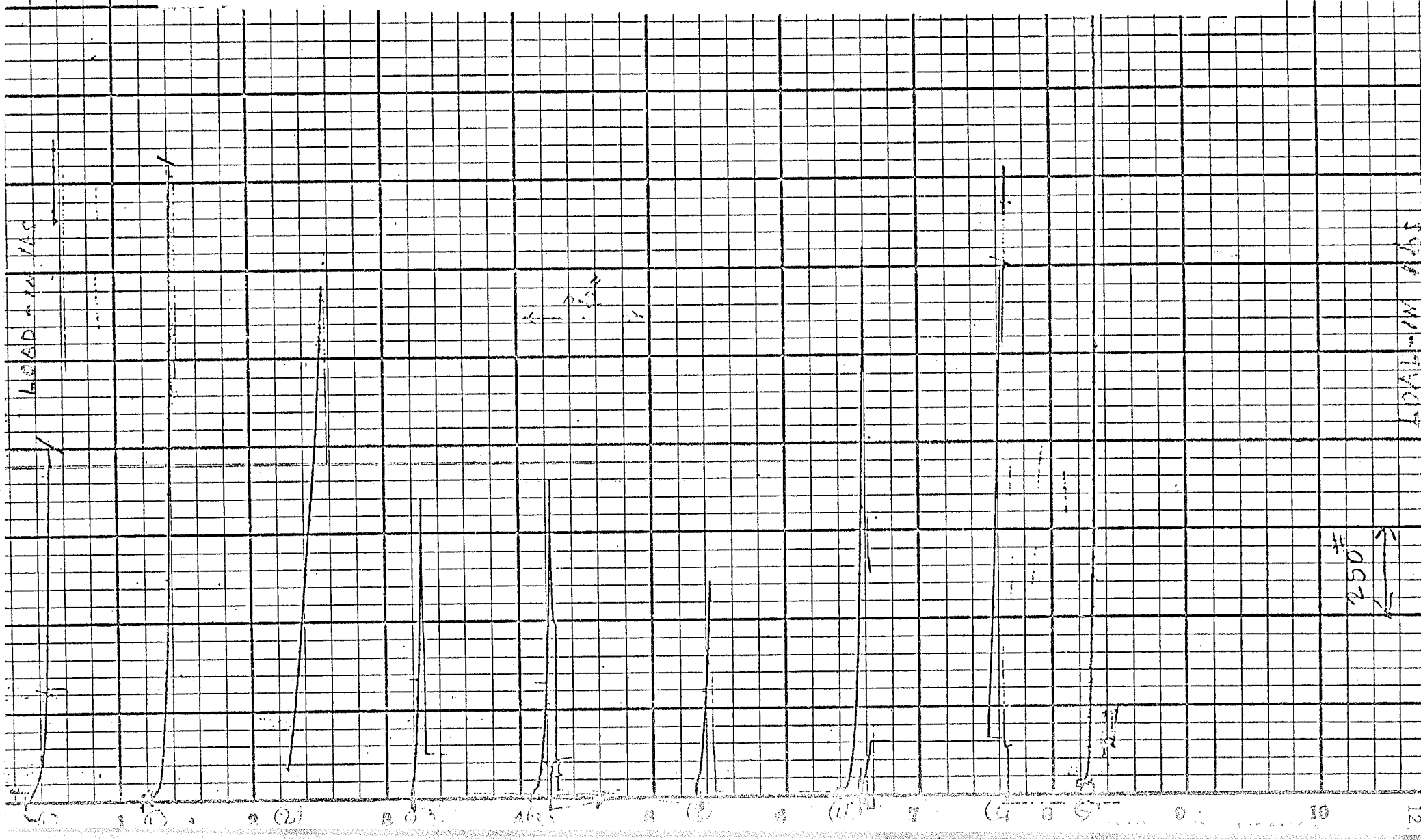


Figure 72 - Relation Between Mortar Compressive Strength and Effective Shear Strength
(For data see Table VI)

Figure 73 - Load-Slippage Curves for Direct Shear Test
From Riehle Testing Machine



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