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**CYCLIC LOADING TESTS OF  
MASONRY SINGLE PIERS  
VOLUME 3 – HEIGHT TO WIDTH RATIO OF 0.5**

by

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Report to:

National Science Foundation

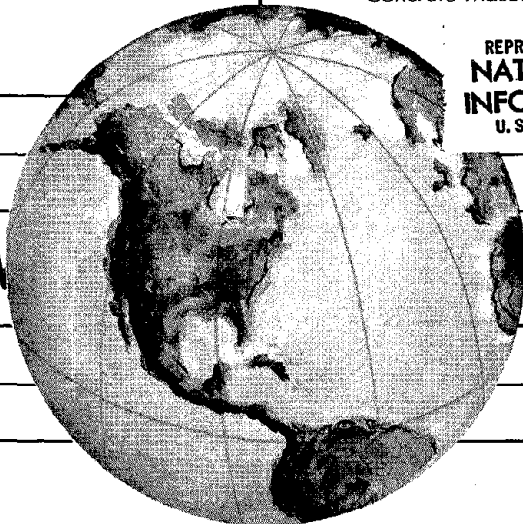
Masonry Institute of America

Western States Clay Products Association

and the

Concrete Masonry Association of California and Nevada

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## 1. INTRODUCTION

### 1.1 The Multistory Masonry Building Research Program

A multistory masonry building research program was initiated at the Earthquake Engineering Research Center in September 1972, and has continued for the past six years. After an extensive review of the literature [5,6]\* dealing with resistance of masonry to earthquakes, it was concluded that shear walls perforated by numerous window openings (Fig. 1.1) were the components of multistory masonry buildings most frequently damaged in past earthquakes, and it was decided that an experimental study of the seismic behavior of such components was necessary.

Two types of structural components can be identified in the shear wall of Fig. 1.1, the piers and the spandrel beams. In order to study the pier behavior, a test fixture was designed to subject typical full-scale double pier specimens to combined static vertical (gravity) and cyclic lateral (seismic) loads (Fig. 1.2). The results obtained from seventeen such specimens have been reported by Mayes et al [8,9]. These results show significant variations in the pier behavior with the various test parameters including the type of grouting, types of reinforcement and the rate of loading. The results were not conclusive and demonstrated the need for more extensive tests to establish definitive parametric relationships.

The cost of the double pier tests, both in money and time, precluded carrying out extensive parametric variations with the double pier test setup and, consequently, a single pier test system was designed

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\*References are arranged in alphabetical order of the authors' names, and are listed at the end of the text.

which greatly simplified the investigation (Fig. 1.3). A series of eighty single pier tests was planned, which included the following test parameters: type of masonry construction, height to width ratio of the piers, type of grouting, and amount and distribution of both vertical and horizontal steel reinforcement. The present report deals with the experimental results of specimens with a height to width ratio of 0.5.

### 1.2 Objectives and Scope of the Single Pier Test Program

In determining the strength of masonry piers and panels, the first step is to evaluate the mode of failure. Because most failures in past earthquakes have been characterized by diagonal cracks, many research programs have concentrated on this type of failure mechanism. Test techniques used by Blume<sup>[1]</sup>, Greenley and Cattaneo<sup>[3]</sup> and others induce the diagonal tension or shear mode of failure. Scrivener<sup>[15]</sup>, Meli<sup>[11]</sup>, Williams<sup>[16]</sup> and Priestley and Bridgeman<sup>[13]</sup>, however, recognized that there are two possible modes of failure for cantilever piers. In addition to the shear or diagonal tension mode they recognized that, for certain piers, a flexural failure could occur. This mechanism is characterized by yielding of the tension steel of the wall, followed by a secondary failure at the compressive toe, with associated buckling of the reinforcement once confinement is lost. Meli<sup>[11]</sup> described the flexural failure as similar to that of an under-reinforced concrete beam; i.e., extensive flexural cracking and strength limited by yielding of the reinforcement, with failure finally due either to crushing of the compressive corner or to rupture of the extreme bars.

Because the double pier tests were the first fixed ended piers to be tested cyclically, the objective of those tests was to determine the effect of various parameters and compare the results with those already

known for cantilever piers. Both the shear and flexural modes of failure were included in that investigation.

One of the main objectives of the single pier test program was to investigate thoroughly the effects of different parameters on the behavior shown in the shear mode of failure. It was evident from the double pier test program that the flexural mode of failure of a fixed ended pier has desirable inelastic characteristics, although these are not as desirable as those obtained by Priestley<sup>[14]</sup> for cantilever piers. Furthermore, it was recognized that for fixed ended piers, with height to width ratios commonly found in multistory buildings, the amount of horizontal reinforcement required to induce a flexural mode of failure is substantially greater than that required by current codes. Therefore, it was decided to investigate the effects of lesser amounts of horizontal reinforcement on the shear mode of failure to determine if desirable inelastic behavior could be obtained.

The eighteen tests reported herein are a part of a total program of eighty single pier tests; a matrix characterizing the first sixty-three tests is shown in Table 1.1. The parameters for the remaining seventeen tests will be selected after an evaluation of these sixty-three. The test parameters, other than the type of construction and height to width ratio, include the amount of reinforcement and the effect of partial grouting. Hollow concrete block piers having height to width ratio of 2 were not included in the single pier test program because such piers were investigated in the original double pier tests.

This report presents the results for piers with a height to width ratio of 0.5, of which six tests were performed on hollow concrete block specimens (HCBL), six on hollow clay brick specimens (HCBR) and six on double wythe grouted core clay brick specimens (CBRC). Previous

reports<sup>[4,2]</sup> presented the results obtained from piers with height to width ratios of 2 and 1. The results from the series of seventeen specimens which will complete the proposed research program will also be presented in a separate report. The organization of the present volume is similar to the two previous ones<sup>[4,2]</sup>. The general background of the single pier test program has been included in this report in order to make it as self-contained as possible.

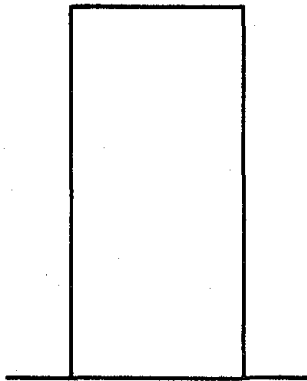


TABLE 1.1

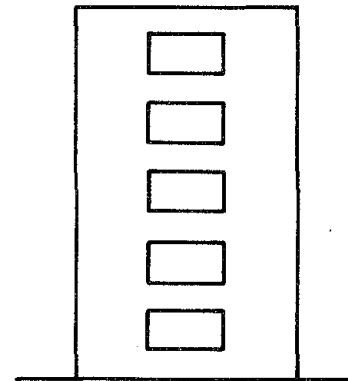
SINGLE PIER TEST PROGRAM\*  
(Number of test specimens)

TYPE OF MASONRY HEIGHT TO WIDTH RATIO	HOLLOW CLAY BRICK (HCBR)	DOUBLE WYTHE GROUTED CORE CLAY BRICK (CBRC)	HOLLOW CONCRETE BLOCK (HCBL)	TOTAL NUMBER
2 : 1	9	5	0	14
1 : 1	13	7	11	31
1 : 2	6	6	6	18

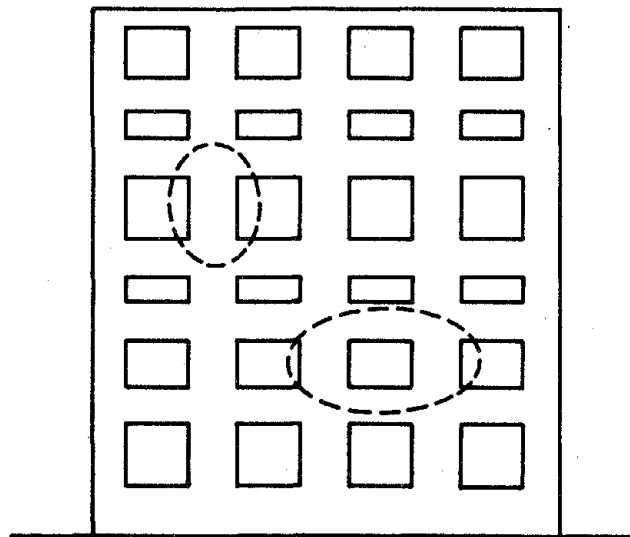
\* Last 17 tests to be decided after this phase is completed



VERTICAL  
CANTILEVER  
SHEAR WALL



COUPLED  
SHEAR WALL



PERFORATED SHEAR WALL

FIG. 1.1 TYPICAL SHEAR WALLS

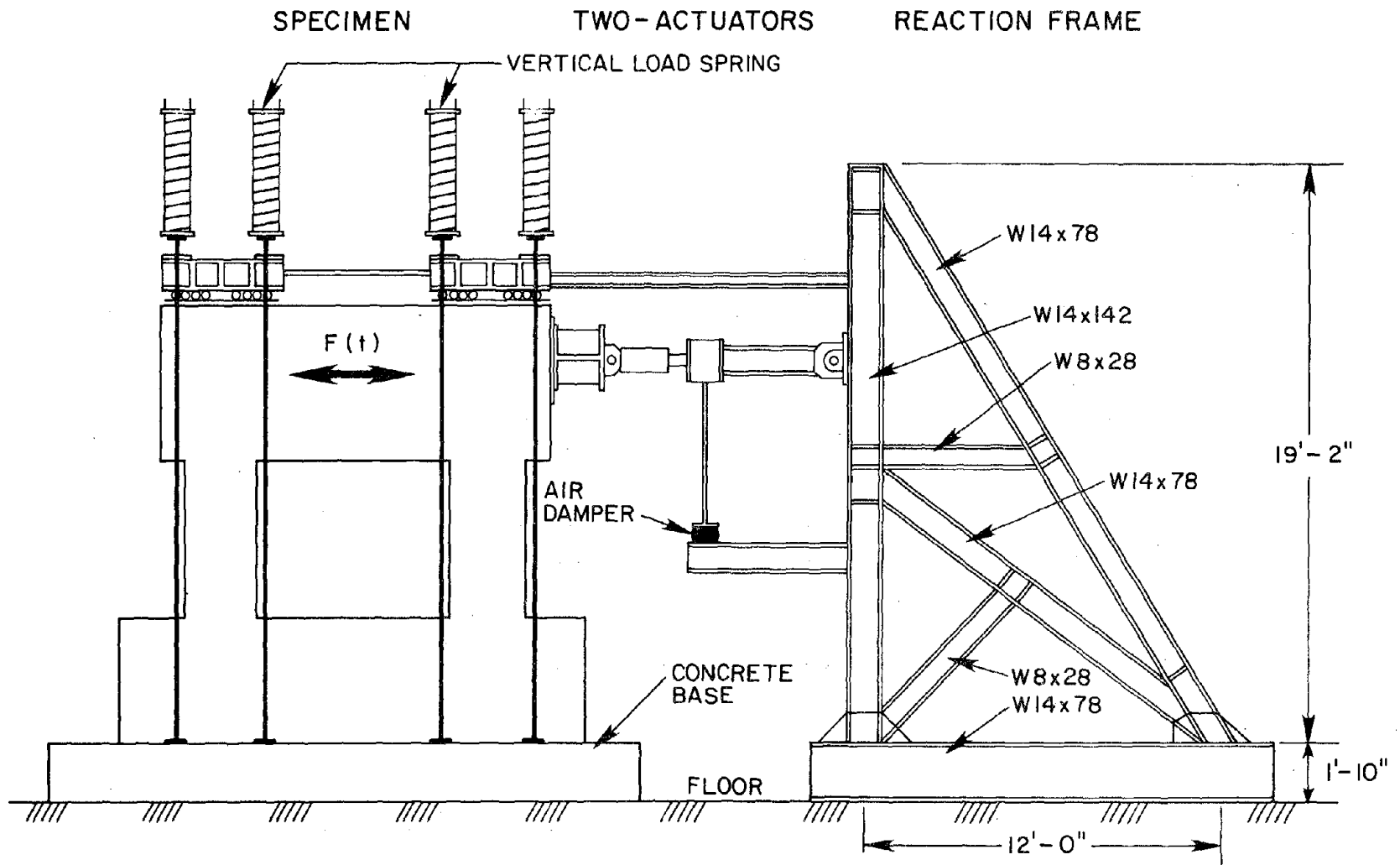


FIG. 1.2 DOUBLE PIER TEST SETUP

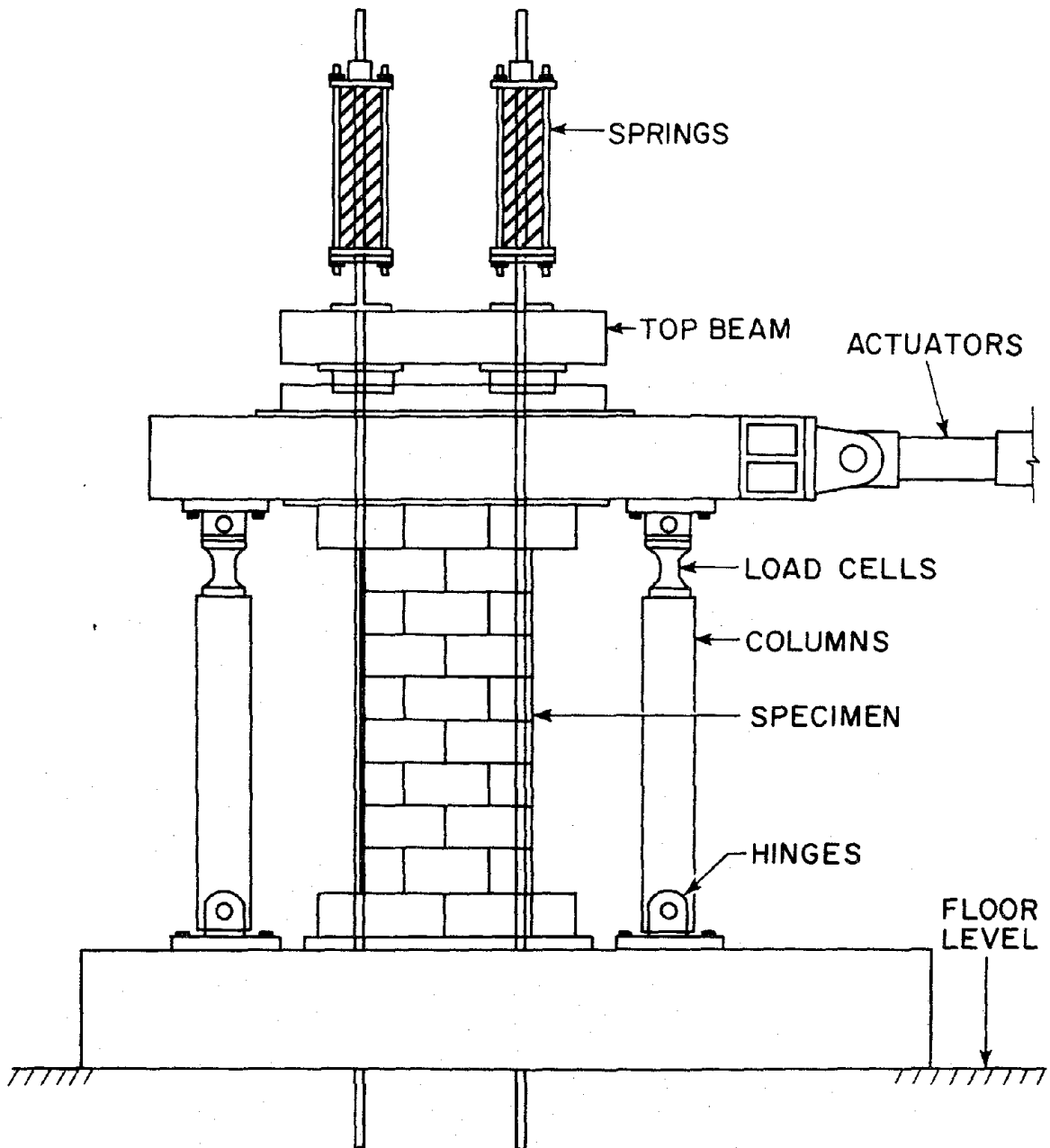


FIG. 1.3 SINGLE PIER TEST SETUP

## 2. TEST SPECIMENS

### 2.1 Design and Construction of Specimens

The overall dimensions of the test specimens discussed here are shown in Fig. 2.1. The height of all eighteen piers was 40 inches. The width was 80 inches for the hollow concrete block piers (HCBL) and 78 inches for the hollow clay brick (HCBR) and double wythe grouted clay brick (CBRC) piers. The thickness was 7-5/8 inches for the HCBL piers, 7-3/8 inches for the HCBR piers and 10 inches for the CBRC piers.

The HCBL panels were constructed from standard two-core hollow concrete blocks, nominally 8 inches wide x 8 inches high x 16 inches long, as shown in Fig. 2.3(a). The cored area of each block is approximately 50.6 square inches and the ratio of net to gross area is 58%.

The HCBR piers were constructed from standard two-core hollow clay bricks, nominally 8 inches wide x 4 inches high x 12 inches long as shown in Fig. 2.3(b). The cored area of each brick is approximately 57.4 square inches and the ratio of net to gross area is 67%.

The CBRC piers were constructed from two wythes of "solid" clay bricks nominally 4 inches wide x 4 inches high x 12 inches long as shown in Fig. 2.3(c). The grouted space between the wythes was 3 inches wide and was filled after the steel reinforcement had been placed in position. The bricks have a core (hollow) area slightly less than 25% of the gross area. The Uniform Building Code definition of a "solid brick" is one with 25% or less coring.

The piers were constructed on 0.75 inch thick steel plates as shown in Fig. 2.2. A similar plate was added on top of the pier after the grout was poured. Both plates had holes to permit anchorage of the vertical steel reinforcement and keys to provide an adequate shear

transfer between the masonry pier and the steel plate. The plates also had welded bolts and holes to anchor the pier to the test rig.

All the eighteen piers were fully grouted. The series of tests was planned to determine the effect of the quantity of horizontal steel reinforcement on the strength and deformation properties of the piers, as shown in the test program (Table 2.1). Details of the reinforcing bar arrangements are shown in Fig. 2.4(a) for the HCBL piers and in Fig. 2.4(b) for the HCBR piers and the CBRC piers. The actual position of the vertical reinforcement is indicated in Fig. 2.1. When horizontal reinforcement was used, the bars were evenly distributed over the height of the pier.

## 2.2 Material Properties

Table 2.2 shows the mechanical properties of the materials used in the construction of the test specimens. The specimens used to determine the material properties are shown in Fig. 2.3(a), (b) and (c).

The tests of the single masonry units followed the ASTM C67-73 Specification<sup>[10]</sup> and were based on three samples for each test.

The joint mortar was specified as standard ASTM type M (i.e., 1 Cement: 1/4 Lime: 2 1/4-3 Sand, by volume), with a minimum compressive strength of 2500 psi at 28 days. The grout was specified as 1 Cement: 3 Sand: 2 G, where G refers to 10 mm maximum size local gravel. Because the specimens were not constructed or grouted at the same time, the mortar and grout strength varied according to normal workmanship. A minimum of three samples of both mortar and grout was taken from each batch used during construction.

ASTM A615 Grade 60 steel was specified for both the vertical and horizontal steel reinforcement. Three samples of each bar size were tested to determine the properties listed in Table 2.2.

Six prisms for uniaxial compression tests and three square panels for diagonal tension tests were constructed from the same mortar and grout used in each set of wall panels. Three of the six prisms had a height to thickness ratio of 5 and the other three had a height to thickness ratio of 2. All prism tests were performed at a loading rate of 100,000 lb/min. (Fig. 2.5). The compressive strengths are shown in Table 2.2.

The square panels were tested as shown in Fig. 2.6 at a loading rate of 20,000 lb/min. The ultimate load for the square panel tests is also shown in Table 2.2. The square panels corresponding to the HCBR piers do not appear in Table 2.2 and Fig. 2.3(b) because they were accidentally broken before the test.

The mortar, grout, prism and square panel samples were cured under the same normal atmospheric conditions as the piers; also the prism and square panel tests were performed during the tests of the corresponding piers.

TABLE 2.1

TEST PROGRAM

Pier General Characteristics	Specimen Designation	Test Frequency (cps)	Grouting Full (F) Solid (S)	Reinforcing Steel	
				Vertical	Horizontal
Masonry type: Hollow Concrete Block	HCBL-12-1	0.02	F	3#7	No
Pier height: H = 40 in	-2	0.02	F	3#7	1#5
Pier width: D = 80 in	-3	0.02	F	3#7	2#5
Pier thickness: 7.625 in	-4	0.02	F	3#7	3#5
Gross section area: 610 in <sup>2</sup>	-5	0.02	F	3#7	4#5
Bearing load: 32 kip	-6	0.02	F	3#7	4#6
Bearing stress: 52 psi					
Masonry type: Hollow Clay Brick	HCBR-12-1	0.02	F	3#7	No
Pier height: H = 40 in	-2	0.02	F	3#7	1#6
Pier width: D = 78 in	-3	0.02	F	3#7	2#6
Pier thickness: 7.375 in	-4	0.02	F	3#7	3#6
Gross section area: 575.25 in <sup>2</sup>	-5	0.02	F	3#7	4#6
Bearing load: 32 kip	-6	0.02	F	3#7	5#7
Bearing stress: 56 psi					
Masonry type: Double Wythe Grouted Core Clay Brick	CBRC-12-1	0.02	S	3#7	No
Pier height: H = 40 in	-2	0.02	S	3#7	1#6
Pier width: D = 78 in	-3	0.02	S	3#7	2#6
Pier thickness: 10 in	-4	0.02	S	3#7	3#6
Gross section area: 780 in <sup>2</sup>	-5	0.02	S	3#7	4#6
Bearing load: 39 kip	-6	0.02	S	3#7	5#7
Bearing stress: 50 psi					



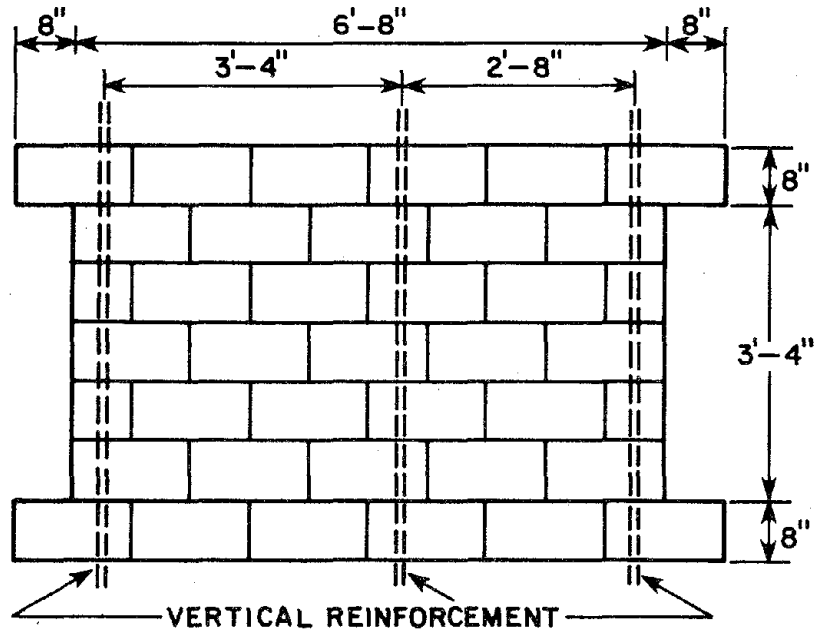
TABLE 2.2

MATERIAL PROPERTIES

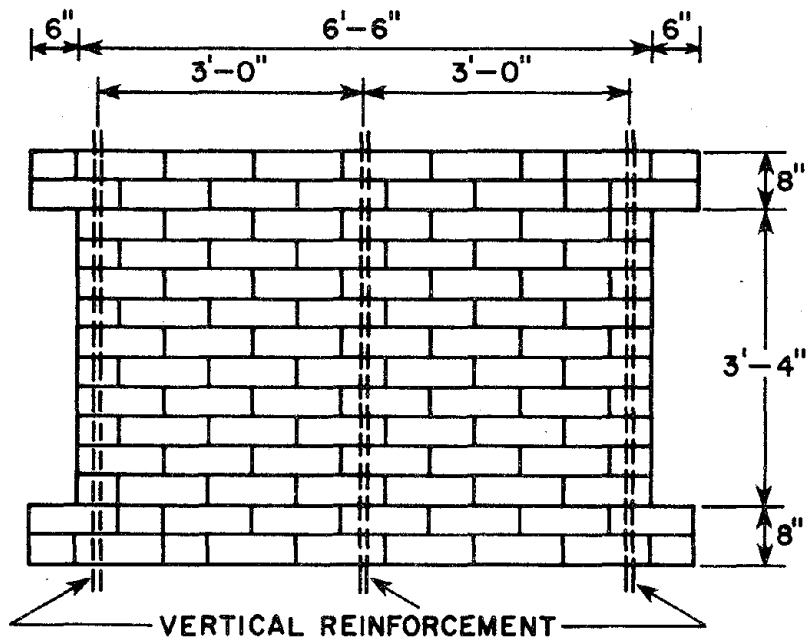
(Average values. Number in parenthesis indicates the standard deviation as percent of average value)

MASONRY	HCBL-12	HCBR-12	CBRC-12
Masonry unit gross compressive strength (psi)	1878 (5%)	5816 (6%)	9422 (4%)
Masonry unit net tensile strength (psi)	221 (14%)	466 (19%)	303 (24%)
Mortar compressive strength (psi)	5530 (26%)	3460 (24%)	4903 (29%)
Grout compressive strength (psi)	3890 (3%)	3890 (3%)	3785 (4%)
Prism (2:1) compressive strength (psi)	3604 (9%)	3589 (7%)	2948 (11%)
Prism (5:1) compressive strength (psi)	2988 (13%)	2838 (15%)	2876 (3%)
Ultimate load of square panel (kip)	155.0 (4%)	—	186.3 (11%)

STEEL REINFORCEMENT	No. 5 bar	No. 6 bar	No. 7 bar
Yield strength (ksi)	69.6 (4%)	67.3 (1%)	80.3 (11%)
Ultimate strength (ksi)	109.8 (4%)	108.6 (1%)	125.4 (9%)
Modulus of elasticity (ksi)	28700	28800	28800
Yield strain (in/in)	0.00243	0.00234	0.00279



### HOLLOW CONCRETE BLOCK PIERS



### HOLLOW CLAY BRICK AND GROUTED CORE BRICK PIERS

FIG. 2.1 PIER DIMENSIONS

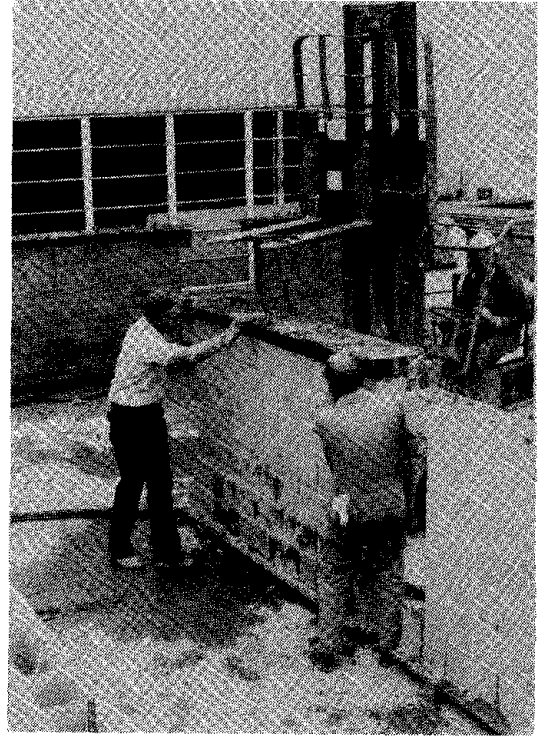
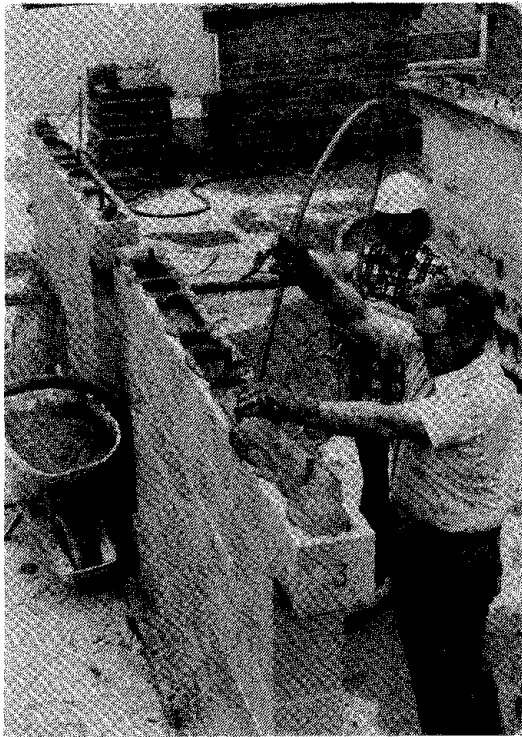
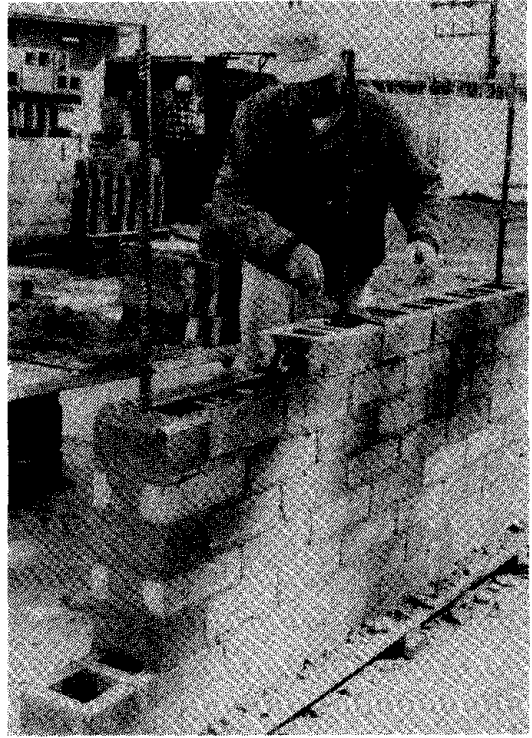
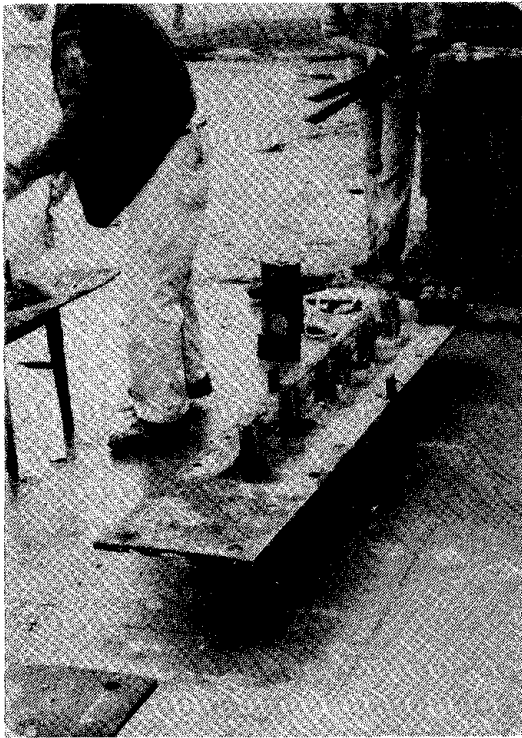
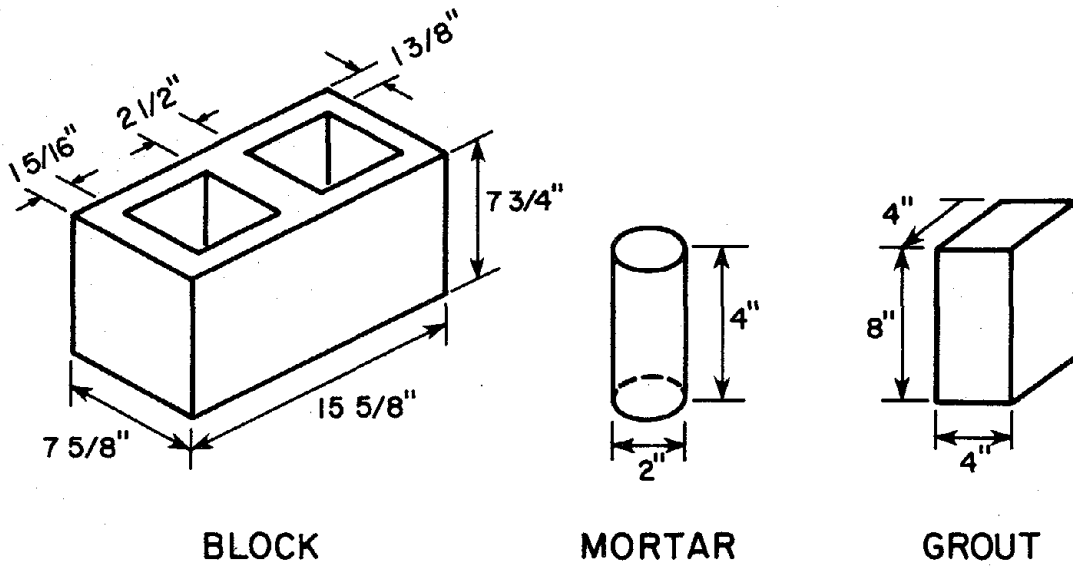


FIG. 2.2 CONSTRUCTION OF TEST SPECIMENS

**BASIC MATERIALS**



**MASONRY SUBASSEMBLAGES**

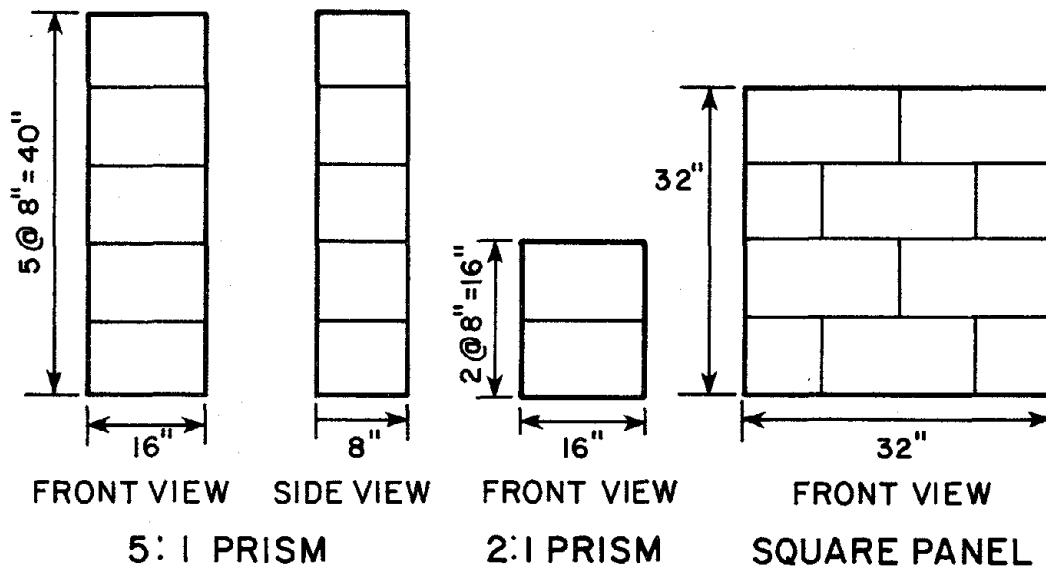
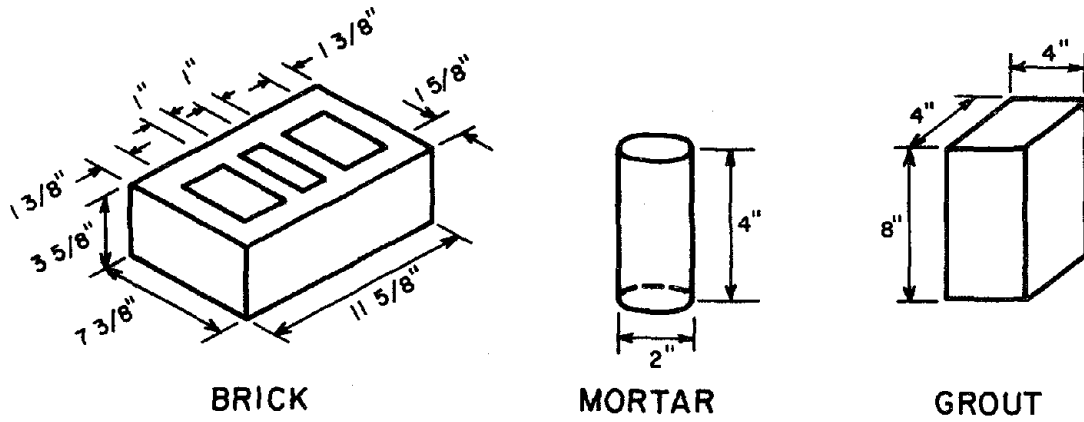


FIG. 2.3(a) SPECIMENS TO DETERMINE MATERIAL PROPERTIES (HCBL)

## BASIC MATERIALS



## MASONRY SUBASSEMBLAGES

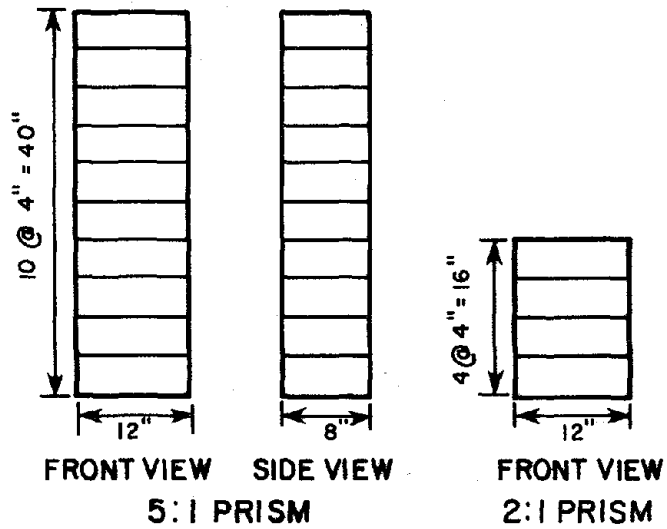
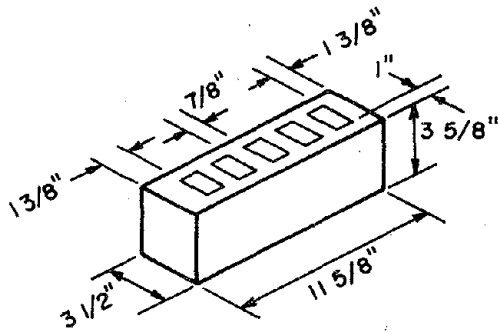
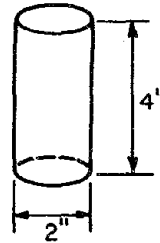


FIG. 2.3(b) SPECIMENS TO DETERMINE MATERIAL PROPERTIES (HCBR)

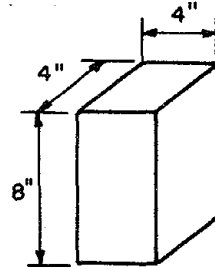
BASIC MATERIALS



BRICK

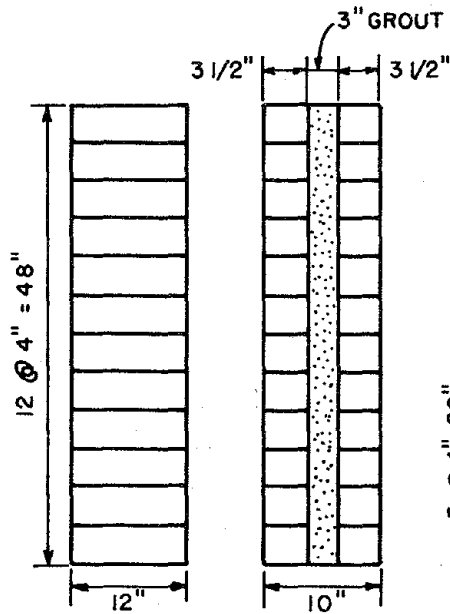


MORTAR

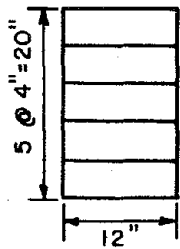


GROUT

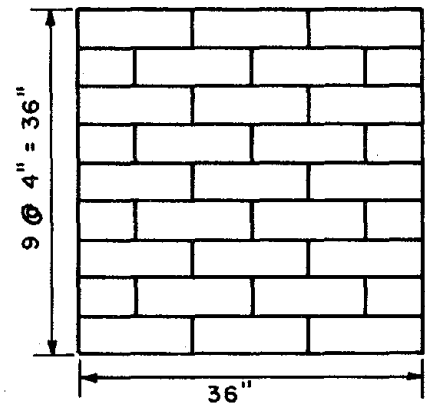
MASONRY SUBASSEMBLAGES



FRONT VIEW SIDE VIEW  
5:1 PRISM

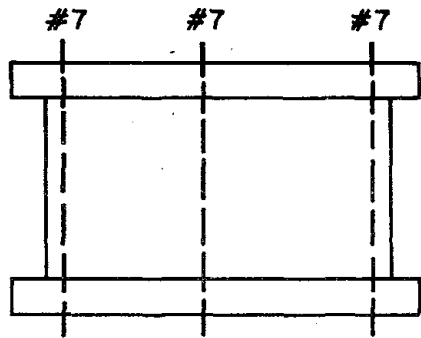


FRONT VIEW  
2:1 PRISM

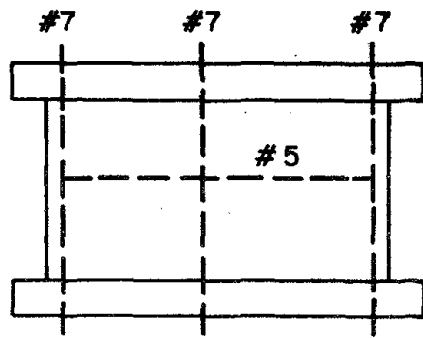


FRONT VIEW  
SQUARE PANEL

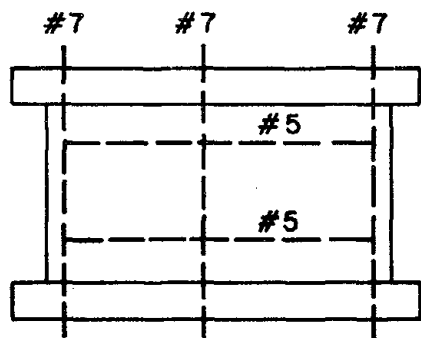
FIG. 2.3(c) SPECIMENS TO DETERMINE MATERIAL PROPERTIES (CBRC)



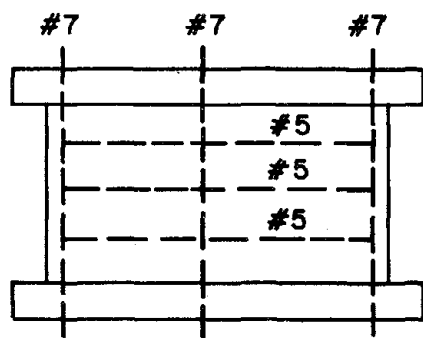
HCBL-12-1 FULL GROUTING



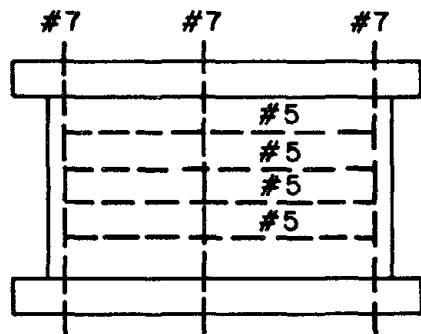
HCBL-12-2 FULL GROUTING



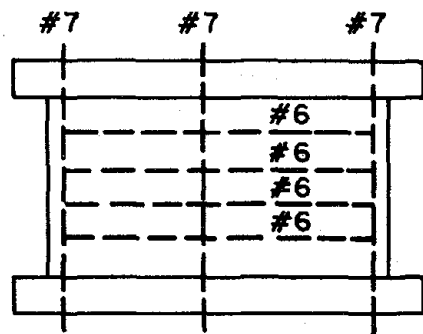
HCBL-12-3 FULL GROUTING



HCBL-12-4 FULL GROUTING

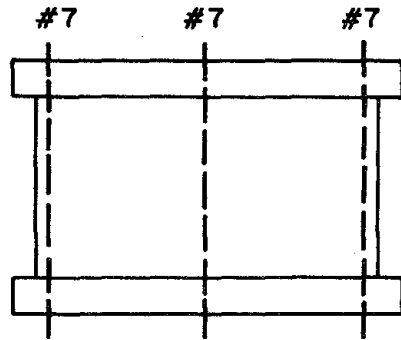


HCBL-12-5 FULL GROUTING

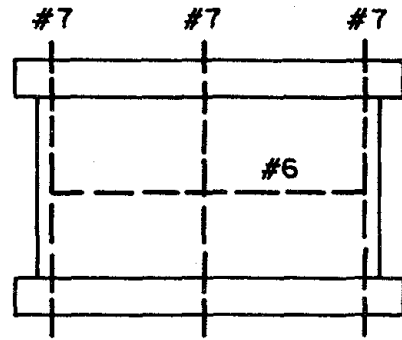


HCBL-12-6 FULL GROUTING

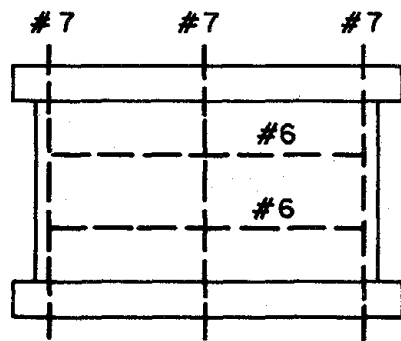
FIG. 2.4(a) REINFORCING STEEL ARRANGEMENTS FOR HOLLOW CONCRETE BLOCK PIERS (HCBL)



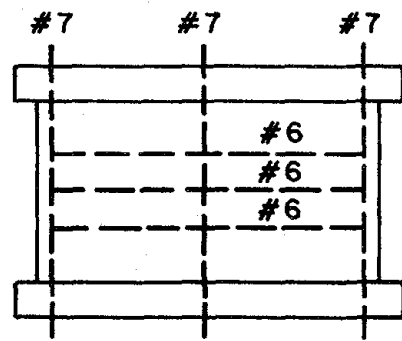
HCBR-12-1 FULL GROUTING  
CBRC-12-1 SOLID GROUTING



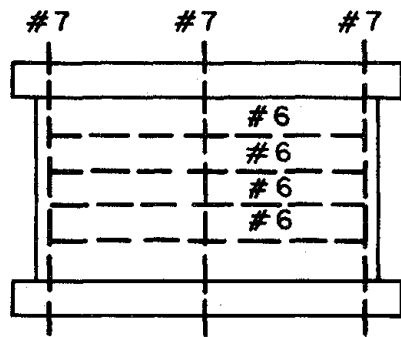
HCBR-12-2 FULL GROUTING  
CBRC-12-2 SOLID GROUTING



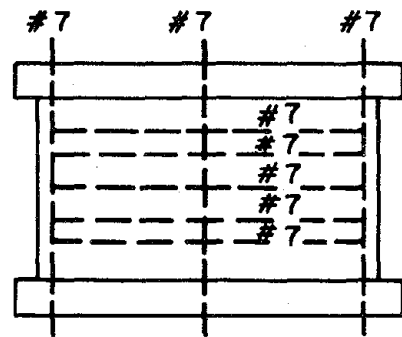
HCBR-12-3 FULL GROUTING  
CBRC-12-3 SOLID GROUTING



HCBR-12-4 FULL GROUTING  
CBRC-12-4 SOLID GROUTING



HCBR-12-5 FULL GROUTING  
CBRC-12-5 SOLID GROUTING



HCBR-12-6 FULL GROUTING  
CBRC-12-6 SOLID GROUTING

FIG. 2.4(b) REINFORCING STEEL ARRANGEMENTS FOR HOLLOW CLAY BRICK AND GROUTED CORE CLAY BRICK PIERS (HCBR AND CBRC)



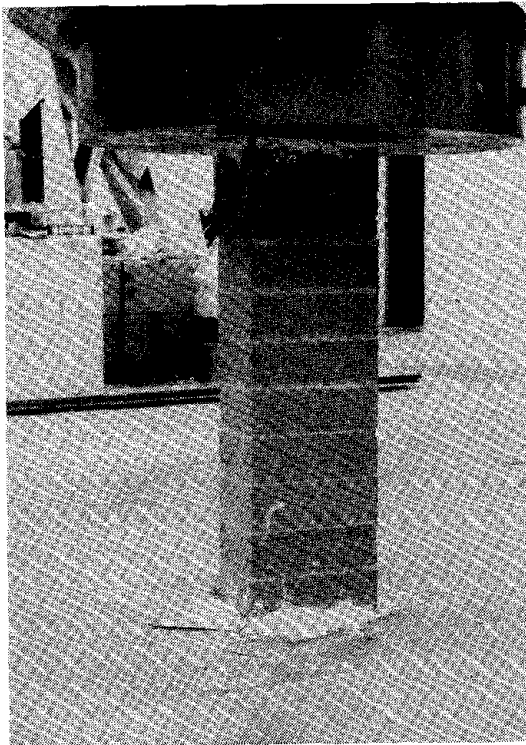
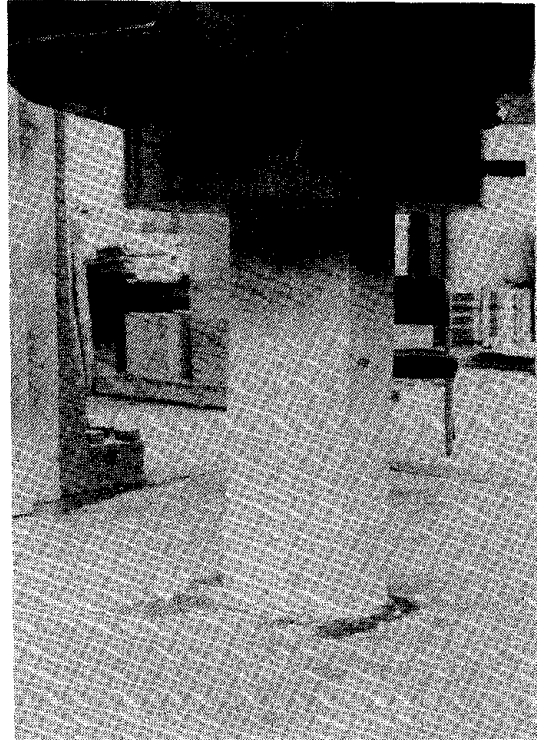


FIG. 2.5 PRISM TEST

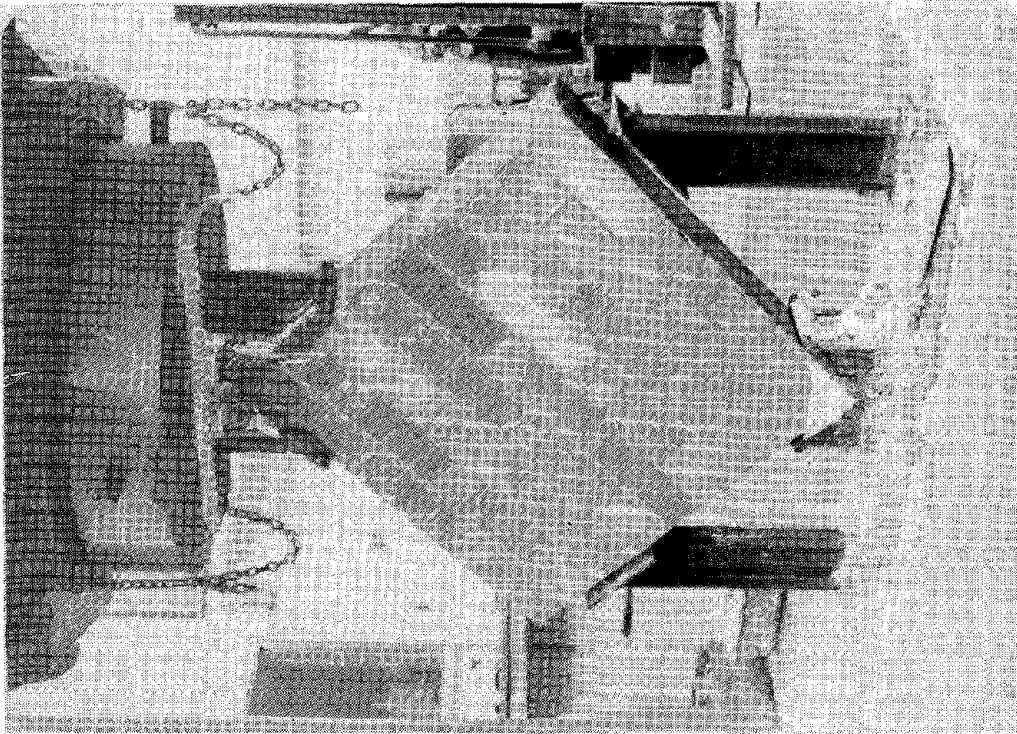
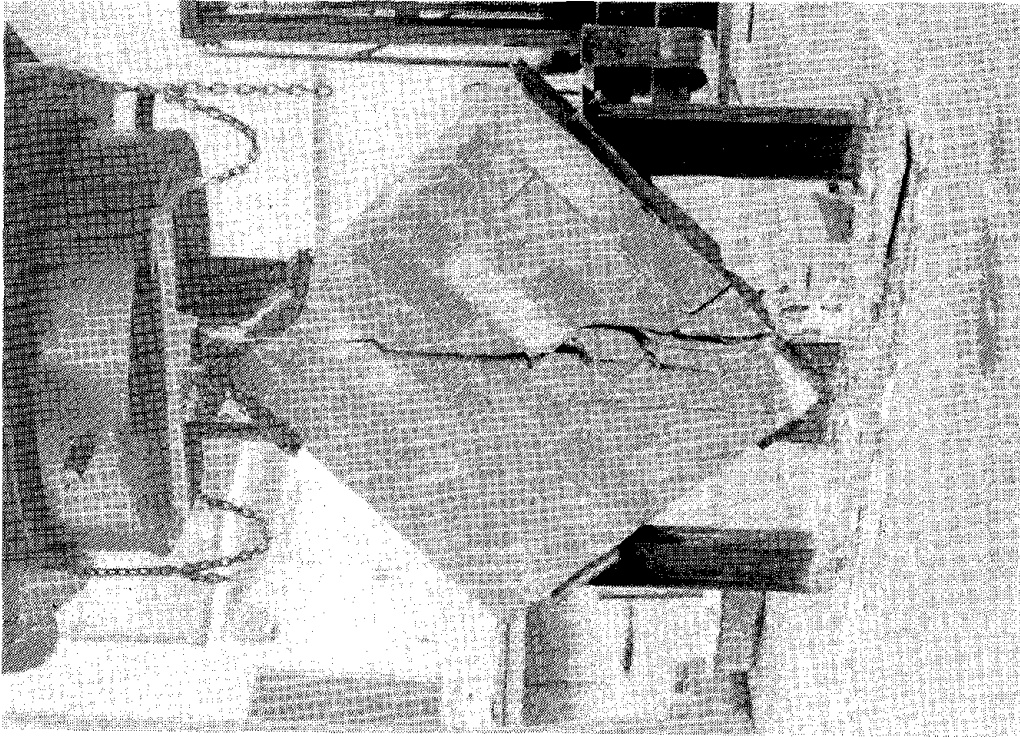


FIG. 2.6 SQUARE PANEL TEST

### 3. TEST EQUIPMENT AND PROCEDURE

#### 3.1 Test Equipment

The test equipment shown in Figs. 3.1 and 3.2 permits lateral loads to be applied in the plane of the piers in a manner similar to which a floor diaphragm would load the piers during earthquake excitation. It consists of two 20 feet high, heavily-braced reaction frames supporting a horizontally acting hydraulic actuator; a mechanism capable of applying vertical bearing loads similar to the gravity loads experienced by the piers in an actual structure; a bottom beam composed of a concrete base and a wide flange steel beam which provides anchorage to the test floor and suitable connection holes to the bottom plate of the specimen; and a top beam fabricated from two wide flange, steel beams as shown in Fig. 3.2. The top and bottom beams simulate the action of the spandrel beams in actual masonry construction; they are connected by two steel columns located 10 feet 7 inches apart, which prevent rotation of the top beam and thus provide approximate fixed-fixed end conditions during the test.

The maximum load which may be developed by the horizontal actuator is 450 kips, using a hydraulic pressure of 3000 psi. Either displacement or load can be controlled with this actuator.

A vertical load up to 160 kips can be applied to the pier through the springs and rollers shown in Fig. 3.2. The Thomson Dual Roundway Bearings connecting the springs to the top of the panel allow the panel to move freely with minimal friction force. The coefficient of friction of bearings is purported to be 0.007.

An additional vertical, compressive load results from the characteristics of this test setup. As significant lateral displacements

are imposed on the top beam by the hydraulic actuator, the constraint provided by the side columns forces the top beam to move in a circular arc. The vertical component of this motion is opposed by the axial stiffness of the pier, resulting in a compressive load being applied to the pier. The significance of this additional, cyclic varying compressive load on the test results is discussed in Chapter 5.

Each pier was constructed on a 0.75 inch thick steel plate and had a similar plate on top, as discussed in Section 2.1. This allowed the piers to be moved into place before each test and bolted to the bottom and top steel beams. Prior to the bolting process, hydrostone was placed between the surfaces of the plates and beam flanges as well as between the top plate and the top brick course of the pier.

### 3.2 Loading Sequence

Each pier was subjected to a series of displacement controlled, in-plane shear loads. The full sequence of loading consisted of sets of three sinusoidal cycles of loading at a specified actuator displacement amplitude. The specified amplitude was gradually increased; the full loading sequence is given in Table 3.1. After each stage, (one set of three sinusoidal displacements at the same amplitude), the walls were visually inspected and the crack pattern identified and photographed. The sinusoidal cycles were applied at a frequency of 0.02 cycles per second throughout the test program.

The test of each pier had a duration of 2-1/2 to 3 hours. The test was usually terminated when the shear strength of the pier had dropped below one third of the maximum shear strength. All of the tests were carried out under a constant primary bearing stress between 50 and 56 psi. Additional cyclic vertical compressive loads were developed

during the test, as described in Section 3.1, and discussed further in the following chapters. All the piers tested were subjected to a maximum input displacement amplitude ranging from 0.50 inch to 0.80 inch.

Because of the flexibility of the reaction frame and other load transferring devices, the lateral displacement actually experienced by the pier was always less than the actuator input displacement, this difference being smaller towards the end of the test when the pier stiffness had attained its lowest values. There was also a slight difference between the maximum loads developed during the push and pull half cycles due to the different types of stress placed on the bolting system and to the different pier stiffnesses associated with non-symmetric crack patterns.

### 3.3 Instrumentation

The total horizontal load applied by the hydraulic actuator, as well as the vertical forces developed by the side columns, were measured using pre-calibrated load cells. Each pier was instrumented as indicated in Fig. 3.3.

DCDT's (direct current differential transformers)  $H_1$ ,  $H_2$  and  $H_3$  were attached to an external reference frame in order to measure the lateral deformation of the pier during each sequence of loading. The difference between  $H_1$  and  $H_3$  was used to indicate the relative lateral deflection of each pier. DCDT's  $D_1$ ,  $D_2$ ,  $D_3$ , and  $D_4$  measured the changes in distance between points along the diagonals of the pier and were used to indicate the shear distortion of the pier as defined in Fig. 3.4. DCDT's  $V_1$  and  $V_2$  were also attached to the external reference frame and measured the rotation at the top steel beam. This provided

a measurement of how well the side columns prevented the rotation of the top section of the pier.

Finally, strain gages were attached by epoxy glue to the vertical reinforcing bars at the bottom section of the pier, in order to measure the steel strain at the sections that were expected to crack first during a test.

### 3.4 Data Acquisition and Data Processing

Two different data acquisition systems were used during the test program. The main one consisted of a high speed scanner able to handle up to 25 channels of information, and the corresponding tape recording system (Fig. 3.5). All the data were acquired and stored on tape after being scanned at a rate of 1 point per second per channel. (No higher rate was necessary because of the low frequency used to run the test). Three computer programs were used to read the original tape data, to input the calibration values and geometrical data of each pier and to reduce the response data to their final presentation in computer plots.

The second data acquisition system was used to monitor the progress of the test and to act as a back-up system in case of any failure in the main system. It consisted of a direct writing oscillograph (visicorder) and was used only to record the most important data; namely, forces at the actuator and side columns, actuator stroke and lateral displacement of the pier. This second data acquisition system proved to be extremely useful in detecting occasional malfunctions of the actuator or the instruments attached to the piers and provided excellent visualization of the behavior of the piers as the test progressed.

TABLE 3.1

LOADING SEQUENCE

STAGE*	INPUT DISPLACEMENT AMPLITUDE (in)	STAGE*	INPUT DISPLACEMENT AMPLITUDE (in)
1	0.005	12	0.22
2	0.010	13	0.26
3	0.015	14	0.30
4	0.020	15	0.35
5	0.03	16	0.40
6	0.04	17	0.45
7	0.06	18	0.50
8	0.08	19	0.55
9	0.10	20	0.60
10	0.14	21	0.70
11	0.18	22	0.80
		23	0.90

\* Each stage consists of three sinusoidal cycles at the amplitude shown

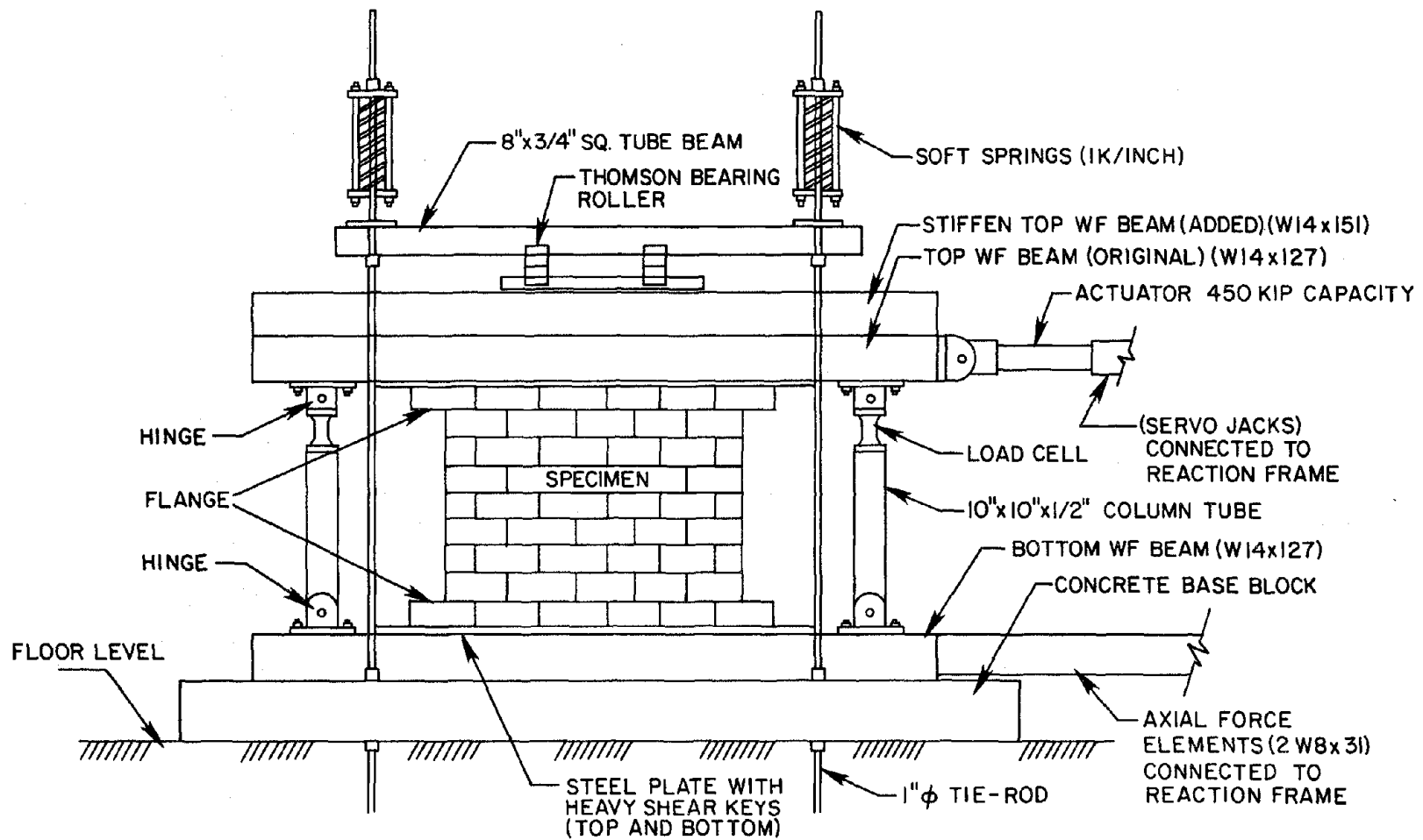


FIG. 3.1 SCHEMATIC ILLUSTRATION OF SINGLE PIER TEST



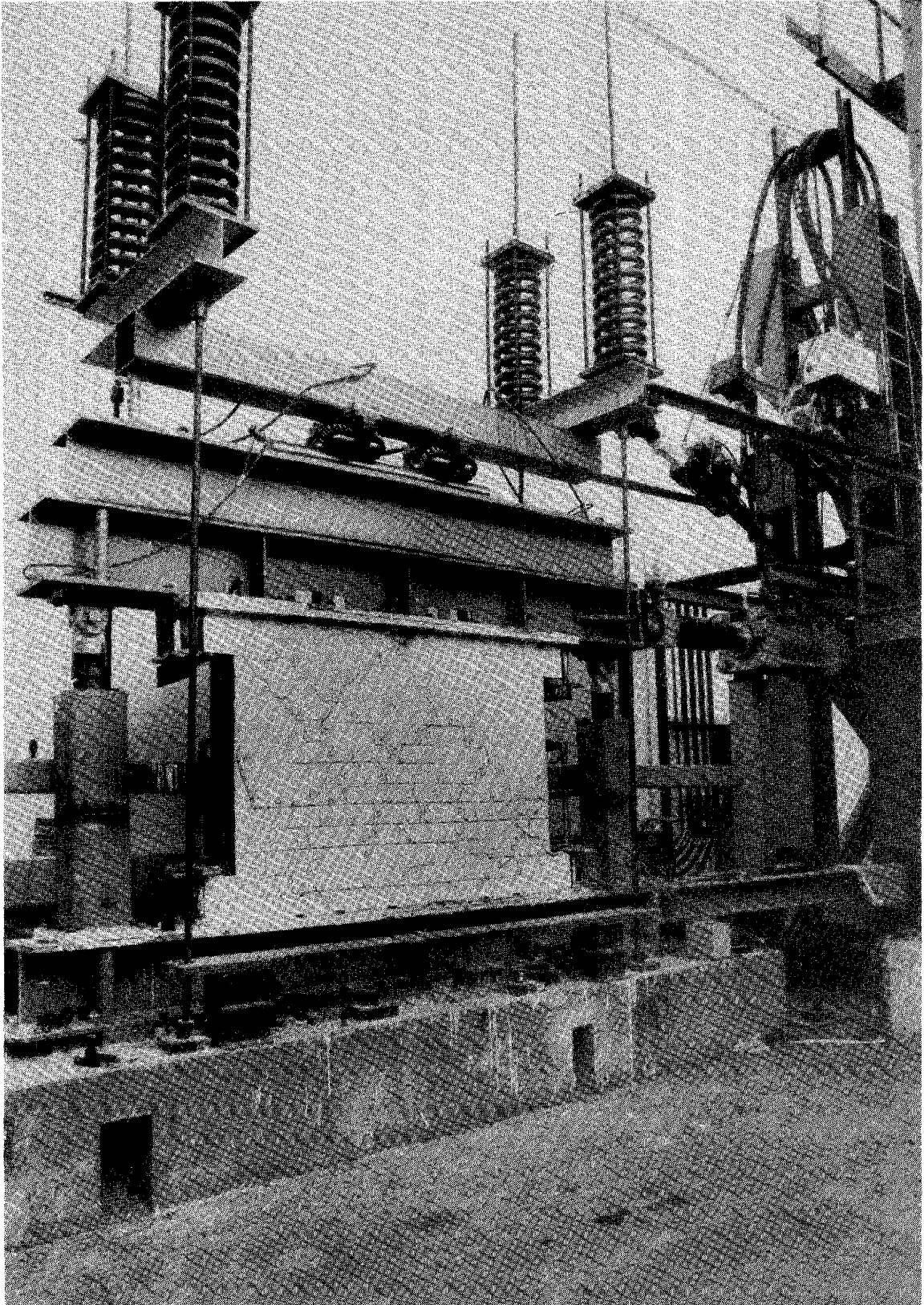


FIG. 3.2 OVERVIEW OF SINGLE PIER TEST

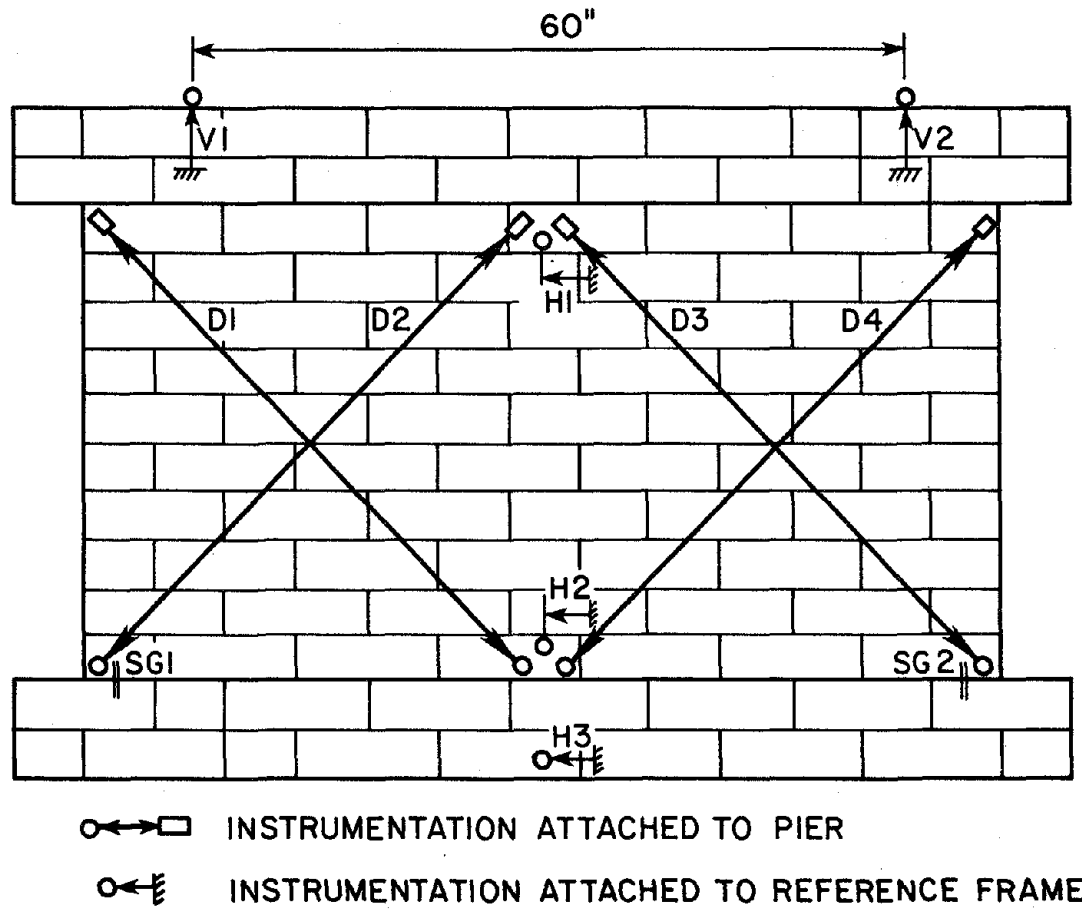
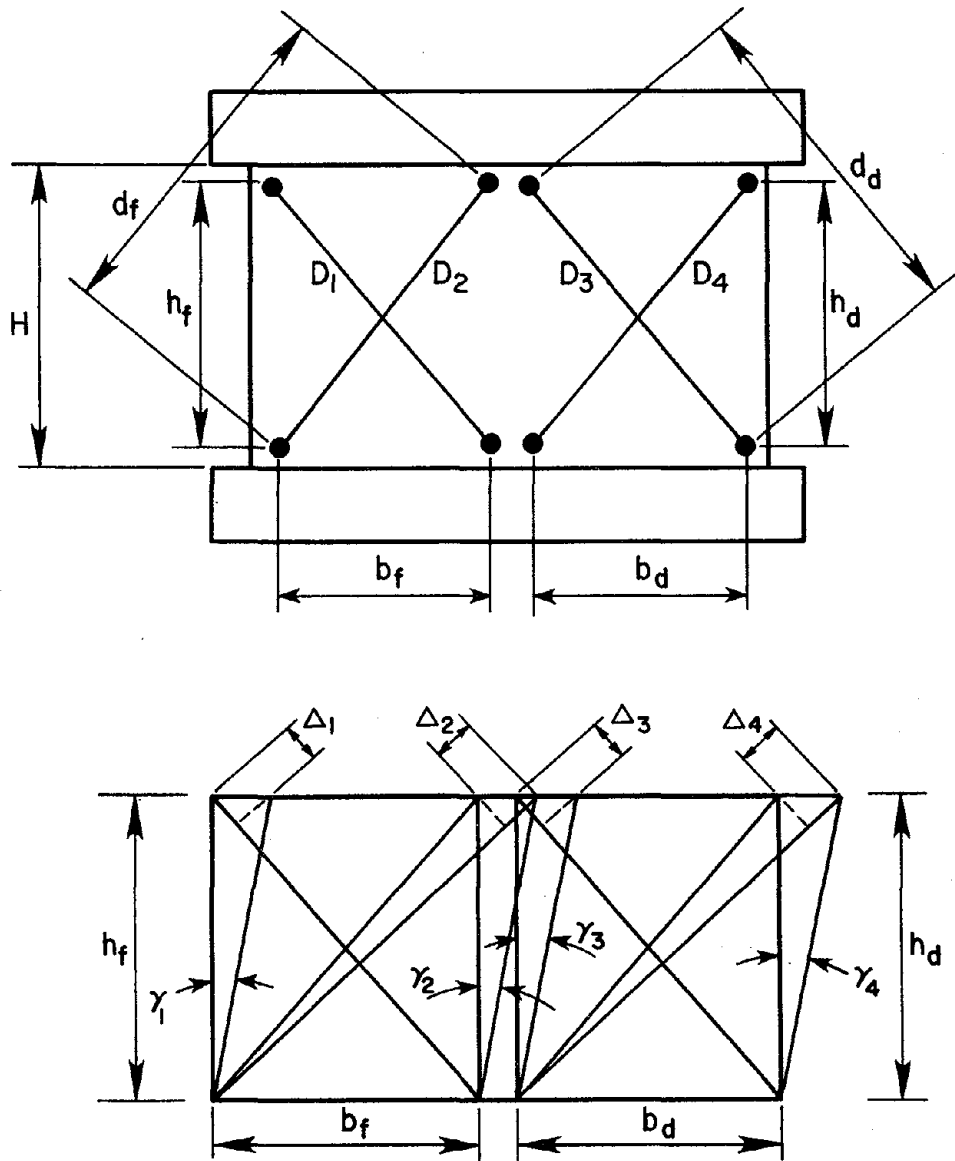


FIG. 3.3 PIER INSTRUMENTATION



$d_f, b_f, h_f$  = AVERAGE DIMENSIONS, FRAME SIDE

$d_d, b_d, h_d$  = AVERAGE DIMENSIONS, DOOR SIDE

$\Delta_i$  = LENGTH CHANGE IN DIAGONAL  $D_i$

$\gamma_i$  = SHEAR ROTATION

$\gamma_{AVG.}$  = AVERAGE SHEAR ROTATION

$\delta_S$  = AVERAGE SHEAR DISTORTION

$$\gamma_i = |\Delta_i| \frac{d_f}{b_f \cdot h_f} \quad i = 1, 2$$

$$\gamma_j = |\Delta_j| \frac{d_d}{b_d \cdot h_d} \quad j = 3, 4$$

$$\gamma_{AVG.} = \frac{1}{4} \sum_{i=1}^4 \gamma_i$$

$$\delta_S = \gamma_{AVG.} \cdot H$$

FIG. 3.4 MEASUREMENT OF AVERAGE SHEAR DISTORTION

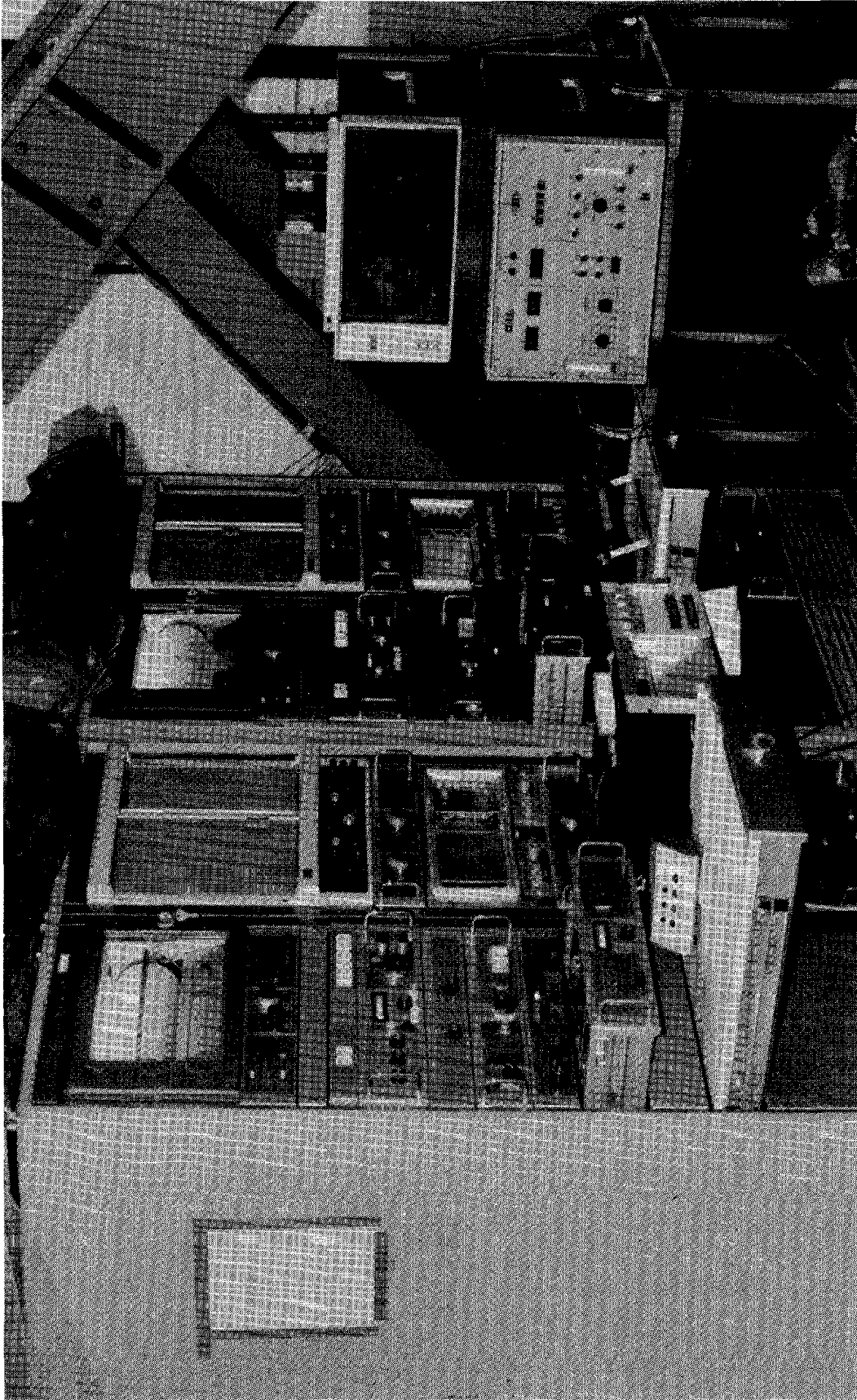


FIG. 3.5 TEST CONTROL CONSOLES AND DATA ACQUISITION SYSTEM

## 4. TEST RESULTS

### 4.1 Introduction

The experimental results for the eighteen piers having a height to width ratio of 0.5 are presented in the form of hysteresis loops, hysteresis envelopes, stiffness degradation properties, energy dissipation characteristics, and relative shear distortion. In addition, a sequence of photographs of the successive crack patterns is given for each test. An explanation of how each of the graphs was obtained and the meaning of the terms used above is included in Section 4.3. The complete presentation of the figures and photographs has been arranged by test numbers and is included in Appendix A. In order to show the relation between the sequence of the crack pattern photographs and the diagrams showing the results, a black dot has been drawn on each of the graphs and next to the corresponding picture of the crack pattern. The loading stage individualized by the black dot generally corresponds to the stage at which the first major diagonal crack occurred.

In addition, data on the ultimate strength and hysteresis indicators for each test are listed in Table 4.1. A discussion of the modes of failure observed follows in Section 4.2 and a discussion of the test results is presented in Chapter 5.

### 4.2 Modes of Failure

Three modes of failure were observed during this series of tests: a shear mode, a combined shear and sliding mode and a combined flexural and sliding mode. The initial crack pattern was similar in all cases: early horizontal (flexural) cracks at the bottom corners of the piers, which later either became diagonal (inclined) cracks (HCBL piers) or

continued horizontally through the bottom courses of the specimens (HCBR and CBRC piers).

The shear mode of failure occurred in tests HCBL-12-1, 2, 3, 6, HCBR-12-2, 3, 4 and CBRC-12-6, and is illustrated in Fig. 4.1(a). This mode of failure is characterized by major diagonal cracks in both directions that finally destroy the moment resisting mechanism when the diagonal crack runs through the compression toe of the pier. Strength degradation occurs because of crushing and grinding along the diagonal cracks.

The characteristics of the shear mode of failure depended on the type of masonry construction. In the case of the HCBL piers, a family of diagonal cracks developed in both directions with none of these cracks involving by itself the whole width of the pier. After the first major diagonal crack occurred, the lateral load strength of the pier appeared to be carried by diagonal compression struts bounded by the diagonal cracks. This shear resistance mechanism has also been observed by Park and Paulay<sup>[12]</sup> in squat reinforced concrete shear walls. In all the HCBL piers that had a shear mode of failure, the final failure was prompted by the buckling of these compression struts which led to the formation of a diagonal crack that completely separated the top from the bottom of the pier (Fig. 4.1a). In some of the cases (particularly HCBL-12-6) this final failure was quite explosive and accompanied by a sharp strength degradation. In the case of the HCBR piers, the diagonal cracks usually involved the whole width of the pier with a horizontal segment above the midheight section (Fig. 4.1a). The final failure occurred when the diagonal crack destroyed the compression toe of the pier. In some cases (HCBR-12-3, Fig. 4.1a) the failure crack included both diagonal and horizontal branches.

The combined shear and sliding mode of failure occurred in tests HCBL-12-4, 5 and CBRC-12-1, 2 and 3 and is illustrated in Fig. 4.1(b). In this case the diagonal cracks developed in a pattern similar to that of the shear mode of failure specimens. However, the final failure mechanism was due to sliding through a bell-shaped path determined by two side, diagonal cracks and an horizontal branch at the top course of the pier. Even though there was crushing and grinding along the side diagonal cracks, typical of a shear failure, the sliding of the top of the pier relative to the bottom part was the feature that controlled the response of these piers, as will be explained in Chapter 5.

The combined flexural and sliding mode of failure was present in tests HCBR-12-1, 5, 6 and CBRC-12-4 and 5. In all of these cases the horizontal (flexural) cracks run continuously through the bottom courses of the piers and the final failure occurred when a sliding motion developed along this horizontal crack. A number of diagonal cracks (HCBR-12-6) or none of them (HCBR-12-1) had developed by the time the sliding failure occurred (Fig. 4.1c). In particular, the failure observed in specimen HCBR-12-6 was of the same type as that reported by Priestley<sup>[14]</sup> in his cantilever tests. It must be noted that none of the HCBL piers developed a continuous horizontal crack through the bottom course and therefore, none of them presented this type of failure.

From a comparative point of view, the crack patterns presented in Appendix A show that the HCBL piers were the specimens that developed the largest number of diagonal cracks and the least number of horizontal cracks; the HCBR piers developed peculiar corner diagonal cracks that did not participate in the final failure mechanism; and the CBRC piers developed the largest number of horizontal cracks,

sometimes through the bottom three or four courses of the piers, and exhibited the least number of major diagonal cracks.

The readings obtained from the strain gages placed at the bottom sections of the side vertical reinforcing bars (Fig. 3.3) indicate that the tensile stresses in these bars increased with increasing values of the lateral load, until the ultimate load was attained. In some of the tests these side vertical bars showed yielding strains for a few stages around the ultimate load, while in the rest of the tests they never attained the yield condition. This strain pattern appears to be independent of the mode of failure exhibited by the pier.

Knowing the tensile strain, and therefore the tensile stress, in the extreme vertical bar, it is possible to find the stress in the central reinforcing bar, by considering the distribution of forces indicated in Fig. 4.2. If a section along the bottom cross section of the pier is considered, the moment equation about  $O$  will permit finding the stress at the bottom section of the central bar, provided both the moment of the resultant of the compressive forces in the masonry and the moment of the force at the reinforcing bar closest to  $O$  are neglected. The results of such analyses show that the central vertical bar at the bottom section of the pier was usually in compression, sometimes with values very close to the yield stress. Even though the assumptions used to compute this stress lead to overestimation of the compressive stresses in the central bar, it is clear that the design objective of avoiding tension yielding in the central reinforcing bar was accomplished.

The same moment equation about the corner point  $O$  may be used to locate the actual position of the resultant of the compressive forces



along the bottom section of the pier, when the forces developed by both the central bar and the side bar closest to 0 are included in that resultant. This analysis indicates that the resultant was usually located in the pier, within 5 to 10 inches from point 0, at the time the maximum lateral load developed. This result verifies the observation that the compressive toe in squat piers is wide enough to carry a significant shear, thus requiring an ultimate lateral load larger than that required to produce the first major diagonal crack.

#### 4.3 Load-Displacement Characteristics

As mentioned above, Table 4.1 summarizes the strength and hysteresis characteristics of the piers and Appendix A presents the test results for each of the specimens. The correspondence between the photographs of the crack patterns and the diagrams showing the results is indicated by a black dot drawn on each of the graphs and next to the corresponding crack pattern.

The details of the derivation of each of the figures in Appendix A are discussed in the following sections.

##### a) Hysteresis Loops. (Shear Stress vs. Lateral Deflection Diagram).

This graph was obtained by plotting the gross shear stress against the relative lateral displacement of the pier for the duration of the test. (Since all the piers were fully grouted, the gross shear stress is equal to the net shear stress.) The gross shear stress was computed by dividing the measured horizontal force by the gross (or net) cross section area of the pier (the thickness multiplied by the width), as indicated in Table 2.1 ( $610 \text{ in}^2$  for the HCBL piers,  $575 \text{ in}^2$  for the HCBR piers and  $780 \text{ in}^2$  for the CBRC piers). The relative lateral displacement was computed from the

difference between the lateral deflections at the top and bottom of the pier ( $H_1 - H_3$  as defined in Fig. 3.3). In the case of the piers that failed in a combined shear and sliding mode of failure, (HCBL-12-4, 5 and CBRC-12-1, 2 and 3), the DCDT at the top of the pier ( $H_1$ ) did not record the horizontal motion of the top portion because the bell-shape crack always developed above  $H_1$ . Therefore, this reading was obtained from the actuator stroke by using the following procedure. First, the flexibility of the reaction frame at the level where the actuator reacts was measured; a linear, elastic behavior of the reaction frame was detected at all load levels, with a flexibility coefficient of 0.0003007 in/kip. Then, the horizontal displacement at the top of the pier was obtained by subtracting the lateral displacement of the reaction frame from the actuator stroke, and the resulting value was plotted against  $H_1$  for the seven tests which did not exhibit combined shear and sliding failure. This plot showed a consistent linear relation between the measured  $H_1$  and the value calculated from the actuator stroke, the actuator load and the reaction frame flexibility, for the whole range of displacements, (the measured  $H_1$  was always 95% of the calculated value). The plot was then used to obtain  $H_1$  for the failure stages of the tests that displayed a combined shear and sliding failure, using the readings from the actuator stroke and the actuator load.

b) Hysteresis Envelopes

The hysteresis envelope was determined from the hysteresis loops by averaging the absolute values of the three extreme positive and the three extreme negative forces (or gross shear stresses) and the

corresponding absolute values of the relative lateral displacement, for each stage of the test at a given input displacement. One point on the hysteresis envelope was obtained for each stage of 3 cycles of loading. The average lateral displacement given by the hysteresis envelope is always less than the input displacement, as explained in Section 3.2.

The black dot indicated on this graph generally corresponds to the stage at which the first major diagonal crack occurred, as observed in the corresponding photographs. This shear crack usually developed during the first of the three cycles that comprised each stage of loading.

The maximum strength obtained from the hysteresis envelope is indicated in Table 4.1 under "average ultimate shear force or stress". The "peak ultimate shear force or stress" values that appear in Table 4.1 were obtained from the average maximum force (stress) developed in any one cycle of loading. The average value is always less than the peak value, varying from 94% to 97% of the peak value. The compressive load at ultimate indicated in Table 4.1 corresponds to the maximum axial compressive load developed during each of the tests. This maximum value always occurred at the same time as the peak ultimate shear force, and was computed from the readings of the load cells located in the vertical columns plus the bearing load applied prior to each test (Table 2.1).

The last two columns of Table 4.1 correspond to hysteresis indicators obtained from the hysteresis envelopes and defined in Fig. 4.3. The level of  $0.70 P_u$  used to define these indicators, where  $P_u$  is the maximum strength indicated by the hysteresis envelope, was arbitrarily chosen. Indicator  $h_1$  tells how much the

pier deviated from its initial, theoretical stiffness, and indicator  $d_2$  gives an indication of the deformation capability of the pier. The initial theoretical stiffness of the pier was computed assuming that the piers were fixed against rotation at both the top and bottom. The moment of inertia was calculated using the gross, uncracked section, neglecting the effect of steel reinforcement; the modulus of elasticity was taken from the measured values (Fig. 2.6, reference [2], for the HCBL piers, Tables 2.3a and 2.3b, reference [4], for the HCBR and CBRC piers, respectively), and Poisson's ratio was assumed to be 0.15. Further discussion on the correlation of the theoretical stiffness and the measured stiffness is presented in Chapter 5.

c) Stiffness Degradation

A cyclic definition of the stiffness, as indicated in Fig. 4.4, was used to measure the stiffness of the piers throughout each test. The three cyclic stiffness values obtained from each stage of loading were averaged and plotted against the average gross shear stress and the relative lateral displacement, as defined for the hysteresis envelope plot.

d) Energy Dissipation

The energy dissipated per cycle of loading was expressed in terms of a dimensionless ratio EDT. EDT is defined as the ratio of the energy dissipated to the total stored strain energy per cycle and is diagrammatically shown in Fig. 4.4. The three EDT values obtained for each stage of loading were averaged and plotted against the average relative lateral displacement.

## e) Shear Distortion

The values of the shear distortion  $\delta_s$  were calculated as indicated in Fig. 3.4. The absolute values of  $\delta_s$  corresponding to the three extreme positive and three extreme negative forces were averaged for each stage of the test, and plotted against the respective average relative lateral displacements (total deformation of the pier), as given by  $H_1 - H_2$  (Fig. 3.3). The plot shows how much of the total deformation of the pier is due to shear distortion as defined in Fig. 3.4. Since the instruments used to measure the diagonal deformations were usually removed two or three stages before the end of the test, the number of stages used to plot this graph is usually smaller than the number used for the previous graphs.

TABLE 4.1

## PIER CHARACTERISTICS AND TEST RESULTS

(Gross cross section of wall: HCBL piers = 610 in<sup>2</sup>; HCBR piers = 575 in<sup>2</sup>; CBRC piers = 780 in<sup>2</sup>)

Specimen	Test Frequency (cps)	Grouting Full (F) Partial (P) Solid (S)	Vert. reinf. steel		Horizontal reinforcing steel				Ratio of Total Area of Steel to Gross Area of Wall $P_v + P_h$	Average Ultimate Shear Force (kip)	Average Ultimate Shear Stress (psi)	Peak Ultimate Shear Force <sup>(1)</sup> (kip)	Peak Ultimate Shear Stress (psi)	Compressive Load at Ultimate <sup>(2)</sup> (kip)	Bearing Stress at Ultimate (psi)	Hysteresis Indicators	
			No. of Bars	$P_v = \frac{A_{vs}}{A_g}$	No. of Bars	Yield Strength (ksi)	$P_h = \frac{A_{hs}}{A_g}$	$A_{hs} f_{hy}$ (kip)								$h_1$	$d_2$ (in)
HCBL-12-1	0.02	F	3#7	0.0030	No	—	—	—	0.0030	189.1	310	200.3	328	118.5	194	4.0	0.35
-2	0.02	F	3#7	0.0030	1#5	69.6	0.0005	21.6	0.0035	201.5	330	211.7	347	122.0	200	2.9	0.35
-3	0.02	F	3#7	0.0030	2#5	69.6	0.0010	43.2	0.0040	242.5	398	251.4	412	148.5	243	3.8	0.39
-4	0.02	F	3#7	0.0030	3#5	69.6	0.0015	64.7	0.0045	209.9	344	218.6	358	129.4	212	2.6	0.45
-5	0.02	F	3#7	0.0030	4#5	69.6	0.0020	86.3	0.0050	220.2	361	228.0	374	130.9	215	3.6	0.49
-6	0.02	F	3#7	0.0030	4#6	67.3	0.0029	118.4	0.0058	252.0	413	261.7	429	143.0	234	3.5	0.37
HCBR-12-1	0.02	F	3#7	0.0031	No	—	—	—	0.0031	208.7	363	220.8	384	101.2	176	4.6	0.46
-2	0.02	F	3#7	0.0031	1#6	67.3	0.0008	29.6	0.0039	182.7	318	191.0	332	96.0	149	4.0	0.34
-3	0.02	F	3#7	0.0031	2#6	67.3	0.0015	59.2	0.0047	211.8	368	220.8	384	114.1	198	4.5	0.45
-4	0.02	F	3#7	0.0031	3#6	67.3	0.0023	88.8	0.0054	245.8	427	255.3	444	142.4	248	4.5	0.42
-5	0.02	F	3#7	0.0031	4#6	67.3	0.0031	118.4	0.0062	223.8	389	232.7	404	100.7	175	3.5	0.30
-6	0.02	F	3#7	0.0031	5#7	80.3	0.0052	240.9	0.0083	251.4	437	259.0	450	128.0	223	5.3	0.47
CBRC-12-1	0.02	S	3#7	0.0023	No	—	—	—	0.0023	190.4	244	197.2	253	83.9	108	3.6	0.30
-2	0.02	S	3#7	0.0023	1#6	67.3	0.0006	29.6	0.0029	186.3	239	194.8	250	98.9	127	3.6	0.38
-3	0.02	S	3#7	0.0023	2#6	67.3	0.0011	59.2	0.0034	207.9	267	217.3	279	117.1	150	3.5	0.48
-4	0.02	S	3#7	0.0023	3#6	67.3	0.0017	88.8	0.0040	227.1	291	235.0	301	96.1	123	3.3	0.24
-5	0.02	S	3#7	0.0023	4#6	67.3	0.0023	118.4	0.0046	183.0	235	192.3	247	109.8	141	4.5	0.31
-6	0.02	S	3#7	0.0023	5#7	80.3	0.0038	240.9	0.0062	207.3	266	216.1	277	110.7	142	3.6	0.28

(1) Peak value among averages of extreme values developed during any one cycle

(2) Average of values at extreme shear forces that produced the peak ultimate shear force

(3) Piers with a combined shear and sliding failure

(4) Piers with a combined flexural and sliding failure

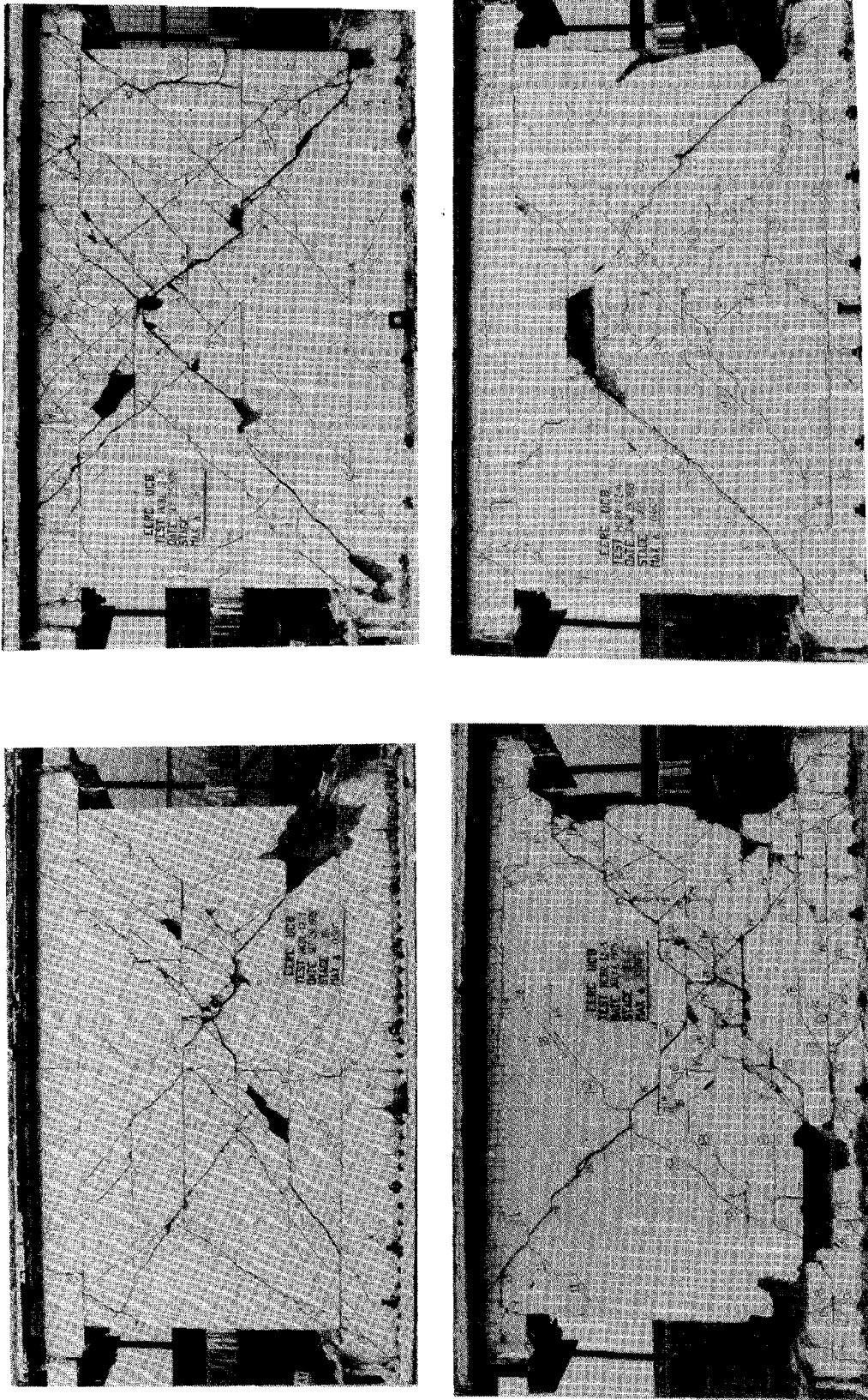


FIG. 4.1(a) SHEAR MODE OF FAILURE

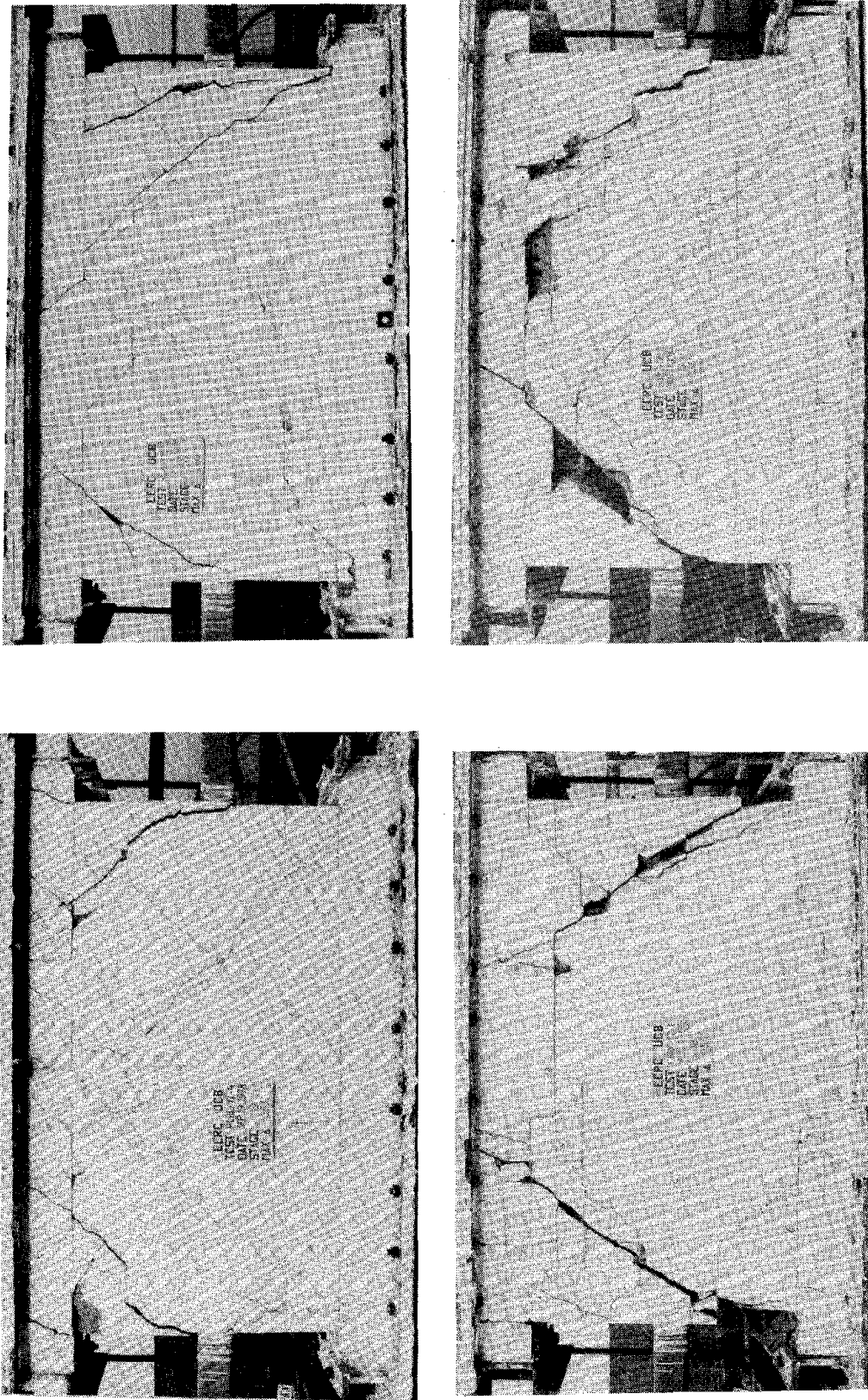


FIG. 4.1(b) COMBINED SHEAR AND SLIDING MODE OF FAILURE



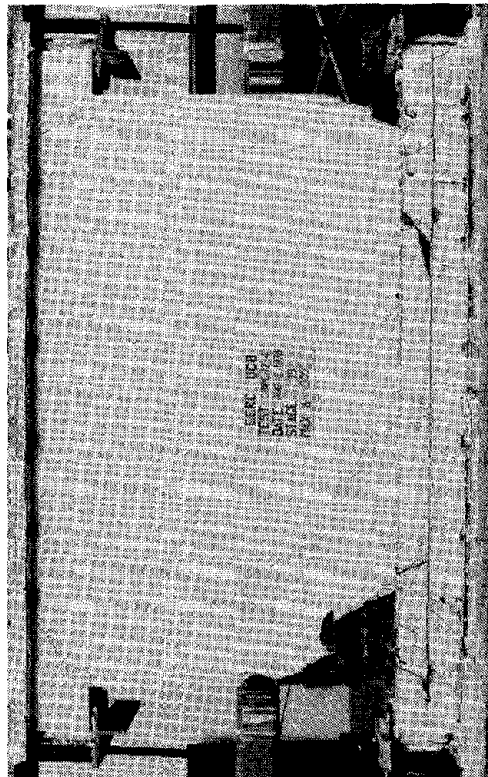
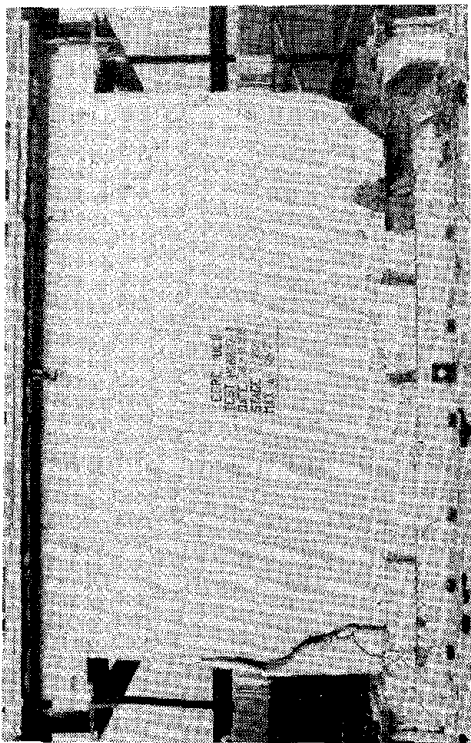
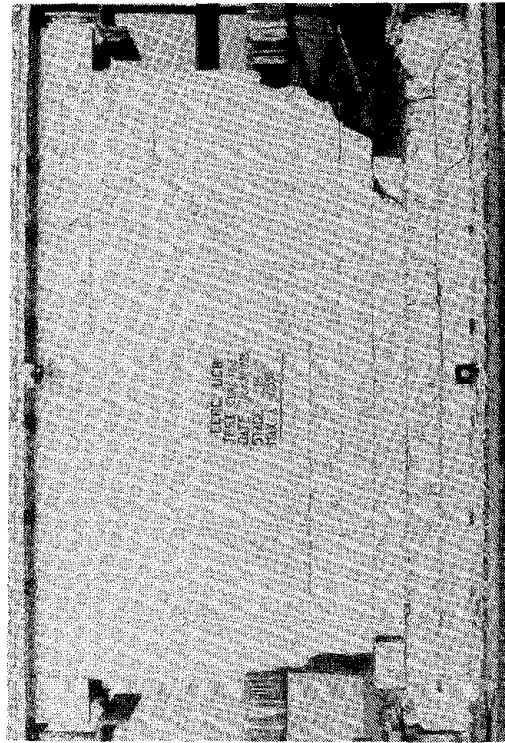
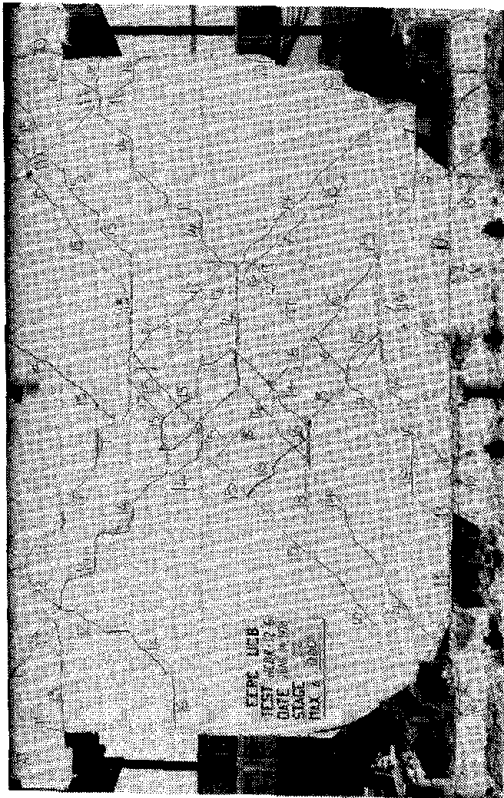


FIG. 4.1(c) COMBINED FLEXURAL AND SLIDING MODE OF FAILURE

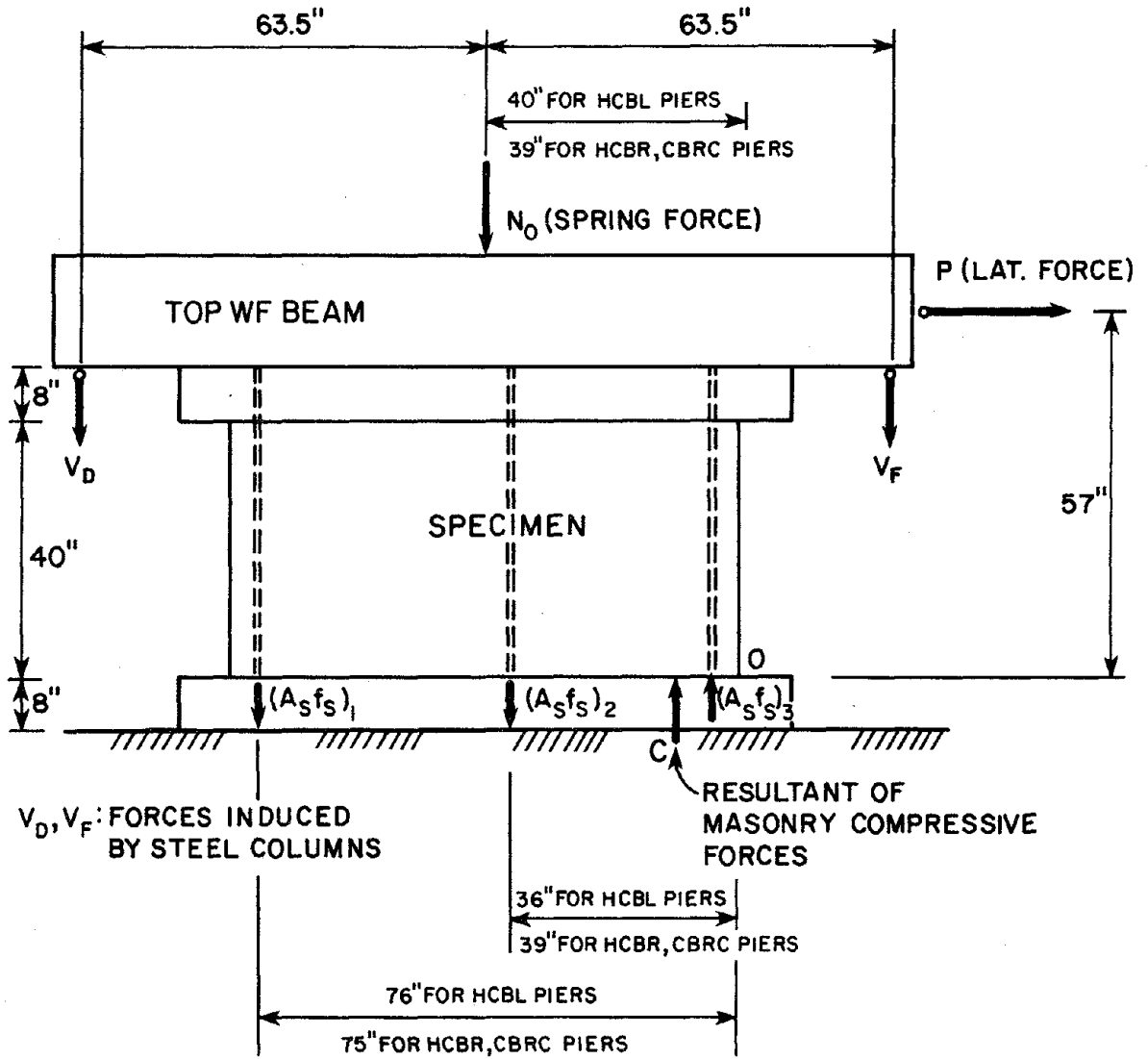
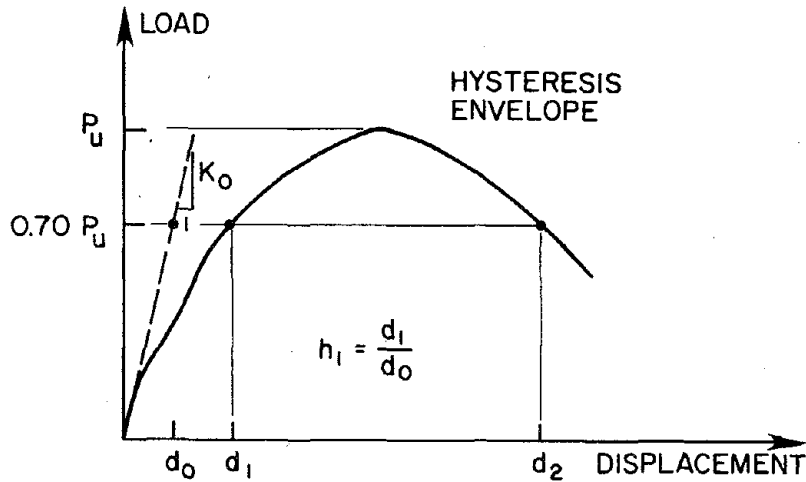


FIG. 4.2 FORCES ACTING ON THE PIER



DEFINITION OF HYSTERESIS INDICATORS  $h_1$  AND  $d_2$

COMPUTATION OF INITIAL STIFFNESS  $K_0$

$$K_0^{-1} = \frac{L^3}{12EI} + 1.2 \frac{L}{AG}$$

$L$  = height of pier

$E$  = modulus of elasticity

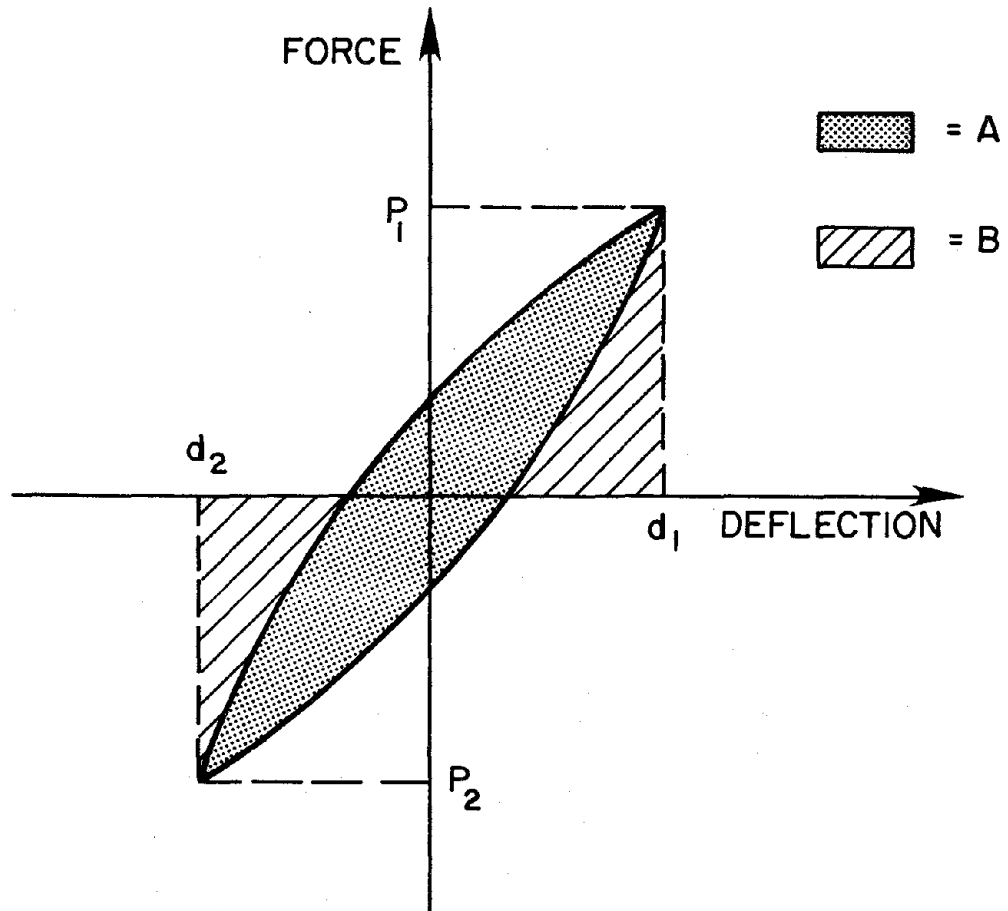
$G = \frac{E}{2(1+\nu)}$  shear modulus

$D$  = width of pier

$t$  = thickness of pier

SPECIMEN	L (in)	D (in)	t (in)	I (in <sup>4</sup> )	A (in <sup>2</sup> )	E (ksi)	$\nu$	$K_0$ (kip/in)
HCBL-12	40	80	7.625	325,333	610.00	1140	0.15	5,776
HCBR-12	40	78	7.375	291,652	575.25	2450	0.15	11,655
CBRC-12	40	78	10.00	395,460	780.00	1720	0.15	11,095

FIG. 4.3 DEFINITION OF HYSTERESIS INDICATORS AND COMPUTATION OF INITIAL STIFFNESS



ENERGY DISSIPATION RATIO:

$$EDT = \frac{\text{DISSIPATED ENERGY}}{\text{TOTAL STORED ENERGY}} = \frac{A}{A+B}$$

PIER STIFFNESS :

$$K = \frac{|P_1 - P_2|}{|d_1 - d_2|} \quad P_1, P_2, d_1, d_2 \text{ MUST BE TAKEN WITH THEIR OWN SIGN}$$

FIG. 4.4 DEFINITIONS OF ENERGY DISSIPATION RATIO AND PIER STIFFNESS

## 5. DISCUSSION OF TEST RESULTS

### 5.1 Introduction

The test results presented in Appendix A and Table 4.1 are discussed in this chapter with reference to the two parameters that were varied during these eighteen tests, namely, the type of masonry construction and the amount of horizontal reinforcement. Other parameters, such as the initial bearing stress, the cyclic frequency, the amount of vertical reinforcement and the type of grouting, which were varied in previous pier tests<sup>[8,4,2]</sup>, were held constant during these eighteen tests. Nevertheless, it must be noted that the test results were also influenced by the modes of failure exhibited by the piers, as described in Section 4.2, and this variable will be considered in the following discussion.

It is also important to note that the results presented herein were obtained from a particular loading sequence. The choice of this loading sequence has been discussed previously<sup>[8]</sup>. Other types of load sequences will be used in some of the additional seventeen tests that complete the single pier test program.

When considering the results of these eighteen tests on 1:2 piers it is important to realize that conclusions which appear valid for these tests may not hold for tests on piers with other height to width ratios. The complexity of the problem requires the completion of the test program (eighty tests) before valid conclusions concerning an adequate design of masonry structural elements can be made.

### 5.2 Modes of Failure

Once the modes of failure exhibited by these squat piers have been identified, it is interesting to speculate about the reasons behind

the development of each of these modes. One conclusion that is apparent from the test results is that the amount of horizontal reinforcement does not appear to have any influence on the mode of failure obtained. The type of masonry construction and the cracking pattern developed prior to failure are the variables that seem to dictate the final type of failure.

None of the HCBL piers developed a continuous horizontal crack along the bottom courses of the pier and consequently, none of them displayed a combined flexural and sliding mode of failure. There is no apparent reason to explain why two of the piers displayed combined shear and sliding failures and four of them showed typical shear failures. The crack patterns were similar in all of the cases and the mode of failure became evident only at the end of the test.

The HCBR piers did not develop any side, diagonal cracks; therefore, they did not exhibit the combined shear and sliding type of failure obtained with the HCBL and CBRC piers. Three of the HCBR piers had a shear failure and the other three a combined flexural and sliding; these last three piers developed different numbers of shear cracks before the sliding failure through the bottom course of the pier: HCBR-12-6 had many diagonal cracks, HCBR-12-5 had only one and HCBR-12-1 had no diagonal cracks at the time of the sliding failure.

The CBRC piers developed the three modes of failure; however, unlike the other piers, they clearly showed a preference for horizontal cracks. Also fewer diagonal cracks developed during the tests.

The ultimate strength associated with the three modes of failure proved to be about the same; this can be seen from the hysteresis envelopes shown in Figs. 5.1, 5.2 and 5.3. Since the mode of failure

with the lowest ultimate strength will determine the mode of failure of a particular pier, there is no clear way to predict the modes of failure experienced by these squat piers. Section 5.3 presents the experimental sliding failure strengths of the piers as a function of the axial compressive stress. The ultimate shear strength is also discussed in Section 5.3.

Finally, it is important to recognize that in the case of an actual multistory masonry structure, for a pier to have a sliding type of failure it is necessary that all the lateral load resisting elements located at the same story level experience a sliding failure. From the results obtained for these squat piers it is clear that this type of failure is less likely to occur than a shear type of failure.

### 5.3 Lateral Load Strength

The lateral load strength of the piers is discussed at two levels: the strength developed by the piers at the time the first major diagonal crack occurs, which has been labeled shear crack strength; and the maximum or ultimate strength developed by the piers during the tests. It must be noted that in the tests of the piers with height to width ratios of 1 or 2 the shear crack strength always coincided with the ultimate strength and no reserve of strength was available after the first major diagonal crack occurred.

#### 5.3.1 Shear Crack Strength

The shear crack strength of the piers (average of extreme values for the cycle where the first major diagonal crack occurred), is indicated in Table 5.1 and has been identified by a black dot on the hysteresis envelopes shown in Figs. 5.1, 5.2 and 5.3. The average shear crack strength is 225 psi for the HCBL piers, 362 psi for the HCBR piers

and 250 psi for the CBRC piers. When comparing the previous values for the different types of masonry, it should be noted that the average compressive stresses at the shear crack strength values were 98 psi, 145 psi and 117 psi for the HCBL, HCBR and CBRC piers, respectively.

The results of Table 5.1 also show an increase in shear crack strength with increasing amounts of horizontal reinforcement in the case of the HCBL and HCBR piers, although there exists some inconsistency (HCBL-12-4) in the trend observed. In the case of the double wythe grouted core clay brick piers (CBRC) there is no relation between the shear crack strength and the amount of horizontal reinforcement.

### 5.3.2 Ultimate Strength

The average ultimate strength is shown in Table 4.1 and Figs. 5.1, 5.2 and 5.3. The average values for each series of six piers are 359 psi, 384 psi and 257 psi for the HCBL, HCBR and CBRC piers, respectively. Consideration of the bearing stresses concurrent with these values is required if a meaningful comparison is desired. As in the case of the shear crack strength, there seems to be a positive correlation between amount of horizontal reinforcement and ultimate strength. However, this correlation is less consistent than before because of the different modes of failure experienced by the piers.

Table 5.1 presents a comparison between the peak ultimate shear strength (peak value among averages of extreme values developed during any one cycle) and the shear crack strength. The last column of Table 5.1 indicates that the reserve strength that was available after the first major diagonal crack occurred was larger for the HCBL than for the HCBR piers, (this reserve was almost negligible for the CBRC piers).



However, it must be noted that the shear crack strength of the HCBL piers was significantly less than that of the HCBR piers.

The experimental sliding strength of the piers, as a function of the axial compressive stress, is presented in Figs 5.4 and 5.5, which show the complete cyclic response of the piers after the sliding failure began to develop. If the loading portions of the curves are considered, the average ratios between the shear stress and the bearing stress are 2.64 (HCBR) and 2.54 (CBRC) for the piers exhibiting a combined flexural and sliding failure (Fig. 5.4). The same ratios for the piers that had a combined shear and sliding failure (Fig. 5.5) are 1.85 (HCBL piers) and 2.29 (CBRC piers).

#### 5.4 Inelastic Behavior

The hysteresis envelopes (average maximum force-deflection curves) are used as a frame of reference to discuss the inelastic behavior of the piers. The question of what can be considered a desirable hysteresis envelope has been discussed in reference [8], pp. 68-70, in qualitative terms. It is appropriate to recall that the usefulness of the hysteresis envelopes is that they provide visual comparisons of ductility and ultimate strength; however, they give no indication of the energy dissipated per cycle, and consideration of this parameter in conjunction with the ultimate strength, the deformation capacity and a comparison of crack patterns at equal displacements is necessary to evaluate completely the inelastic characteristics of the pier behavior.

Figures 5.1, 5.2 and 5.3 show the changes in the hysteresis envelopes as the amount of horizontal reinforcement varies. At the same time, these figures also show how the inelastic behavior is affected by the mode of failure experienced by the piers.

As the amount of horizontal reinforcement increases, the piers show a trend towards a higher ultimate strength and higher ultimate deformation capacity, although this trend is nonuniform since some piers show less desirable inelastic behavior than a similar pier with significantly less horizontal reinforcement. The influence of the horizontal reinforcement is also obscured by the mode of failure of the piers. Piers displaying a combined shear and sliding mode of failure show a larger deformation capacity than piers exhibiting a shear or a combined flexural and sliding type of failure, since the bell-shaped sliding crack, typical of the combined shear and sliding failure, leaves a healthy bottom portion of the pier, which continues to take load when the sliding crack closes. This effect leads to an elasto-plastic hysteresis envelope before the load finally drops off because of a transfer of load deterioration along the bell-shaped crack.

The discussion presented above is quantitatively expressed by the hysteresis indicators shown in the last two columns of Table 4.1. While hysteresis indicator  $h_1$  shows a nearly constant value (3.4 for the HCBL piers, 4.4 for the HCBR piers and 3.7 for the CBRC piers), hysteresis indicator  $d_2$  shows significant increases for the piers that exhibited a combined shear and sliding mode of failure (from 0.37 inch to 0.47 inch in the case of the HCBL piers, and from 0.28 inch to 0.39 inch in the case of the CBRC piers). It is also interesting to note that the hysteresis indicator  $d_2$  shows a general increase from the CBRC type piers, to the HCBL, and finally to the HCBR type piers.

### 5.5 Stiffness Degradation

All the piers suffered substantial stiffness degradation when subjected to gradually increasing lateral displacements. Table 5.2

summarizes this effect and shows two types of results. The first is a comparison between the theoretical initial stiffness and the maximum stiffness measured during the early stages of the test. The theoretical initial stiffness has been computed in Fig. 4.3 and the assumptions used are indicated in Section 4.3(b). The measured value is always smaller than the theoretical value and it ranges from 75% to 99% of the theoretical value for the HCBL piers, from 49% to 94% for the HCBR piers and from 55% to 94% for the CBRC piers. The differences between theoretical and measured initial values are significantly smaller than those obtained for single piers with height to width ratios of 1 and 2 [2,4]. This fact supports the hypothesis that these differences are due to the flexibility of the boundary conditions at small lateral displacements, because the width of these piers is almost twice that of the previous single piers, and the same horizontal displacement at the top is related to a smaller rotation and a better reproduction of a fixed rotation condition at the top of the pier. An excessively large value was measured for the initial stiffness of specimen HCBL-12-6, probably caused by an improper measurement of the lateral displacement of this particular pier, and it has not been included in Table 5.2. Even though the hysteresis envelope of HCBL-12-6 looks normal (Fig. 5.1), any small error in the measurement of very small displacements may have led to an erroneous stiffness measurement.

The second set of results presented in Table 5.2 is a comparison of the measured stiffnesses of all piers at applied shear stresses of 50 psi, 75 psi, 100 psi, and 150 psi, and the percentage decreases in stiffness at these stress levels with respect to the maximum initial measured value. If specimen HCBL-12-6 is not considered, the average percentage decreases at 50 psi were 5%, 11% and 22% for the HCBL, HCBR

and CBRC piers, respectively. The same average percentage decreases at 100 psi were 20%, 28% and 39%. It should be noted that the first visible cracks appeared at stress levels of 66 psi, 100 psi and 62 psi for the HCBL, HCBR and CBRC piers, respectively, (average values for each series of six specimens).

All the stiffness degradation results were obtained with displacement increments that gradually increase. Later tests will determine if the type of degradation observed is similar under a more random type of loading sequence.

Figures 5.6, 5.7 and 5.8 present the stiffness degradation curves for different amounts of horizontal reinforcement for HCBL, HCBR and CBRC piers, respectively. It is difficult to visualize any relation between the amount of vertical or horizontal reinforcement and the rate at which the stiffness degrades.

#### 5.6 Energy Dissipation

The effects of horizontal reinforcement and mode of failure on the EDT ratio are shown in Fig. 5.9 for the HCBL piers, Fig. 5.10 for the HCBR piers and Fig. 5.11 for the CBRC piers. It can be concluded from these graphs that the energy dissipation capacity of the piers seems to be independent of the amount of horizontal reinforcement. Likewise, the mode of failure of the piers seems to have little influence on the EDT ratio, except that the piers exhibiting the combined shear and sliding mode of failure show a smaller energy dissipation rate when compared with piers that had the shear or the combined flexural and sliding mode of failure.

As with stiffness degradation, investigation of the energy dissipation characteristics of the piers under a more random load sequence is important before analytical models based on the results are formulated.

### 5.7 Effect of Compressive Load on Inelastic Behavior

The additional compressive load imposed by the side columns during the tests has been mentioned in Section 3.1 and has been discussed and analyzed in detail in Section 5.6 of reference [4].

There is no indication at all, that the present piers changed their mode of failure because of the additional compressive load developed by the test setup, although this was the case for some of the piers with height to width ratio of 2 or 1. The maximum bearing stress developed by the piers reached values varying from 150 psi (CBRC piers), to 250 psi (HCBL and HCBR piers), which were significantly smaller than the maximum bearing stresses obtained with the more slender pier tests. Nevertheless, it is expected that the additional bearing stress, (in excess of the initial bearing stress of 50 psi), contributed to an increase of the ultimate strength of the piers exhibiting either a shear or a sliding mode of failure.

Since these squat piers never experienced the beginning of a flexural mode of failure, (Section 4.2), it is not possible to repeat the analysis performed for the piers with other height to width ratios<sup>[4,2]</sup> to obtain the amount of the horizontal load that can be associated with the additional compressive load developed by the test setup.

### 5.8 Correlation Between Square Panel and Pier Critical Tensile Strengths

This analysis is presented in Table 5.3; it is discussed in more detail in reference[9]. The purpose of this investigation is to evaluate an alternative and more appropriate test procedure for determining the code allowable shear strength of masonry walls.

Currently, the code allowable shear strength is based on the compressive strength of a masonry prism.

The square panel critical tensile strength has been determined from a study made by Blume<sup>[1]</sup>, who proposed the expression shown in Table 5.3. The ultimate load P was taken as the average value obtained from three square panel tests for each type of pier, as indicated in Table 2.2. The square panel test results for the HCBR piers were not available, as explained in Section 2.2. Therefore, the correlation is presented for the HCBL and CBRC piers only.

The critical tensile strength of the piers has been computed at the neutral axis of the pier sections, following the simple beam theory for a section under combined flexure, shear and axial force. A parabolic distribution of shear stresses over the cross section has been assumed. The piers developed their first major diagonal shear crack before the ultimate strength was attained, as mentioned in Section 5.3. The shear crack strength (lateral load required to develop the first major diagonal crack), and its corresponding compressive load (Table 5.1) have been used to evaluate the pier critical tensile strength.

In spite of the squatness of the piers, the results shown in the last column of Table 5.3 indicate that the pier critical tensile strength has been accurately estimated. In fact, the correlation obtained is better than that obtained for piers with larger height to width ratios (Tables 5.2 and 5.3 in references [2] and [4], respectively). A comparison between the square panel test and the prism compressive test to predict the shear strength of masonry walls will be presented in a future report containing the overall results of the single pier test program.

### 5.9 Other Test Results

The last graph in the test results (Appendix A) is a comparison between the relative lateral displacement of the piers, as measured by  $H_1-H_2$  (Fig. 3.3) and the percentage of this displacement that can be attributed to shear distortion as defined in Fig. 3.4. These results reflect the amount of diagonal cracking present at each stage of the test, and corroborate the observation presented in Section 4.2 that the number of diagonal cracks developed during the tests increased from the CBRC piers, to the HCBR piers, to the HCBL piers.

It is also interesting to comment on the shear distortion presented by the specimens HCBR-12-1, CBRC-12-4 and CBRC-12-5 toward the end of the tests. These piers had a combined flexural and sliding failure. When sliding began to occur through the bottom course of the specimens, most of the lateral displacement took place at the bottom course of the piers and  $H_1-H_2$  began to decrease compared with previous stages. Nevertheless, the percentage of shear distortion continued to increase.

For these squat piers, the flexural and shear components of the deformation used to compute the initial stiffness (Fig. 4.3) are in the ratio of 1:11. The greater importance of the shear deformation of these piers in comparison with the importance of the shear deformation of piers with larger height to width ratio<sup>[2,4]</sup> is reflected in the larger amount of shear distortion exhibited by the squat piers.

Finally, it is appropriate to report on how well the test rig reproduced the fixed end condition at the top of the pier. There are two measures of the rotation of the top section; one is an absolute measure obtained with the instruments placed at the top of the pier, (DCDT'S  $V_1$  and  $V_2$  in Fig. 3.3), and the other is the computation of the

location of the inflection point from the forces acting on the pier. The results of these measurements during the early stages of the tests show that the absolute rotation of the top spandrel was confined to values smaller than  $0.008^\circ$ ; however, these rotations are very small compared to the precision of the instruments and no valid conclusions can be drawn from them. On the other hand, the position of the inflection point throughout the tests never rose more than 12 inches (30% of the pier height) from the mid-height section of the pier.



TABLE 5.1

COMPARISON OF SHEAR CRACK STRENGTH AND ULTIMATE STRENGTH  
(Average of extreme values during any one cycle considered)

Specimen	Grouting Full (F) Solid (S)	Initial Bearing Stress (psi)	Vertical Reinforcement No. Bars	Horizontal Reinforcement		Shear Crack Strength		Compressive Stress at Shear Crack (psi)	Peak Ultimate Shear		Compressive Stress at Ultimate (psi)	Peak Ult. Shear Shear Cr. Strength
				No. Bars	$A_{hs} f_y$ (kip)	Force (kip)	Stress (psi)		Force (kip)	Stress (psi)		
HCBL-12-1	F	52	3#7	—	—	121.7	200	85	200.3	328	194	1.65
-2	F	52	3#7	1#5	21.6	125.4	206	86	211.7	347	200	1.69
-3	F	52	3#7	2#5	43.2	130.9	215	83	251.4	412	243	1.92
-4	F	52	3#7	3#5	64.7	159.1	261	127	218.6	358	212	1.37 *
-5	F	52	3#7	4#5	86.3	137.9	226	106	228.0	374	215	1.65 *
-6	F	52	3#7	4#6	118.4	148.9	244	102	261.7	429	234	1.76
HCBR-12-1	F	56	3#7	—	—	Did not have a shear crack			220.8	384	176	— **
-2	F	56	3#7	1#6	29.6	183.6	319	125	191.0	332	149	1.04
-3	F	56	3#7	2#6	59.2	202.1	351	150	220.8	384	198	1.09
-4	F	56	3#7	3#6	88.8	204.7	356	143	255.3	444	248	1.25
-5	F	56	3#7	4#6	118.4	226.8	394	154	232.7	404	175	1.03 **
-6	F	56	3#7	5#7	240.9	225.7	392	153	259.0	450	223	1.15 **
CBRC-12-1	S	50	3#7	—	—	197.2	253	108	197.2	253	108	1.00 *
-2	S	50	3#7	1#6	29.6	194.8	250	127	194.8	250	127	1.00 *
-3	S	50	3#7	2#6	59.2	214.6	275	138	217.3	279	150	1.01 *
-4	S	50	3#7	3#6	88.8	Did not have a shear crack			235.0	301	123	— **
-5	S	50	3#7	4#6	118.4	179.8	231	116	192.3	247	141	1.07 **
-6	S	50	3#7	5#7	240.9	187.5	240	94	216.1	277	142	1.15

\* Piers with a combined shear and sliding failure

\*\* Piers with a combined flexural and sliding failure

TABLE 5.2

## EFFECT OF SHEAR STRESS AND STEEL REINFORCEMENT ON STIFFNESS DEGRADATION

Specimen	Grouting Full (F) Solid (S)	Vertical Steel Reinforcement	Horizontal Steel Reinforcement	Theoretical Initial Stiffness (kip/in)	Measured Maximum Initial Stiffness (kip/in)	Stiffness at 50 psi		Stiffness at 75 psi		Stiffness at 100 psi		Stiffness at 150 psi	
						Measured (kip/in)	Percentage Decrease (%)	Measured (kip/in)	Percentage Decrease (%)	Measured (kip/in)	Percentage Decrease (%)	Measured (kip/in)	Percentage Decrease (%)
HCBL-12-1	F	3#7	No	5776	4353	*	*	4209	3	3549	18	2955	32
-2	F	3#7	1#5	5776	5096	4927	3	4504	12	3882	24	3286	36
-3	F	3#7	2#5	5776	5546	5423	2	4913	11	4780	14	3723	33
-4	F	3#7	3#5	5776	5693	5588	2	4855	15	4446	22	3450	39
-5	F	3#7	4#5	5776	5098	4471	12	4182	18	3976	22	3131	39
-6	F	3#7	4#6	5776	—	—	—	—	—	—	—	—	—
HCBR-12-1	F	3#7	No	11655	8874	7131	20	6169	30	5418	39	4450	50
-2	F	3#7	1#6	11655	7367	6951	6	6650	10	5848	21	4802	35
-3	F	3#7	2#6	11655	10902	9227	15	8107	26	6369	42	4501	59
-4	F	3#7	3#6	11655	8585	7978	7	7622	11	6659	22	5120	40
-5	F	3#7	4#6	11655	7957	7579	5	6662	16	6320	21	5191	35
-6	F	3#7	5#7	11655	5751	4937	14	4700	18	4408	23	3756	35
CBRC-12-1	S	3#7	No	11095	6971	6841	2	5719	18	5020	28	3839	45
-2	S	3#7	1#6	11095	9642	7061	27	5851	39	4716	51	3469	64
-3	S	3#7	2#6	11095	6152	6031	2	5670	8	4932	20	3985	35
-4	S	3#7	3#6	11095	9456	7163	24	6449	32	5423	43	4503	52
-5	S	3#7	4#6	11095	10443	6300	40	5253	50	3887	63	2762	74
-6	S	3#7	5#7	11095	6907	4533	34	5392	22	4819	30	3714	46

\* Maximum initial stiffness obtained after 50 psi

TABLE 5.3

CORRELATION BETWEEN SQUARE PANEL AND PIER CRITICAL TENSILE STRENGTH  
(Average of extreme values during any one cycle considered)

Specimen	SQUARE PANEL (1)				PIER (2)						$\frac{\sigma_{tcr}^o}{\sigma_{tcr}}$
	Ultimate Load P(kip)	Side Area A(in <sup>2</sup> )	$\tau = \frac{P}{\sqrt{2A}}$ (psi)	Blume's Formula $\sigma_{tcr}^o = 0.734\tau$ (psi)	Shear Crack Force P(kip)	Compressive Load at Shear Crack N(kip)	Cross Section A(in <sup>2</sup> )	Shear Crack Stress $\frac{P}{A}$ (psi)	Bearing Stress at Shear Crack $\frac{N}{A}$ (psi)	Critical Strength $\sigma_{tcr}$ (psi)	
HCBL-12-1	155.0	244	449.2	329.7	121.7	51.8	610.0	199.5	84.9	259.8	1.27
-2	155.0	244	449.2	329.7	125.4	52.6		205.6	86.2	268.2	1.23
-3	155.0	244	449.2	329.7	130.9	50.6		214.6	83.0	283.1	1.16
-4	155.0	244	449.2	329.7	159.1	77.3		260.8	126.7	333.0	0.99
-5	155.0	244	449.2	329.7	137.9	64.8		226.1	106.2	290.1	1.14
-6	155.0	244	449.2	329.7	148.9	62.1		244.1	101.8	318.8	1.03
CBRC-12-1	186.3	360	365.9	268.6	197.2	83.9	780.0	252.8	107.6	329.2	0.82
-2	186.3	360	365.9	268.6	194.8	98.9		249.7	126.8	316.5	0.85
-3	186.3	360	365.9	268.6	214.6	107.9		275.1	138.3	349.3	0.77
-4	186.3	360	365.9	268.6	Did not have a shear crack			—	—	—	—
-5	186.3	360	365.9	268.6	179.8	90.5		230.5	116.0	292.6	0.92
-6	186.3	360	365.9	268.6	187.5	73.6		240.4	94.4	316.5	0.85

(1) Square Panel Critical Tensile Strength

$$\text{Blume's formula: } \sigma_{tcr}^o = -0.582 \frac{P}{A} - \frac{\sigma_c}{2} + \frac{1}{2} \sqrt{4.849 \left(\frac{P}{A}\right)^2 + \sigma_c^2}$$

$$\text{If edge pressure } \sigma_c = 0, \quad \sigma_{tcr}^o = 0.734 \frac{P}{\sqrt{2A}}$$

(2) Pier Critical Tensile Strength

Assuming a parabolic distribution of shear stresses

$$\sigma_{tcr} = -\frac{\sigma_c}{2} + \sqrt{(1.5\tau)^2 + \left(\frac{\sigma_c}{2}\right)^2}$$

 $\sigma_c = \frac{N}{A}$  : applied compressive stress $\tau = \frac{P}{A}$  : average shear stress

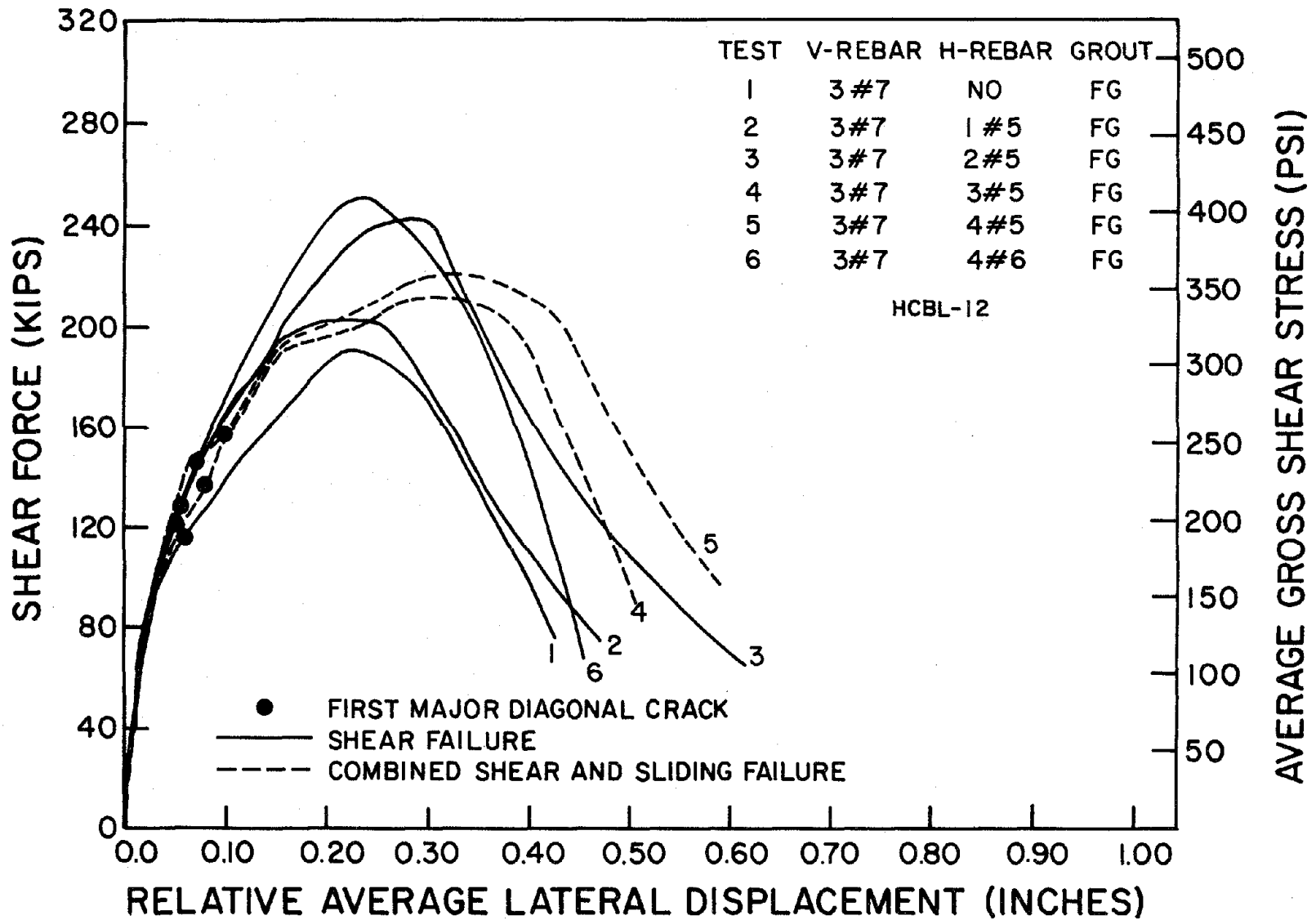


FIG. 5.1 HYSTERESIS ENVELOPES FOR HOLLOW CONCRETE BLOCK PIERS

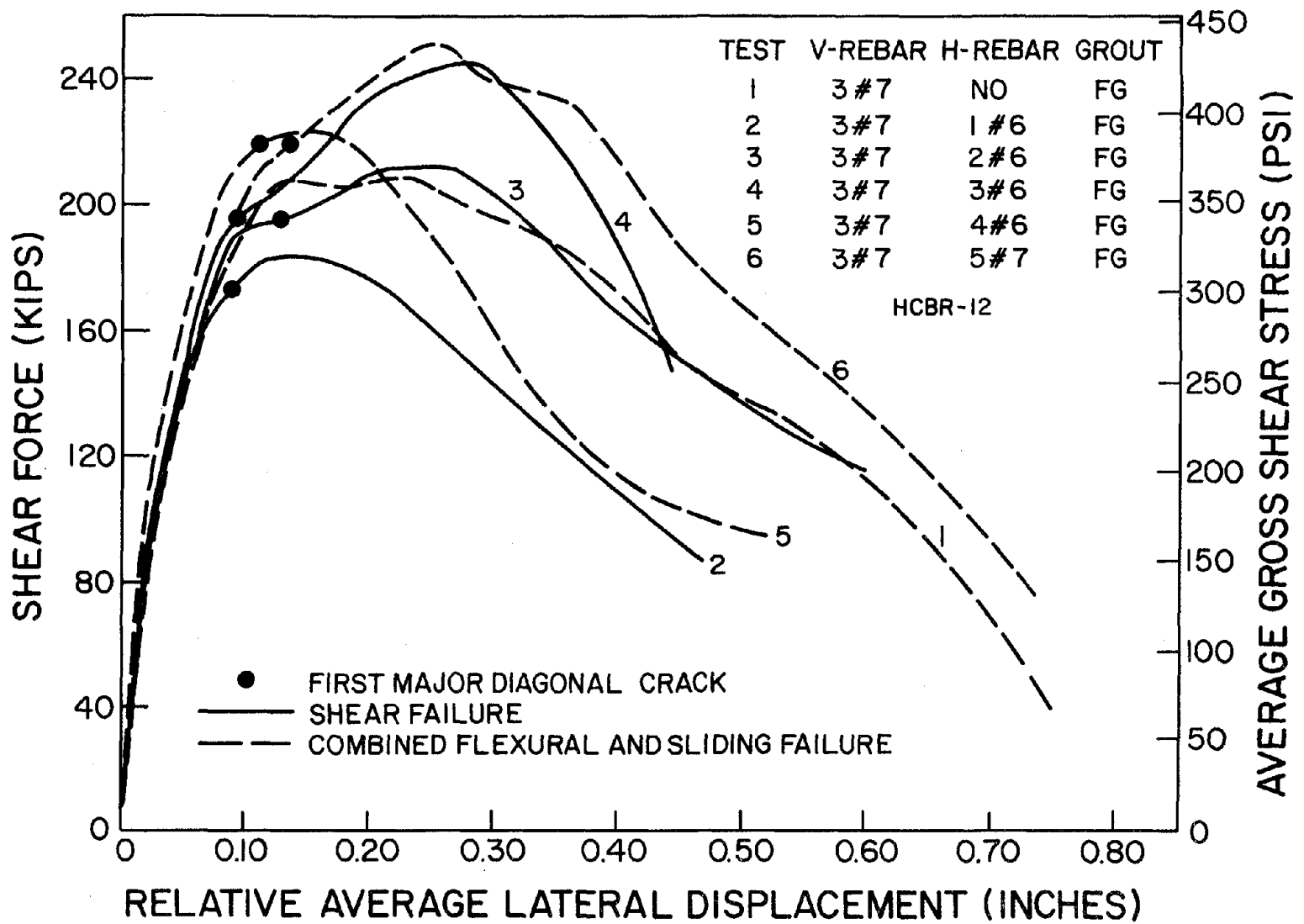


FIG. 5.2 HYSTERESIS ENVELOPES FOR HOLLOW CLAY BRICK PIERS

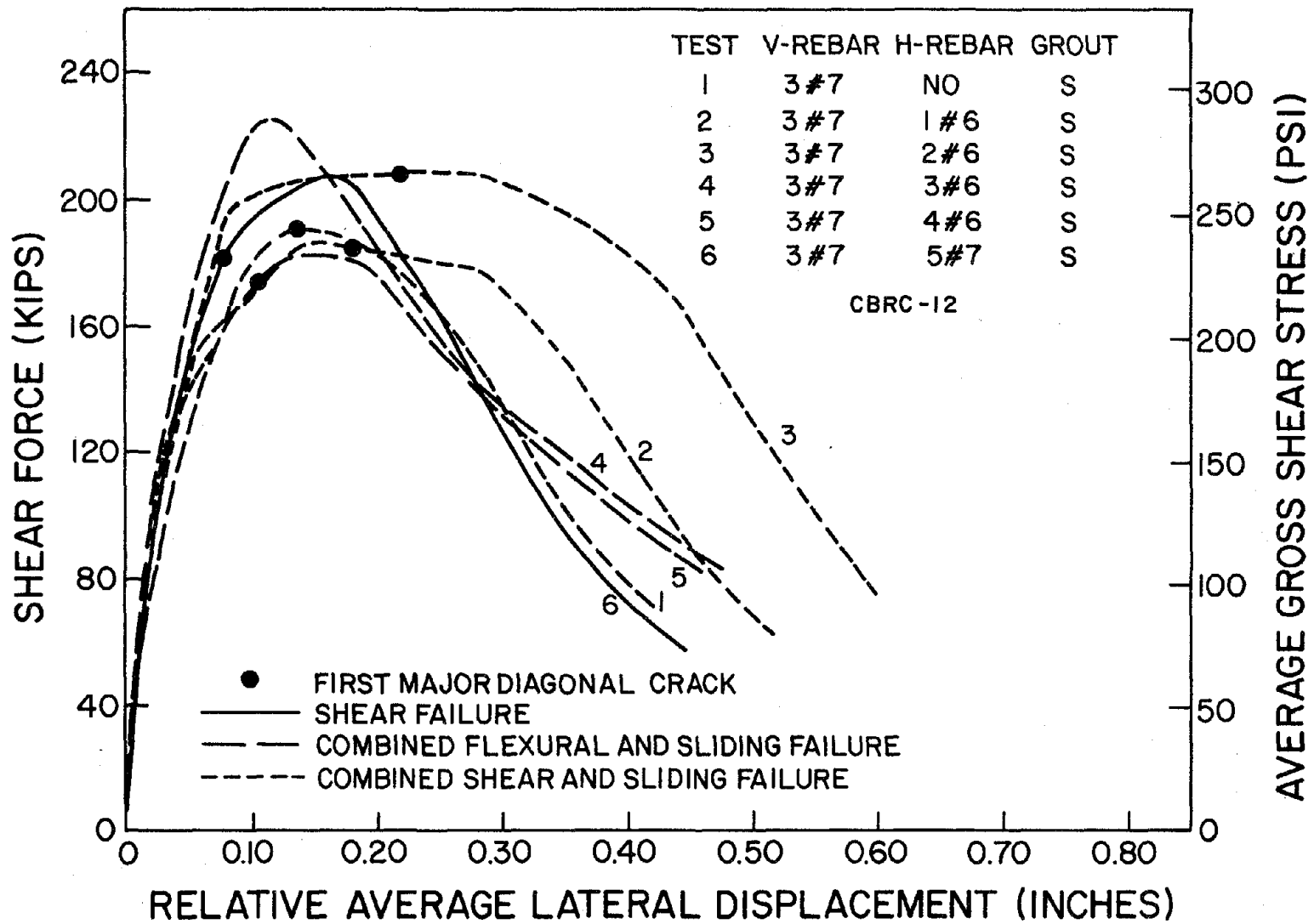


FIG. 5.3 HYSTERESIS ENVELOPES FOR GROUTED CORE CLAY BRICK PIERS

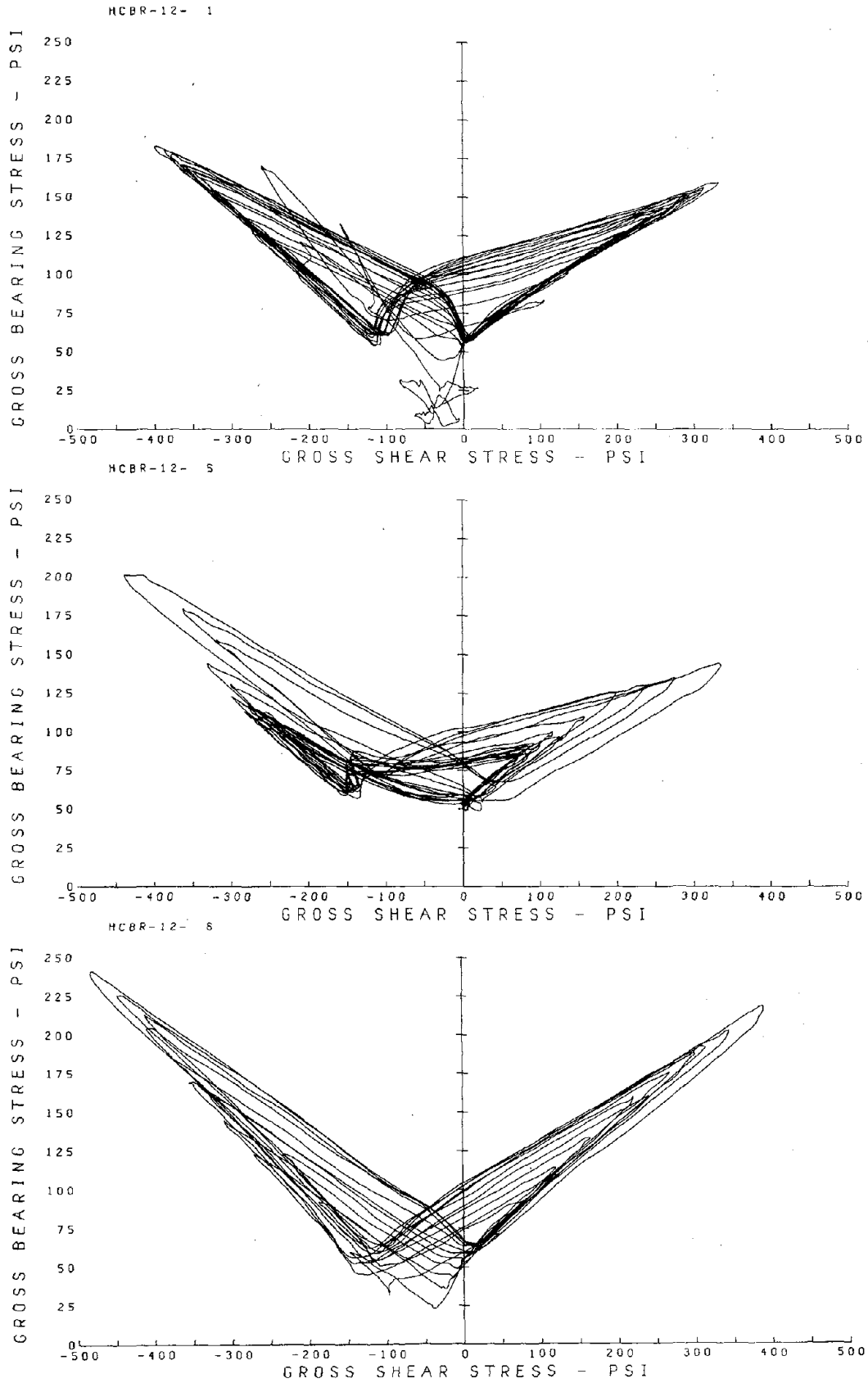


FIG. 5.4(a) SLIDING STRENGTH HCBR PIERS - COMBINED FLEXURAL AND SLIDING MODE OF FAILURE

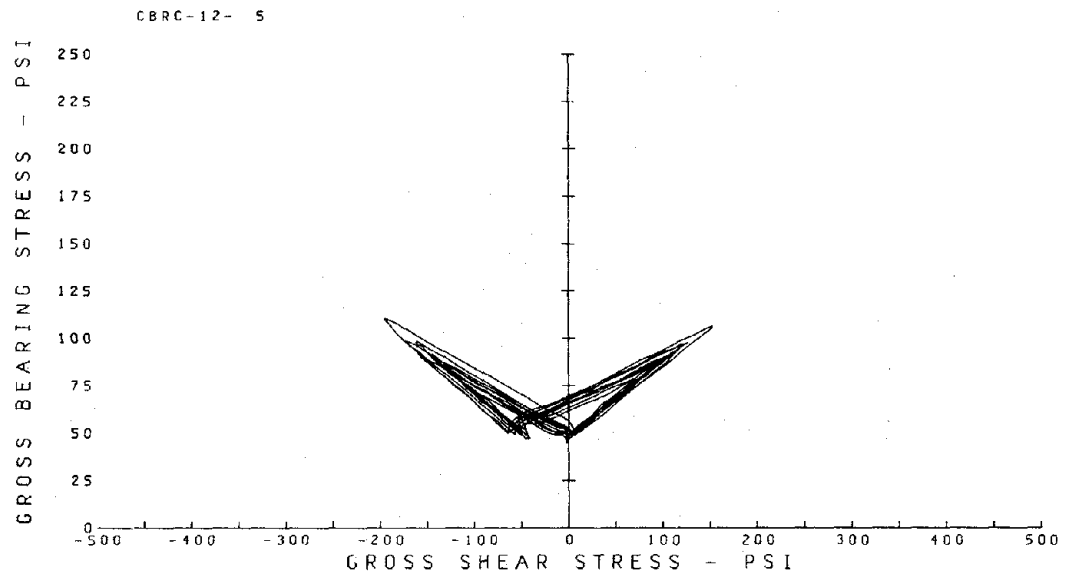
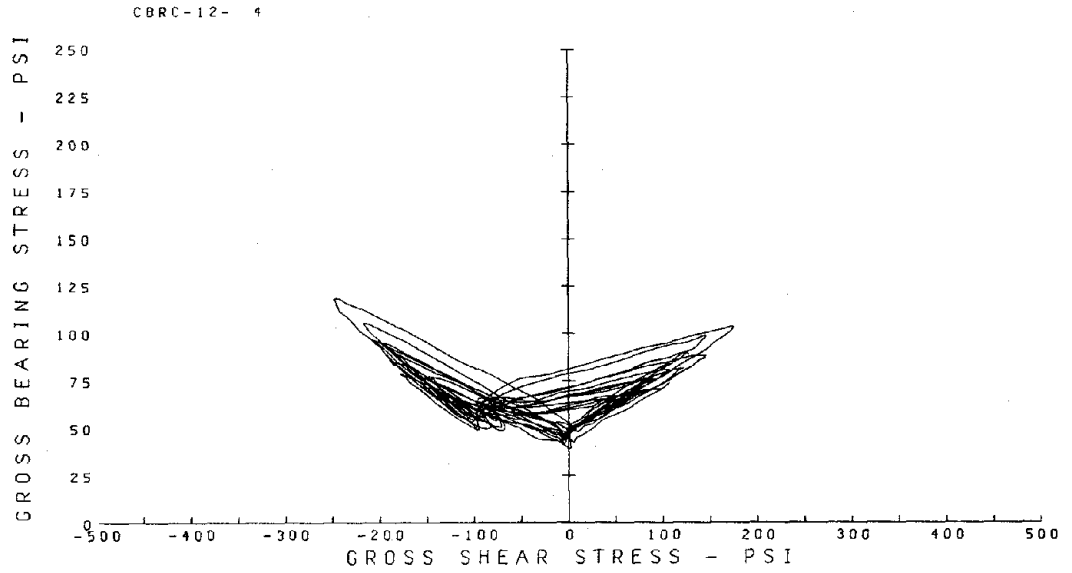


FIG. 5.4(b) SLIDING STRENGTH CBRC PIERS - COMBINED FLEXURAL AND SLIDING MODE OF FAILURE



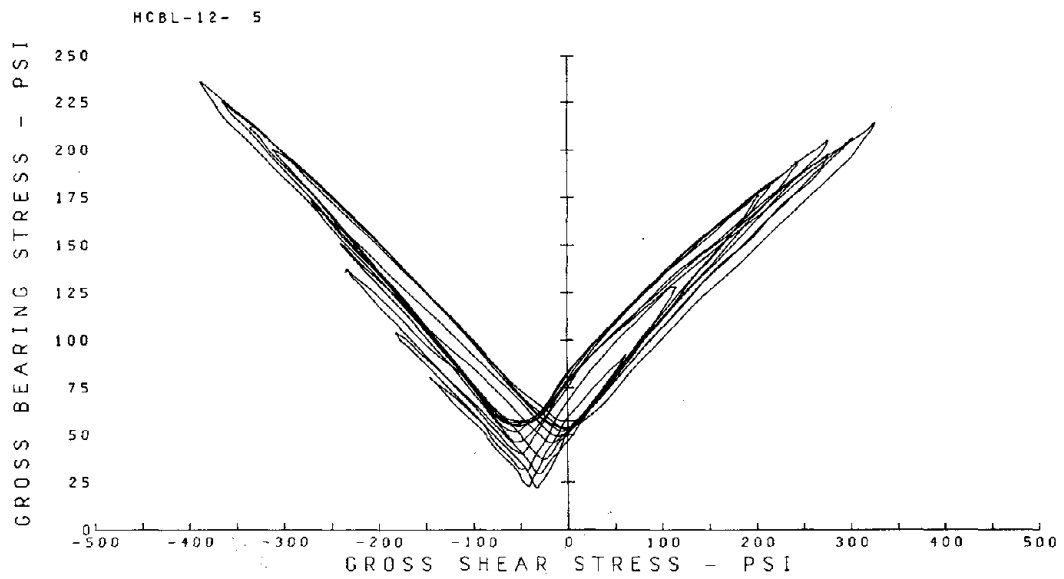
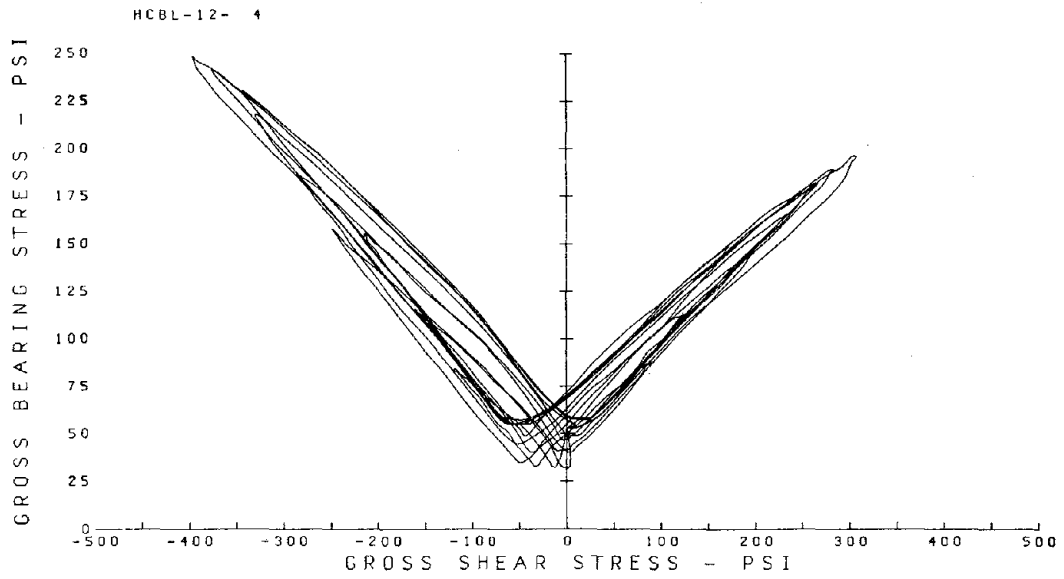


FIG. 5.5(a) SLIDING STRENGTH HCBL PIERS - COMBINED SHEAR AND SLIDING MODE OF FAILURE

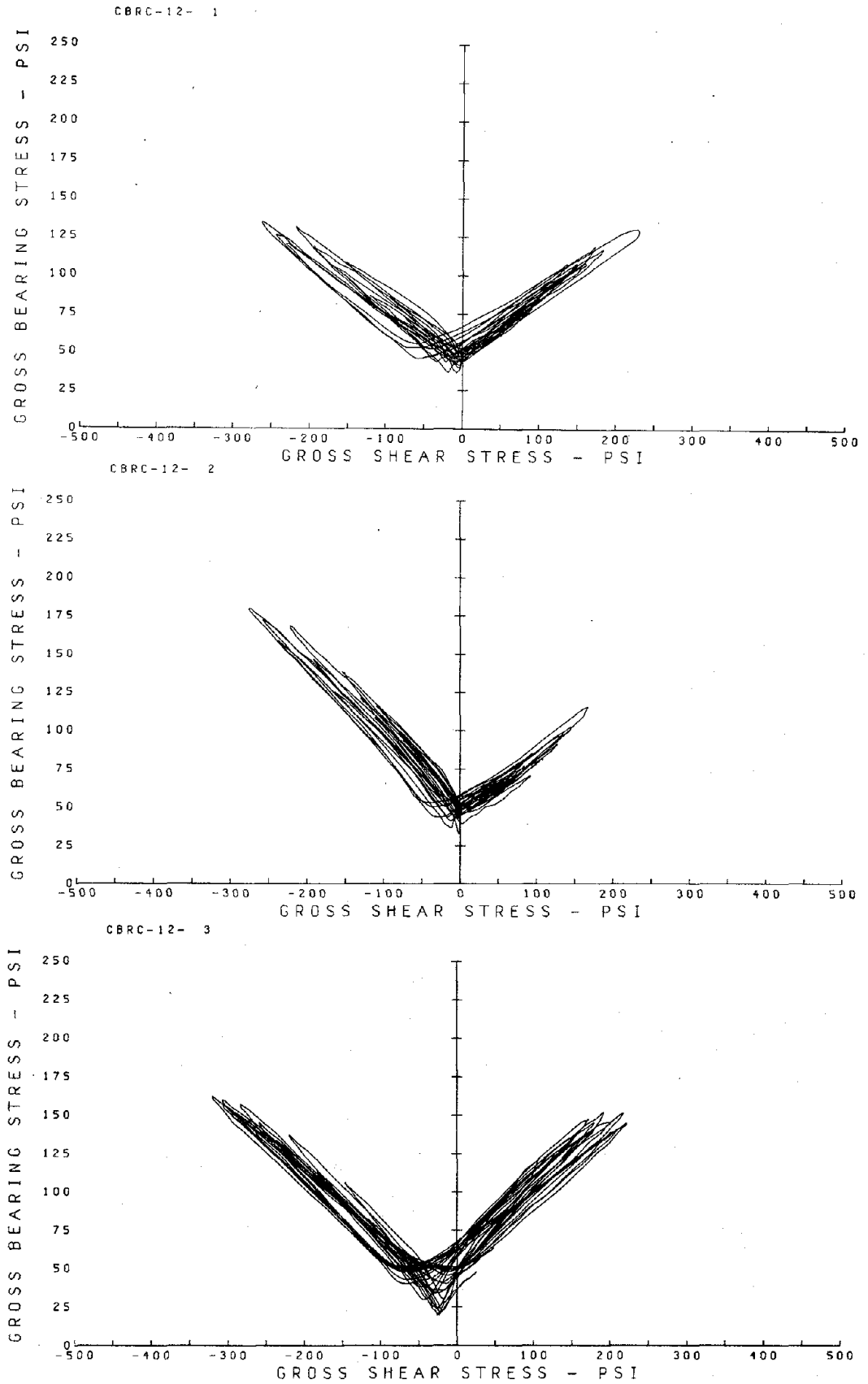


FIG. 5.5(b) SLIDING STRENGTH CBRC PIERS - COMBINED SHEAR AND SLIDING MODE OF FAILURE

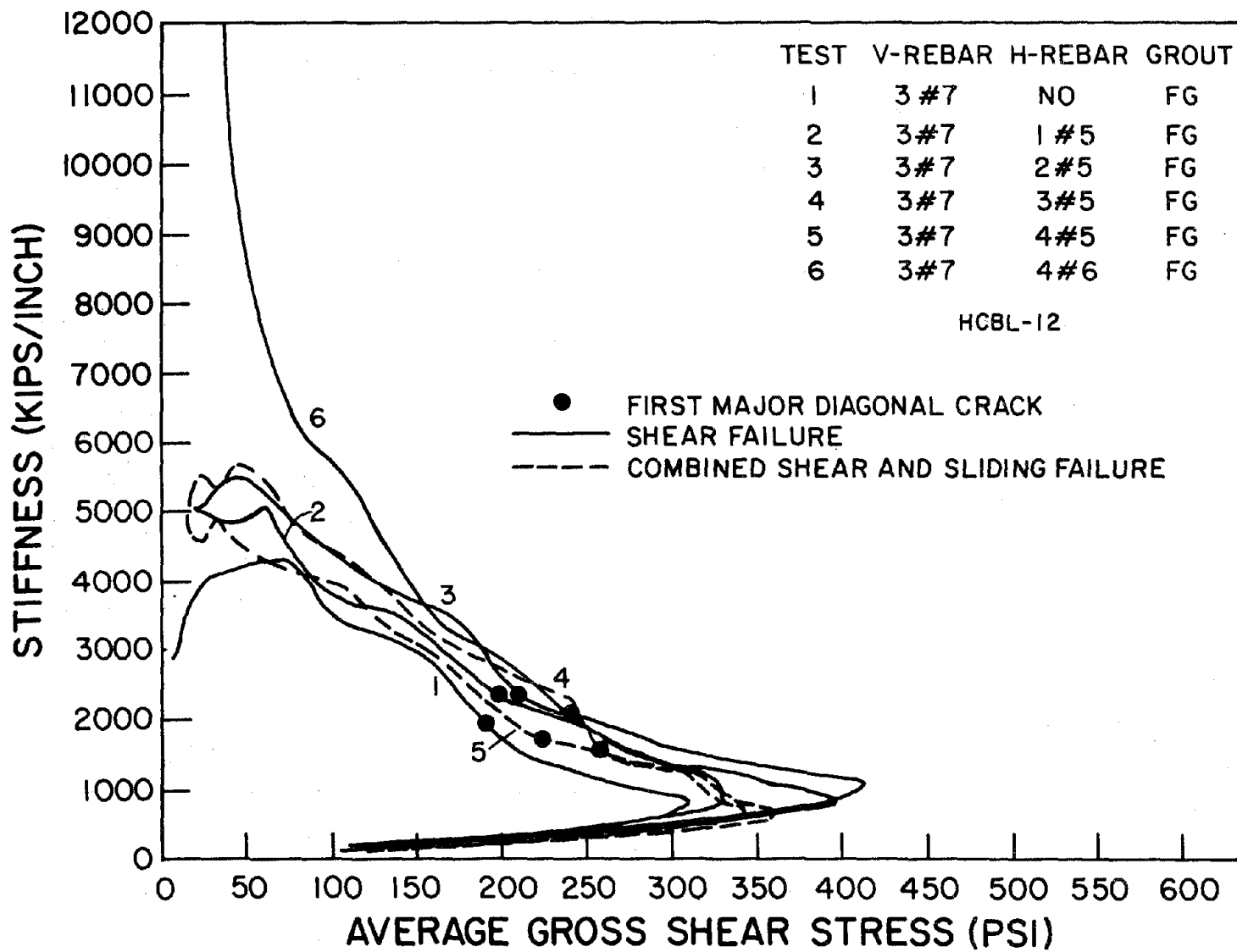


FIG. 5.6 STIFFNESS DEGRADATION FOR HOLLOW CONCRETE BLOCK PIERS

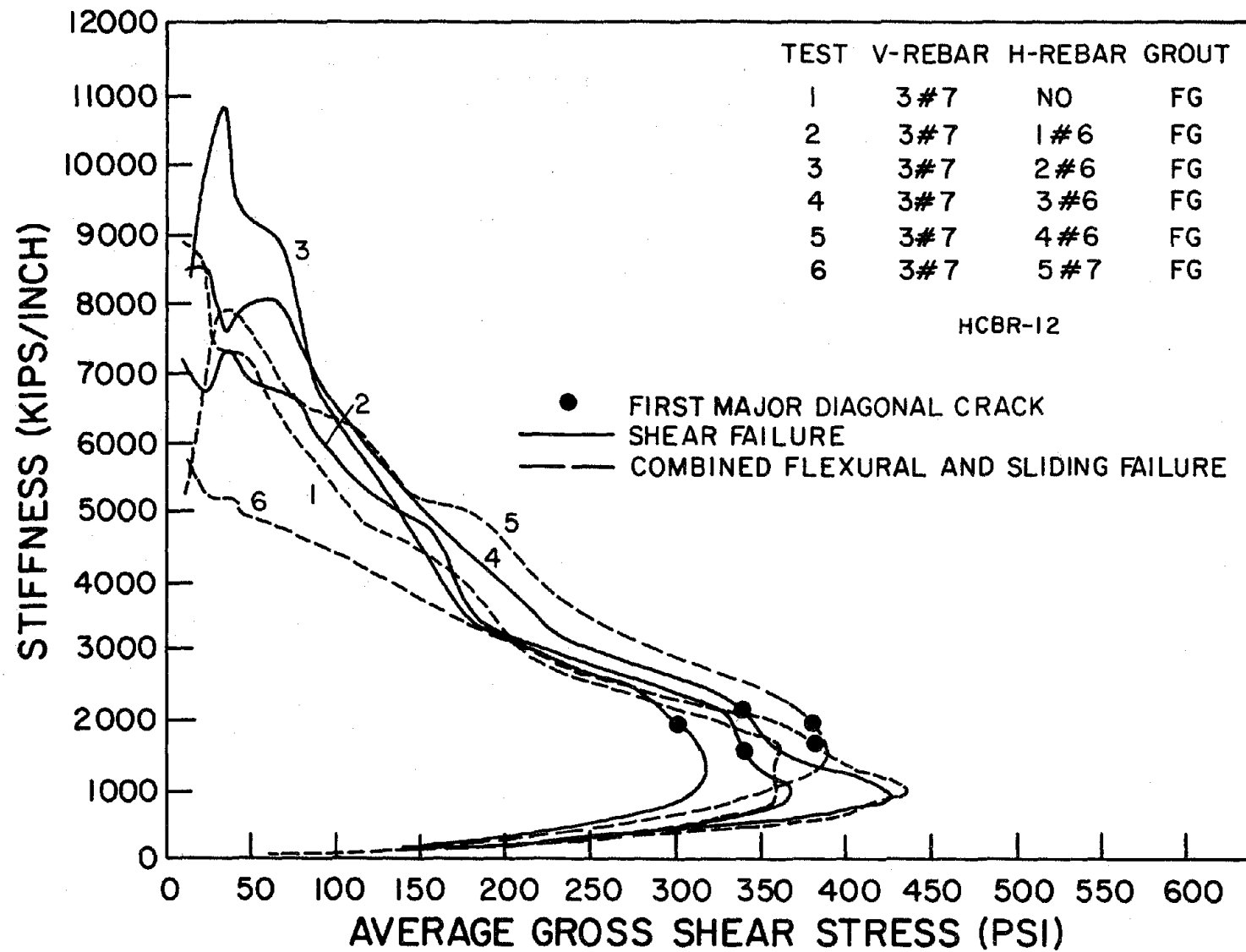


FIG. 5.7 STIFFNESS DEGRADATION FOR HOLLOW CLAY BRICK PIERS

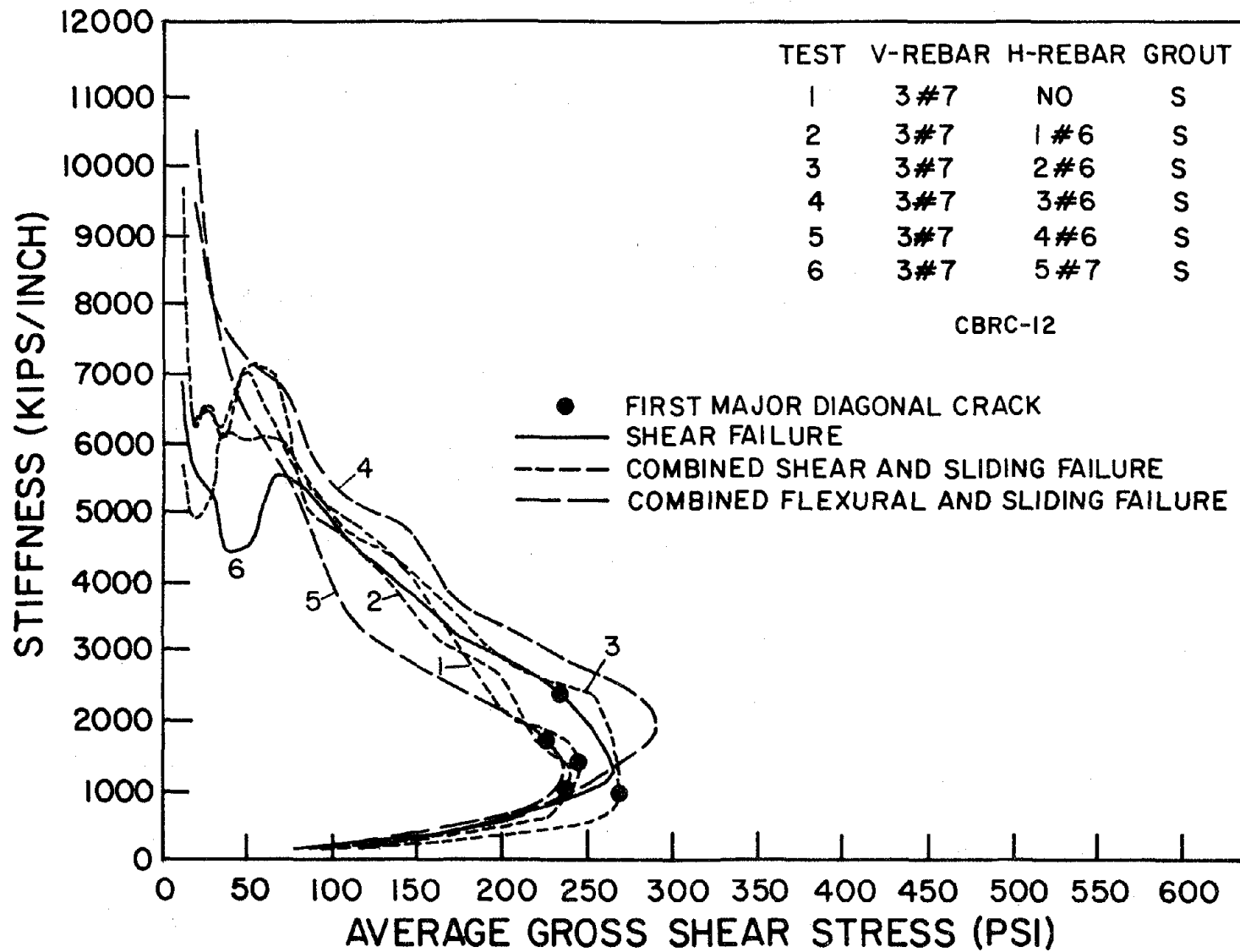


FIG. 5.8 STIFFNESS DEGRADATION FOR GROUTED CORE CLAY BRICK PIERS

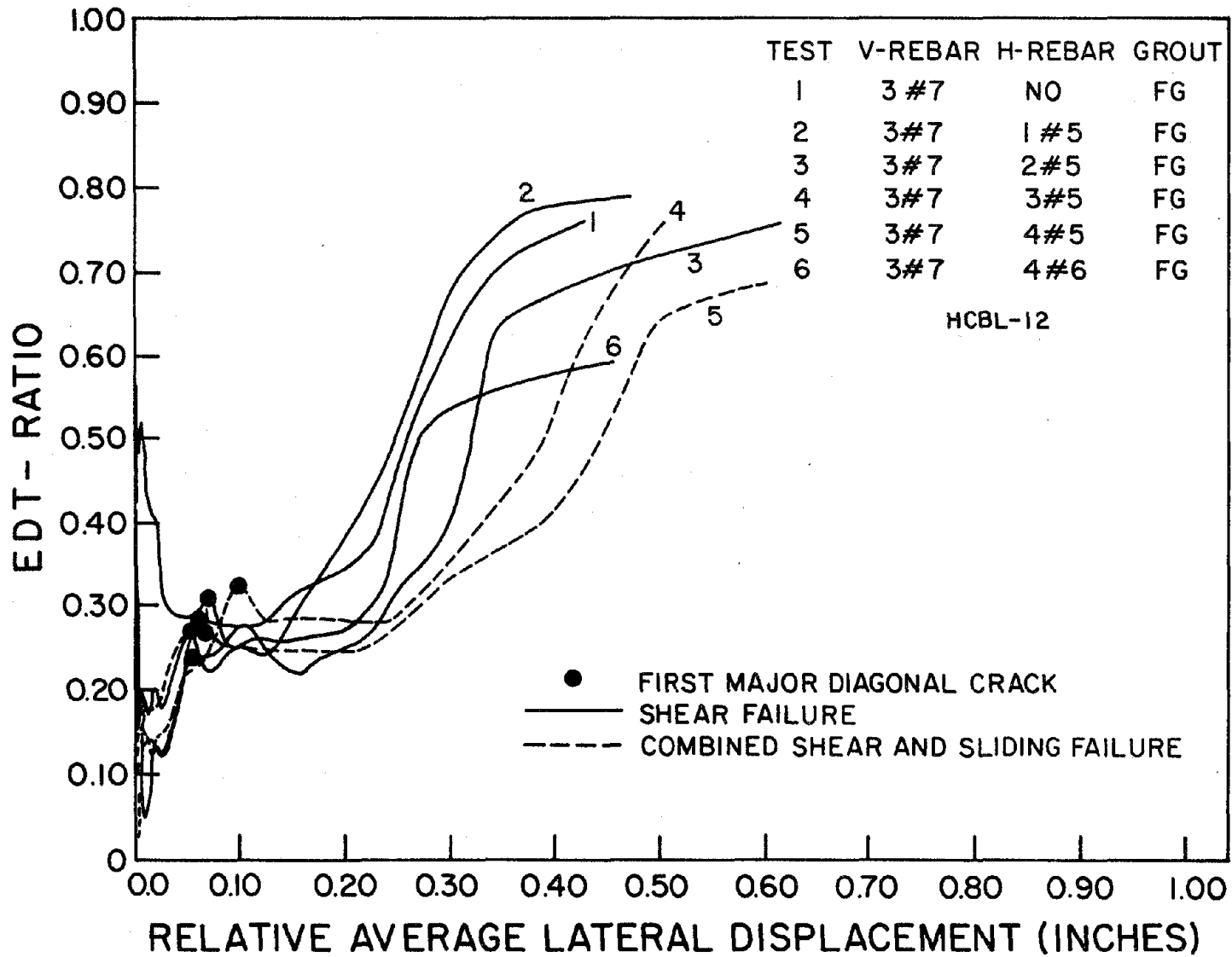


FIG. 5.9 ENERGY DISSIPATION FOR HOLLOW CONCRETE BLOCK PIERS

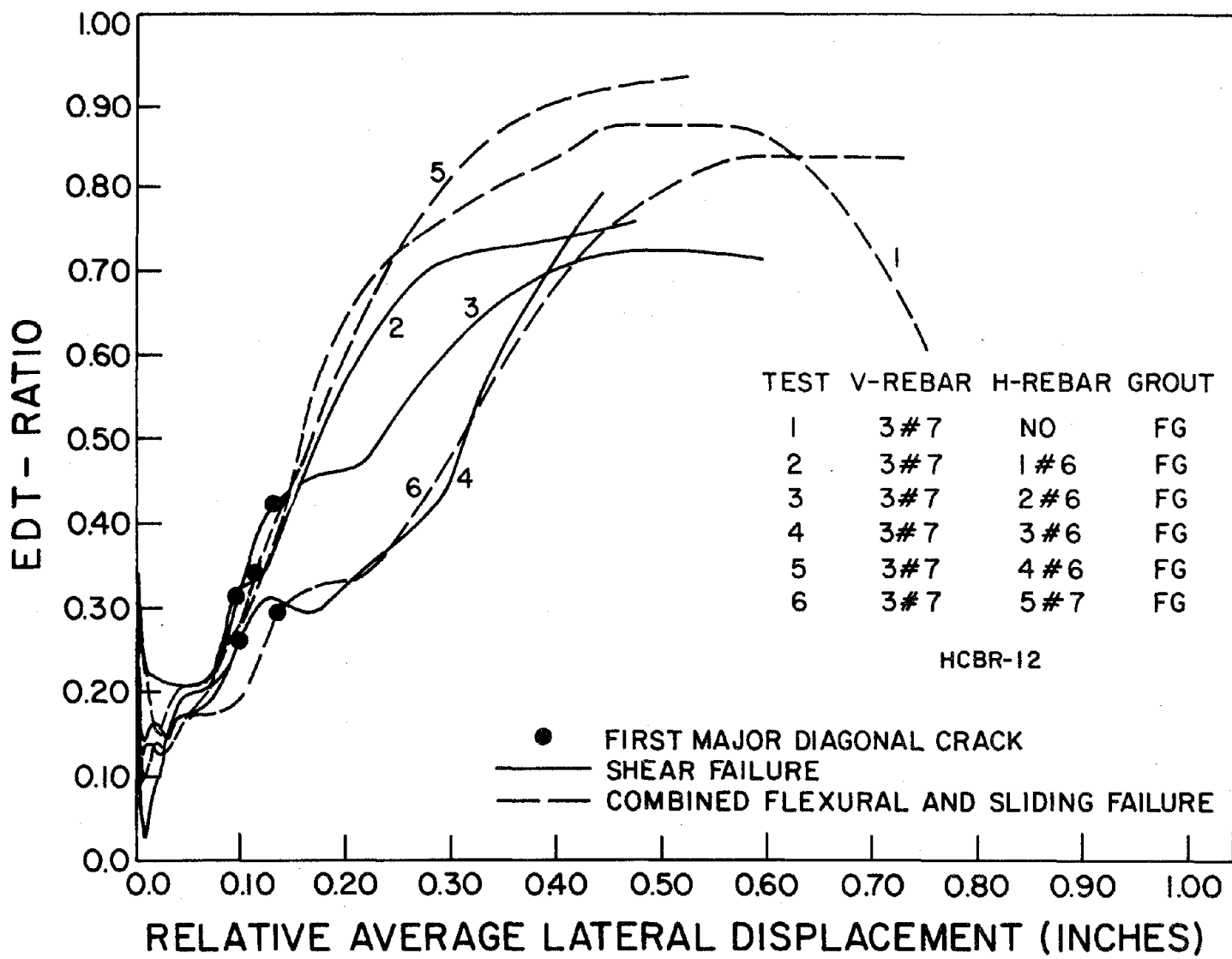


FIG. 5.10 ENERGY DISSIPATION FOR HOLLOW CLAY BRICK PIERS

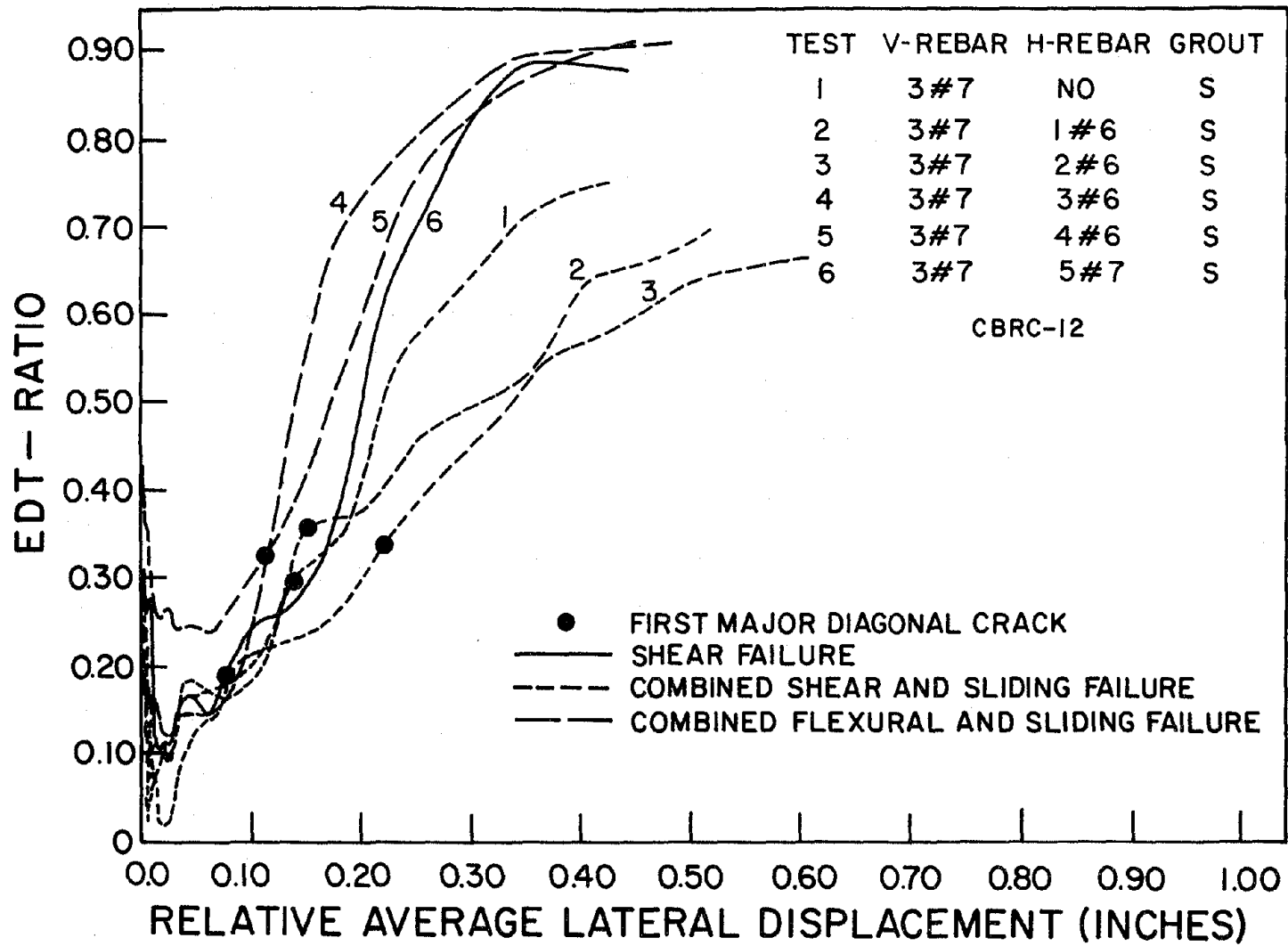


FIG. 5.11 ENERGY DISSIPATION FOR GROUTED CORE CLAY BRICK PIERS



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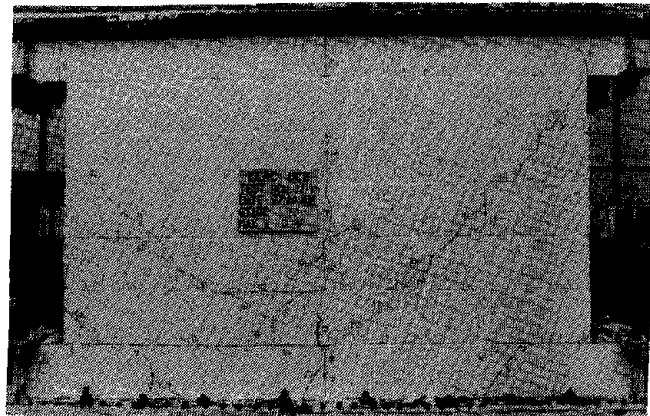
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APPENDIX A  
CATALOG OF TEST RESULTS

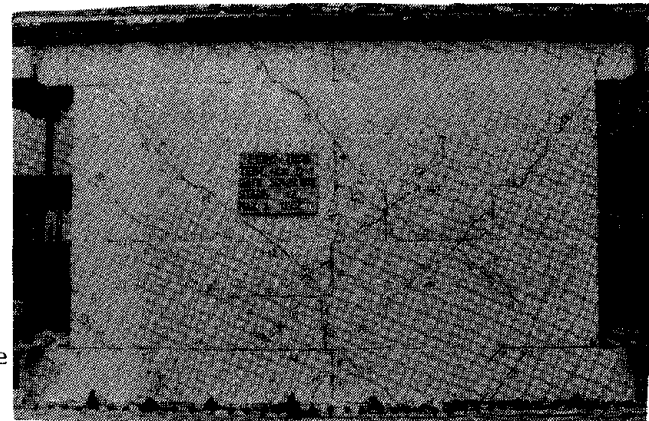
The experimental results for each test are arranged on three pages containing six photographs of the successive crack patterns and six graphs obtained from the data collected during the test. These graphs include the hysteresis loops, the hysteresis envelope, stiffness degradation, energy dissipation and amount of shear distortion as compared with total deformation.

In order to show the relation between the photographs of the crack patterns and the diagrams showing the results, a black dot has been drawn on each of the graphs and next to the corresponding picture of the crack pattern. The dot generally corresponds to the stage at which the first major diagonal crack occurred.

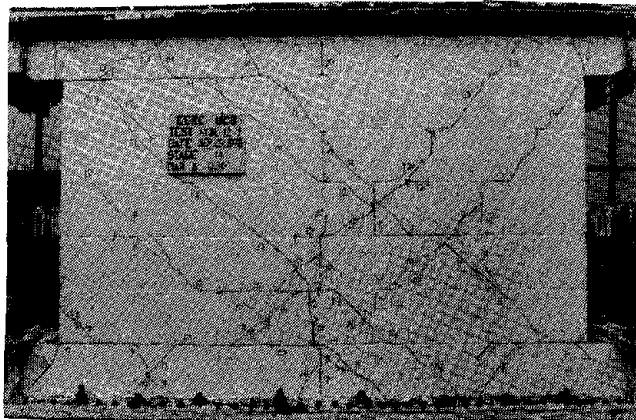
Details on how each of the diagrams was obtained are presented in Chapter 4.



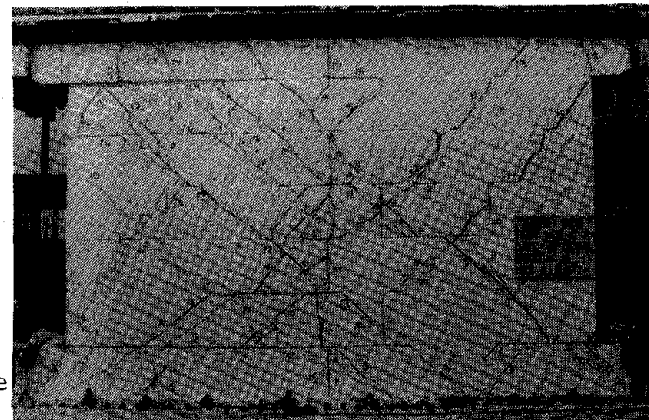
Stage  
10



Stage  
12

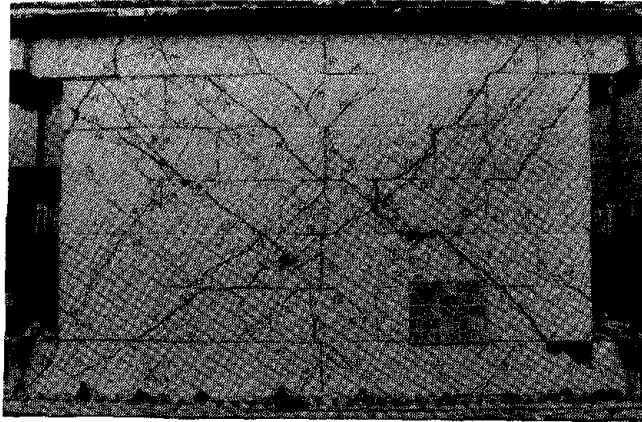


Stage  
14

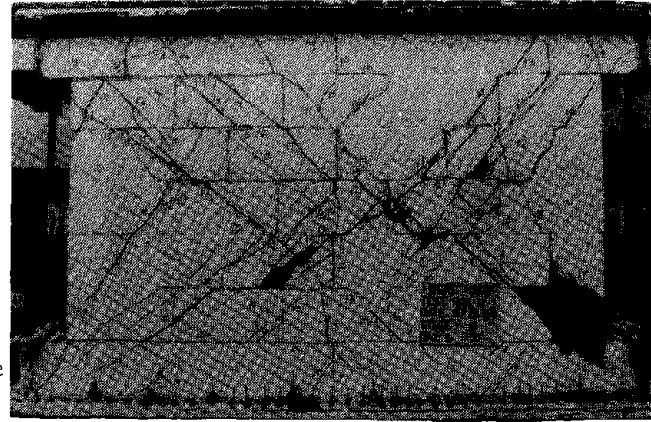


Stage  
16

FIG. A.1 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBL-12-1



Stage  
17



Stage  
18

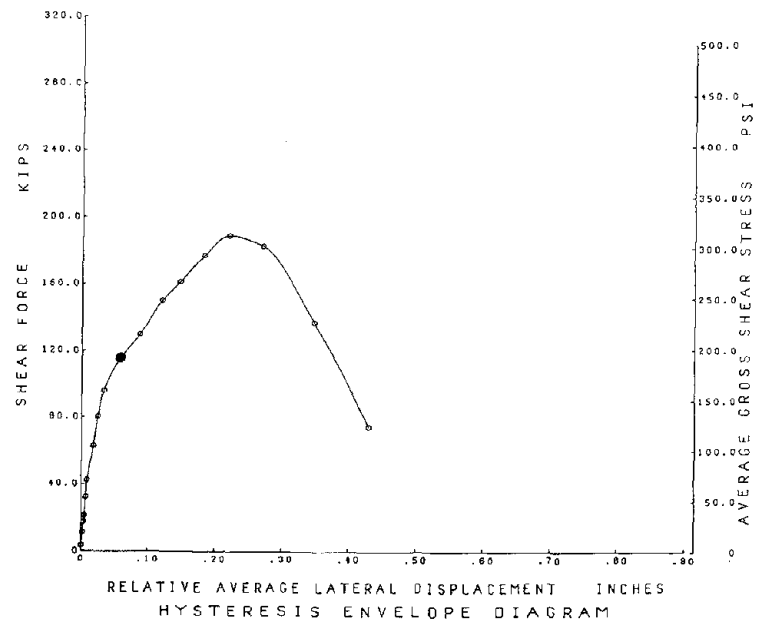
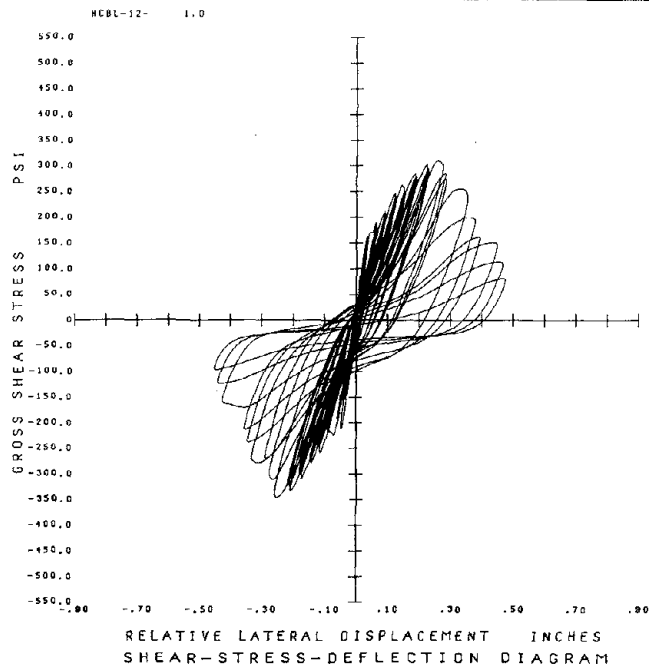


FIG. A.1 CONTINUED HCBL-12-1

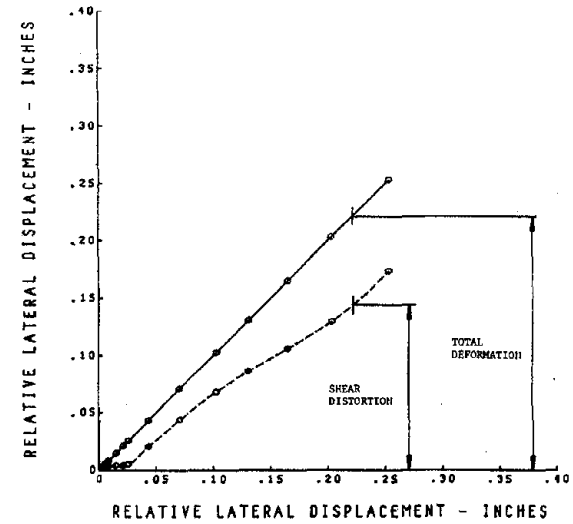
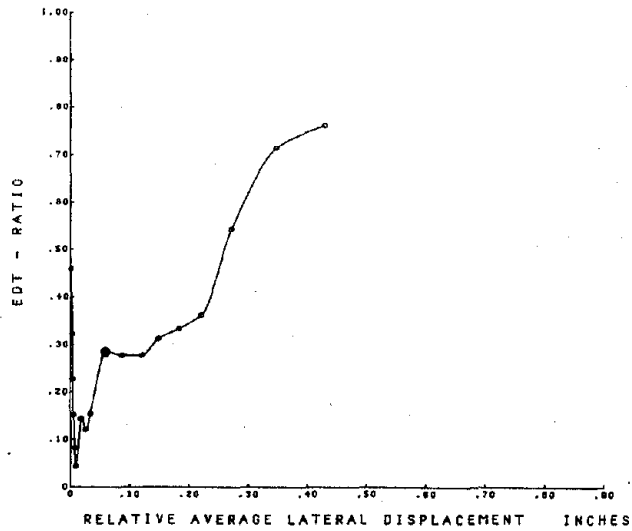
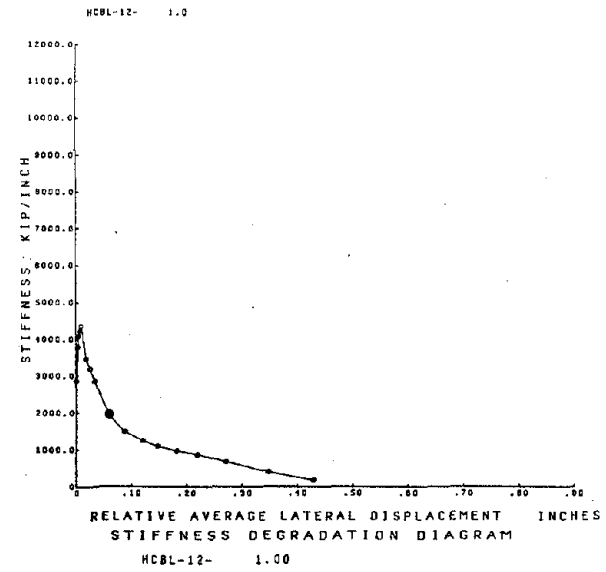
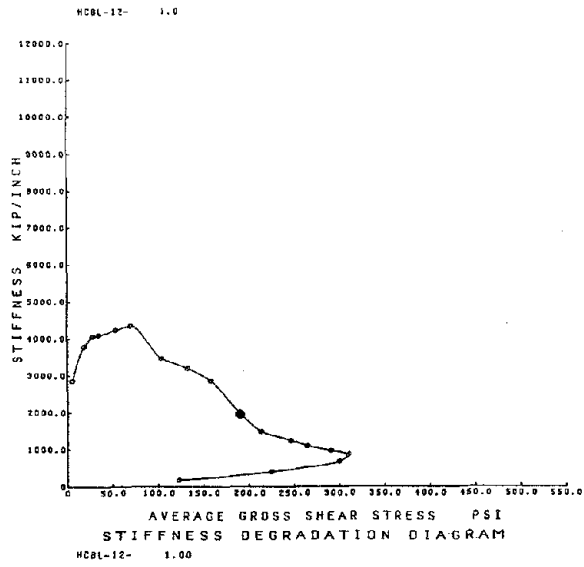
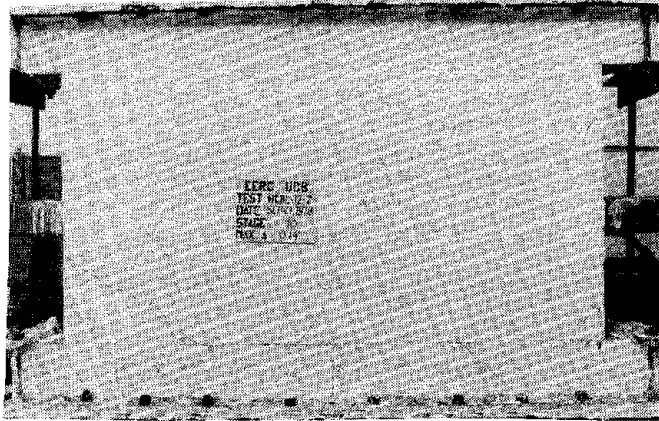
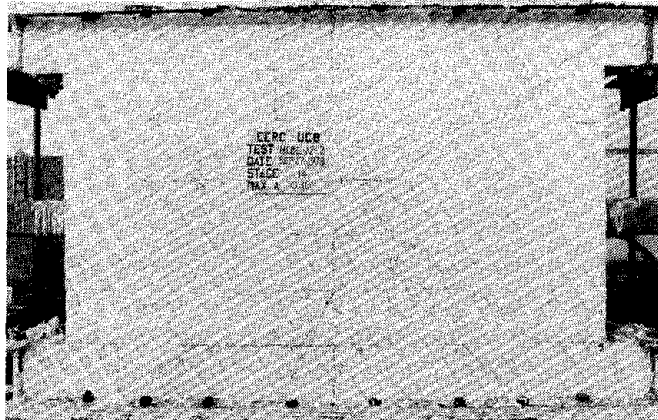
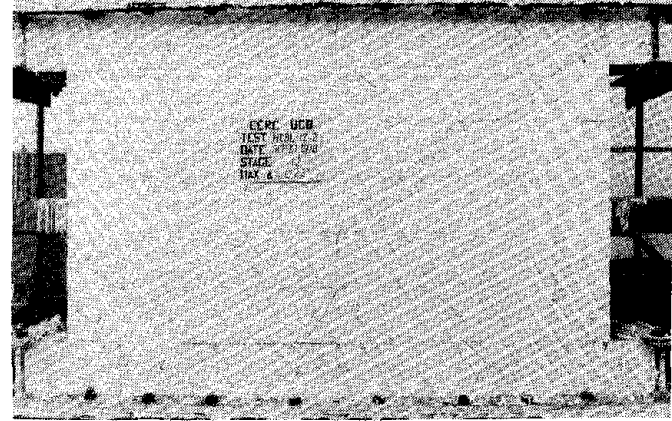


FIG. A.1 CONTINUED HCBL-12-1



●  
Stage 10    Stage 12



Stage 14    Stage 16

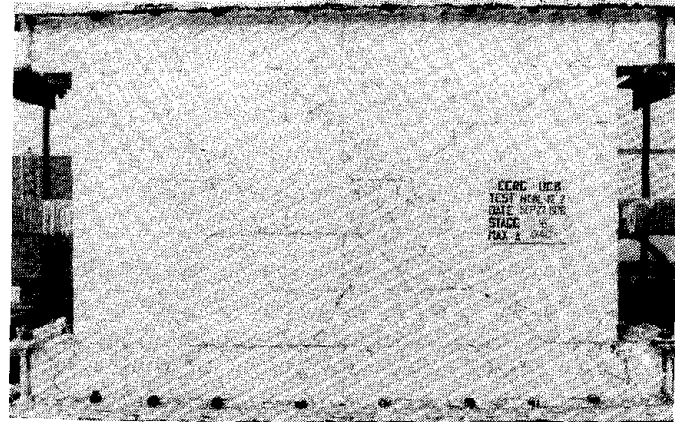
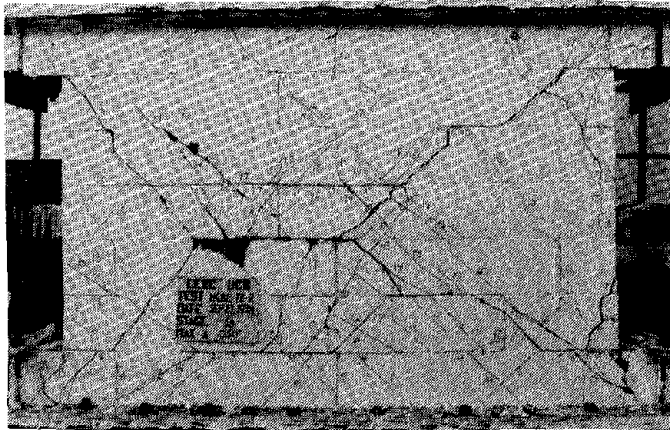


FIG. A.2 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBL-12-2



Stage Stage  
18 19

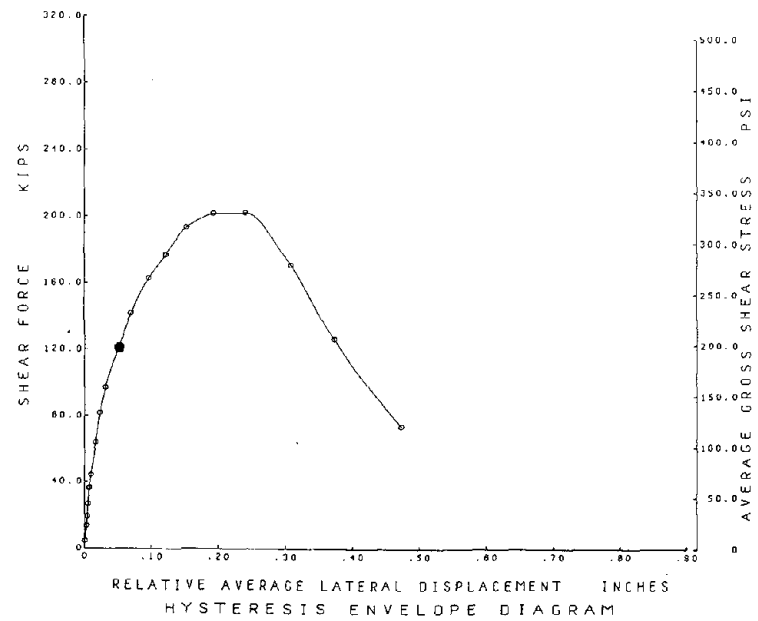
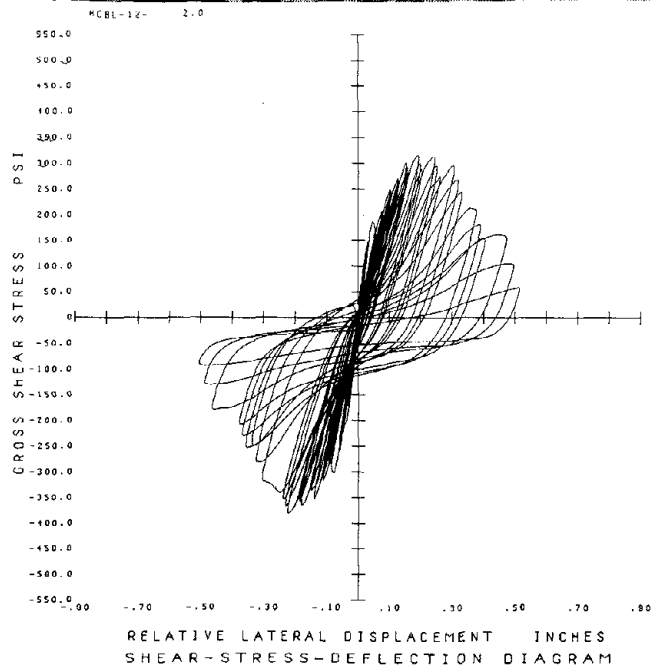


FIG. A.2 CONTINUED HCBL-12-2



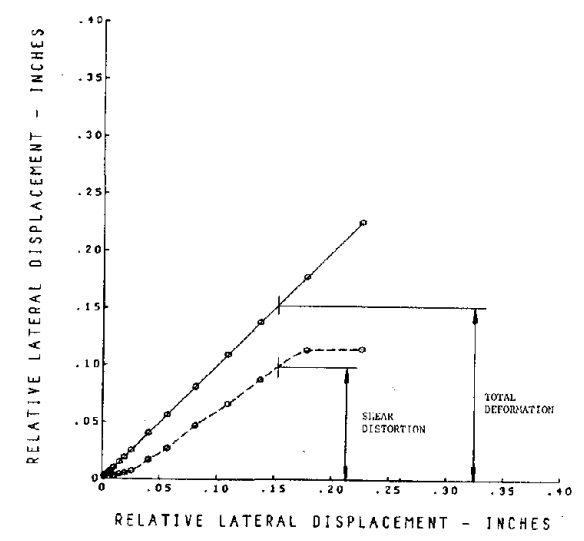
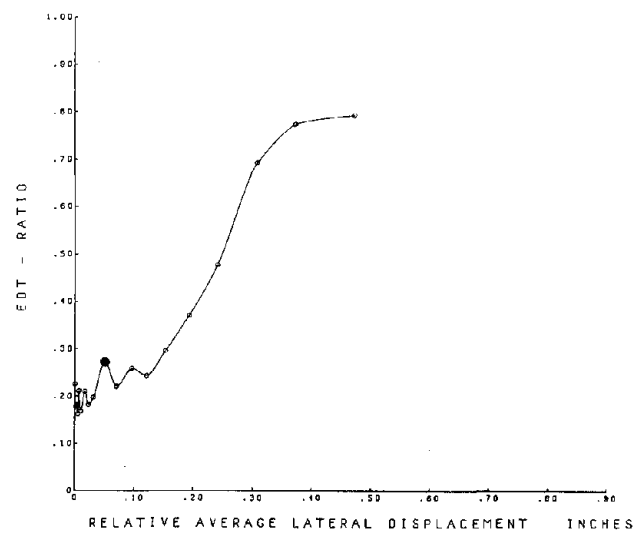
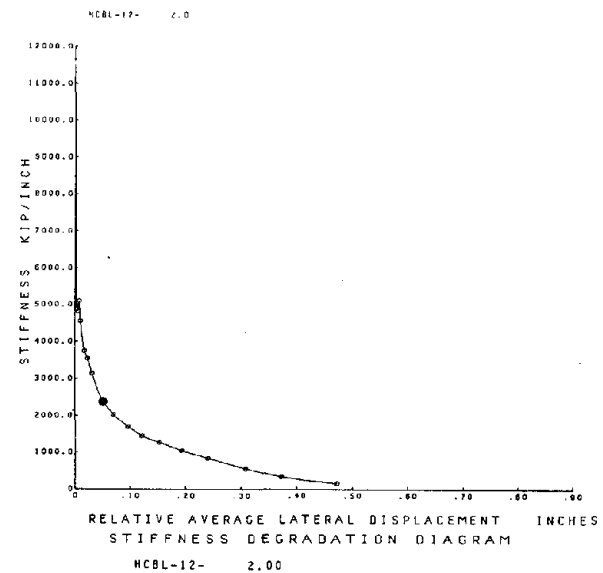
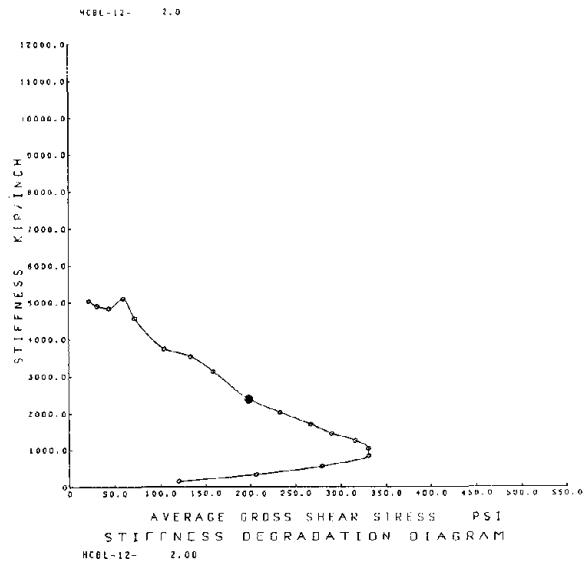
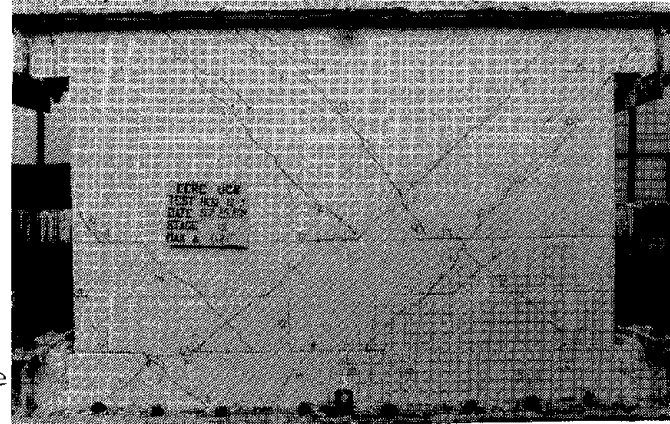


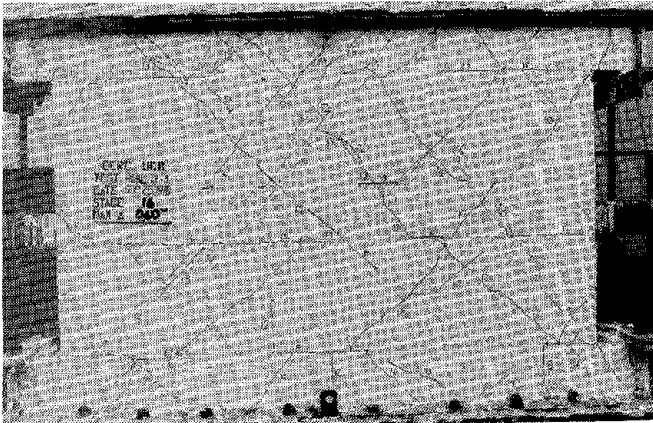
FIG. A.2 CONTINUED HCBL-12-2



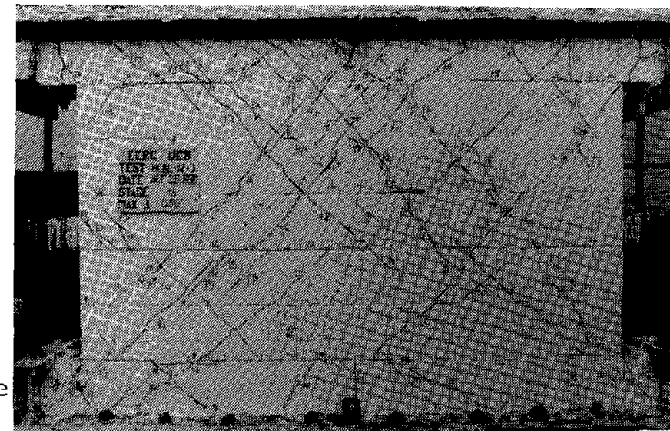
Stage  
10



Stage  
12



Stage  
16

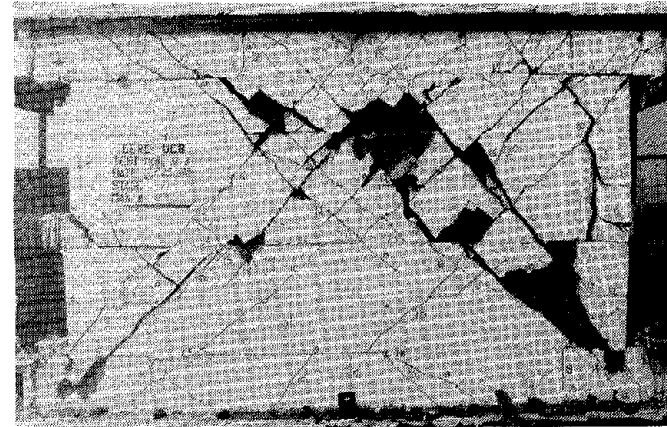


Stage  
18

FIG. A.3 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBL-12-3



Stage Stage  
20 21



HCBL-12- 3.0

HCBL-12- 3.0

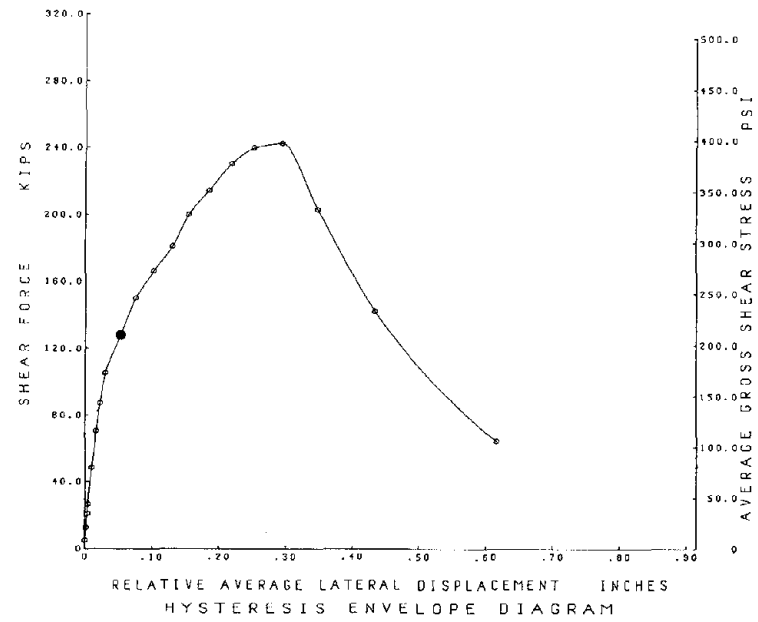
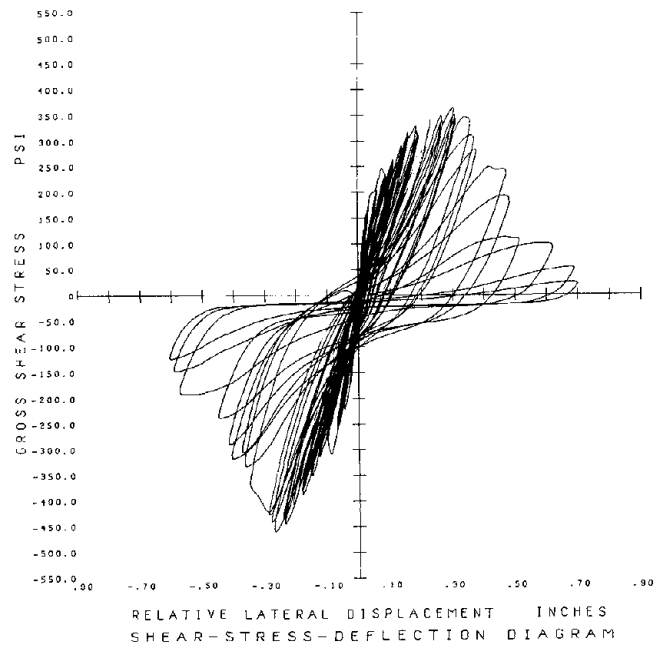


FIG. A.3 CONTINUED HCBL-12-3

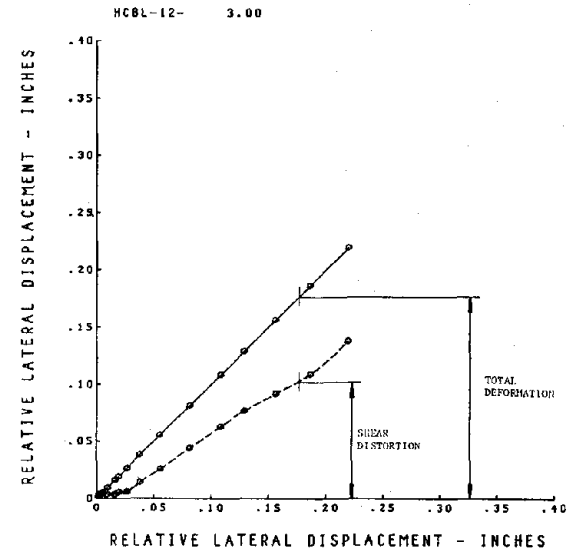
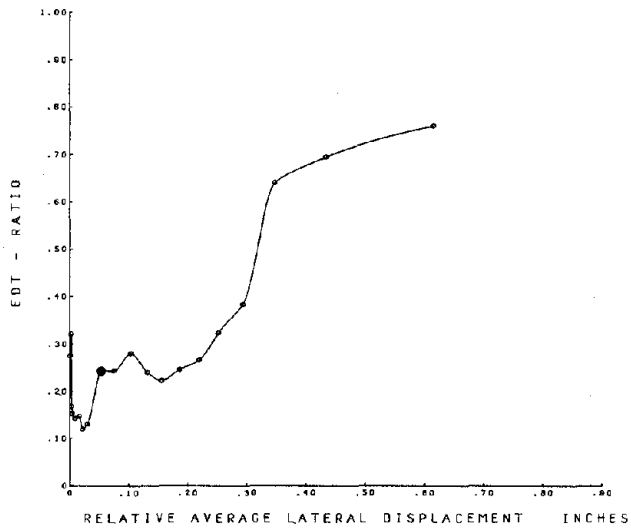
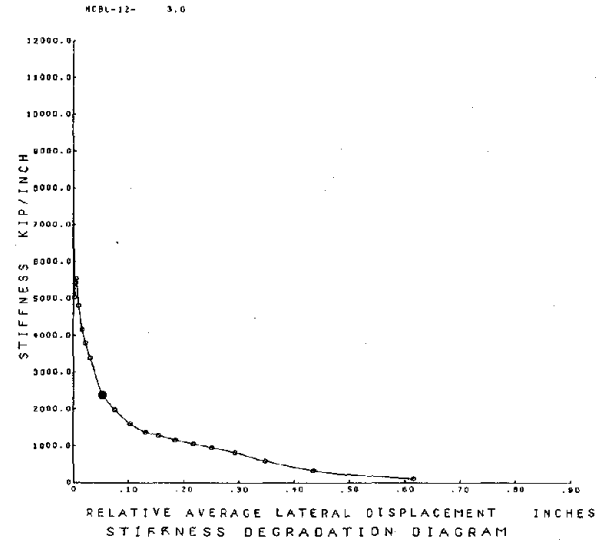
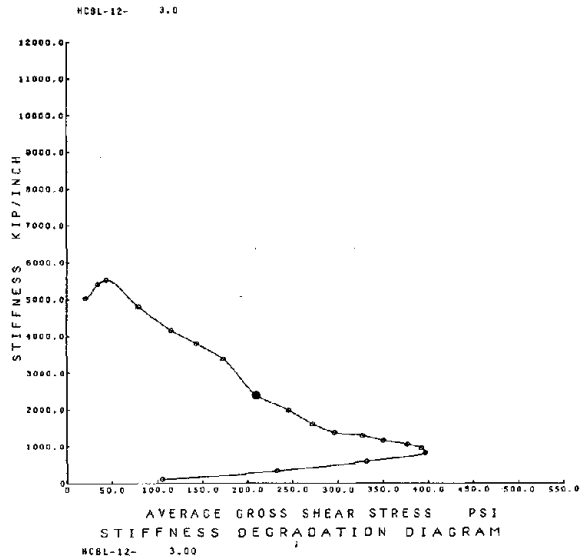
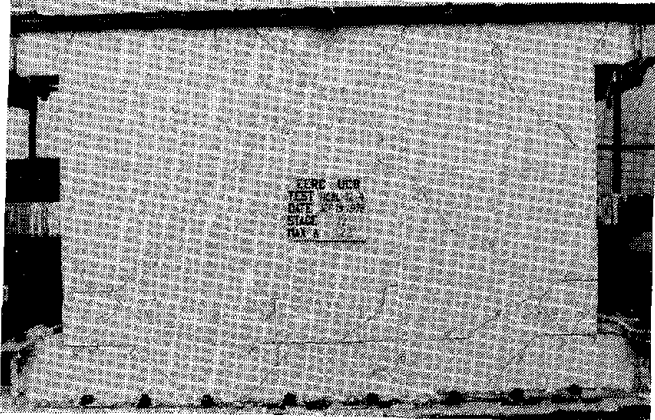
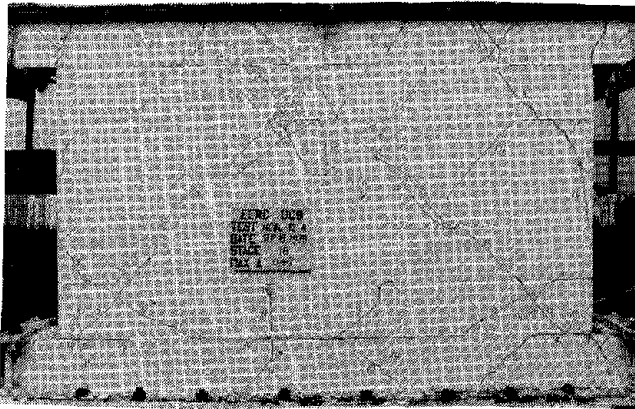
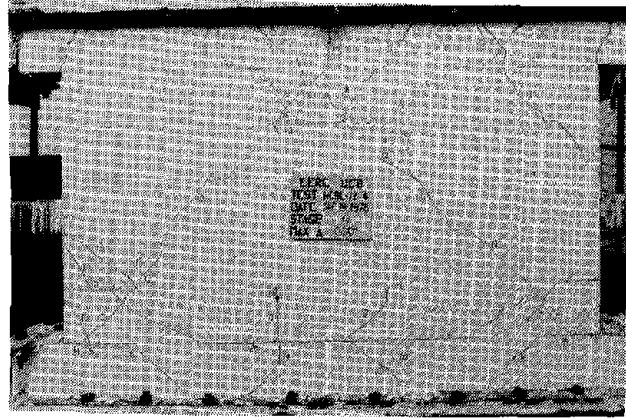


FIG. A.3 CONTINUED HCBL-12-3



●  
Stage  
12

Stage  
14



Stage  
16

Stage  
17

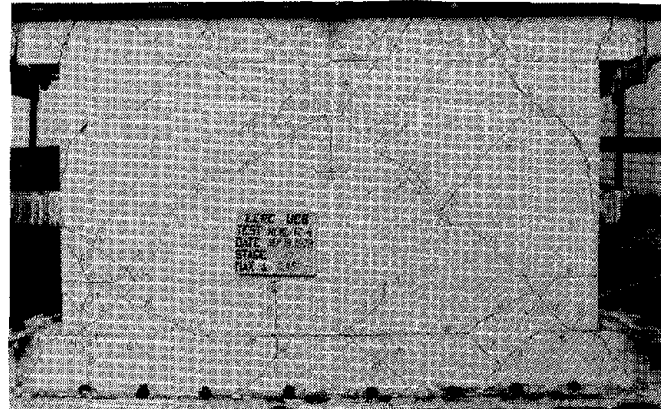
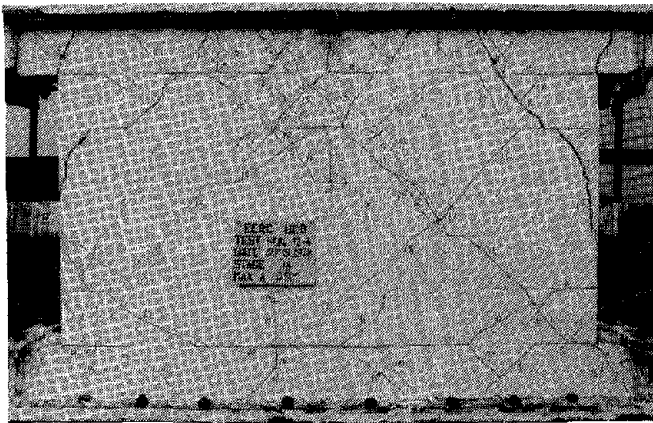


FIG. A.4 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBL-12-4



Stage Stage  
18 20

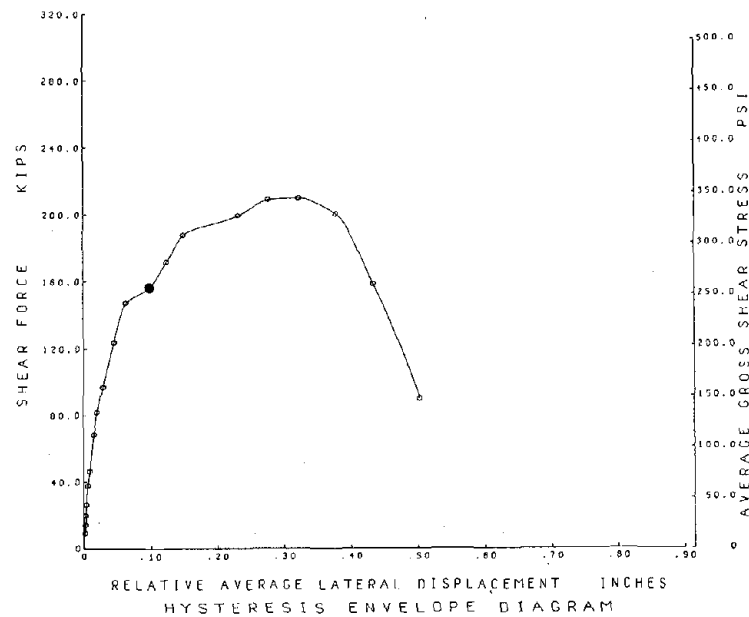
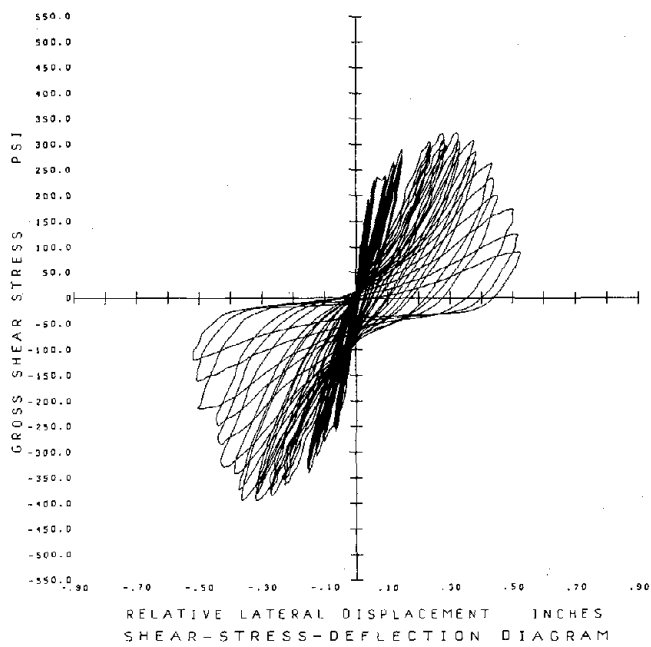
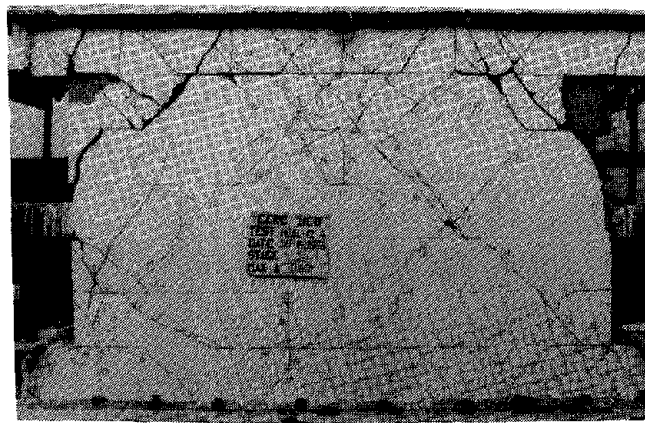


FIG. A.4 CONTINUED HCBL-12-4

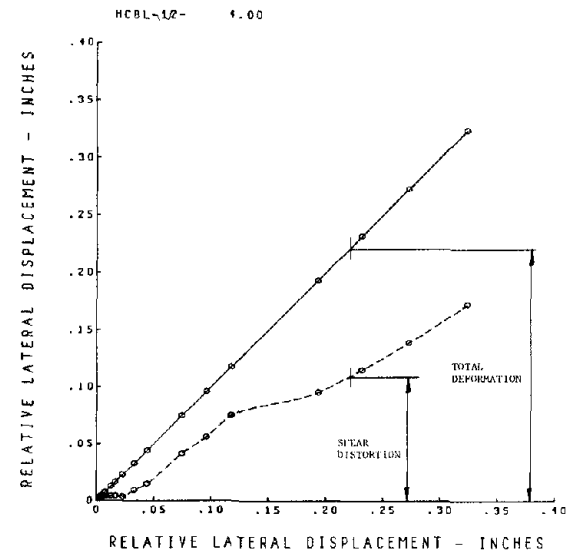
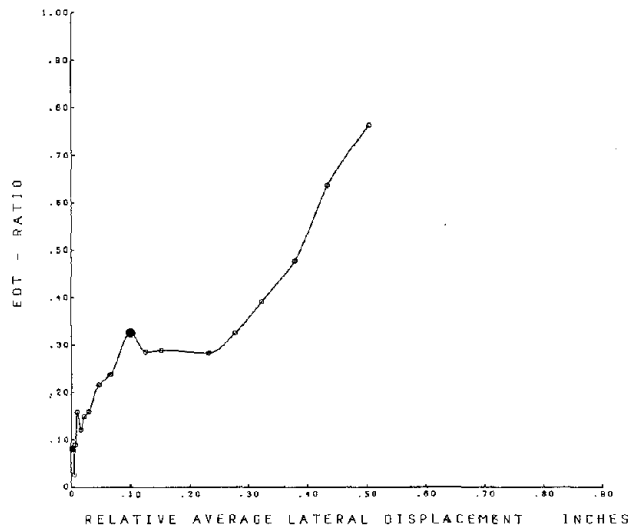
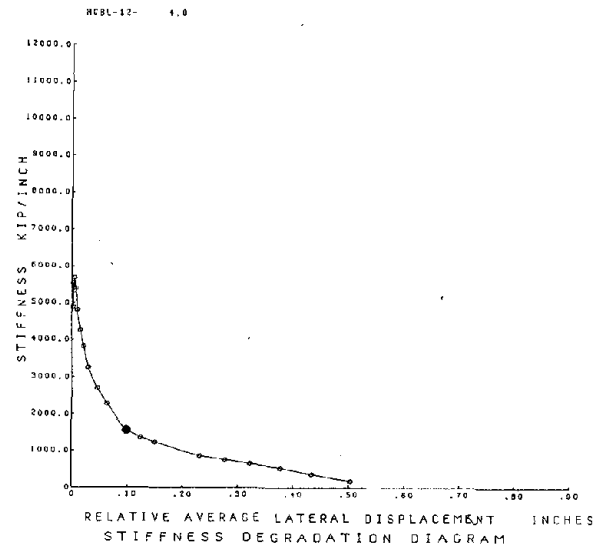
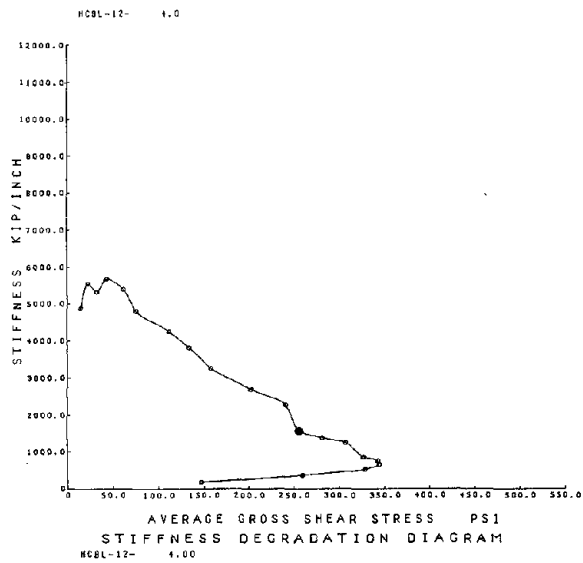
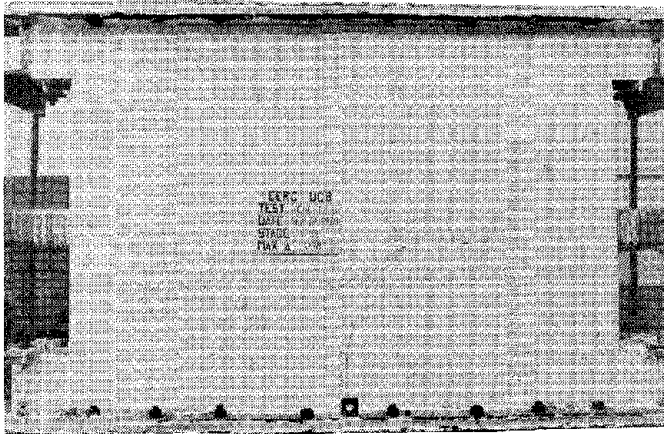
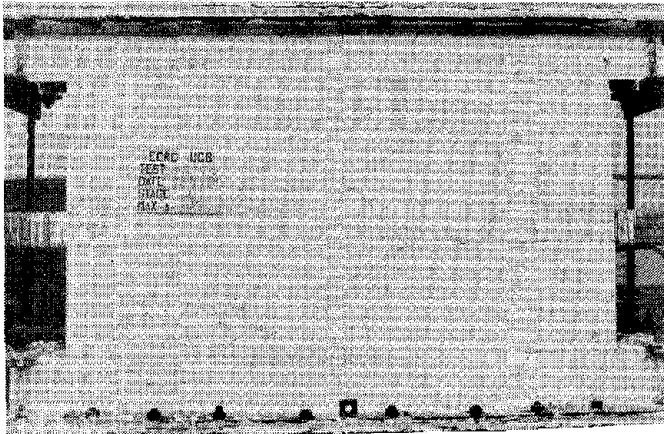
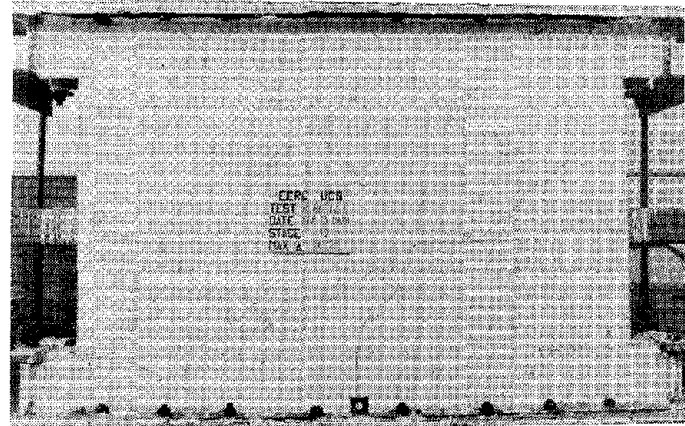


FIG. A.4 CONTINUED HCBL-12-4



●  
Stage 11    Stage 12



Stage 15    Stage 18

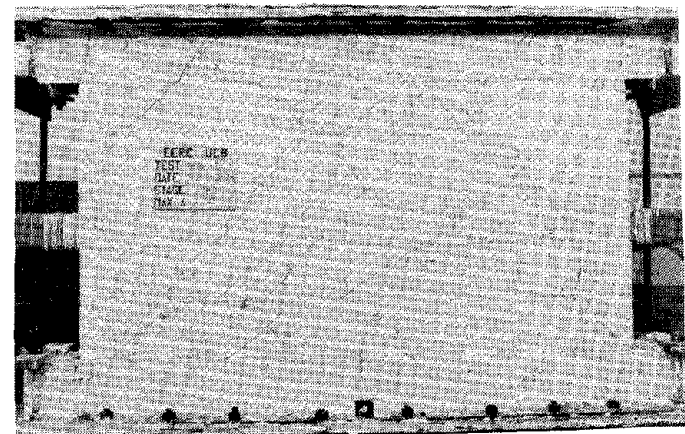
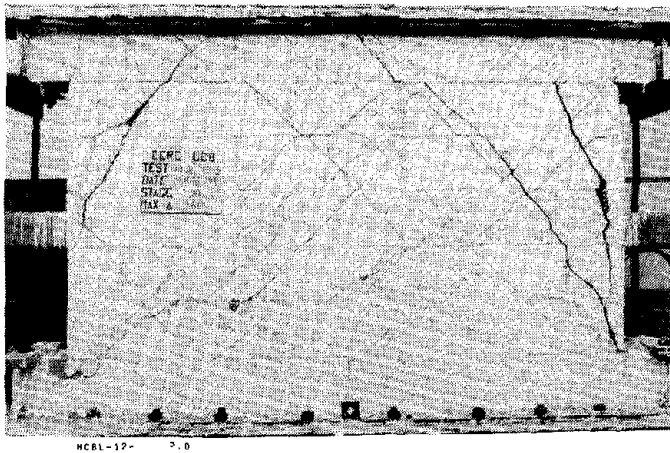


FIG. A.5 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBL-12-5





Stage Stage  
20 21

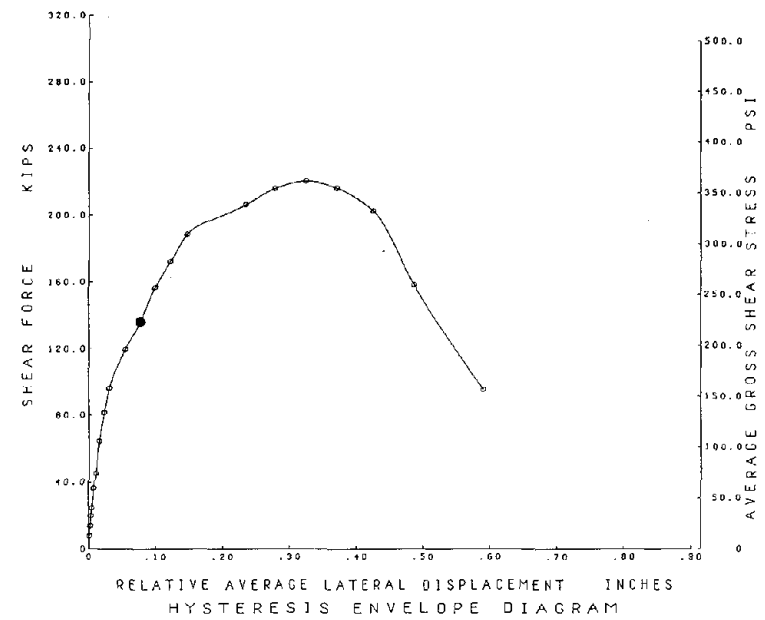
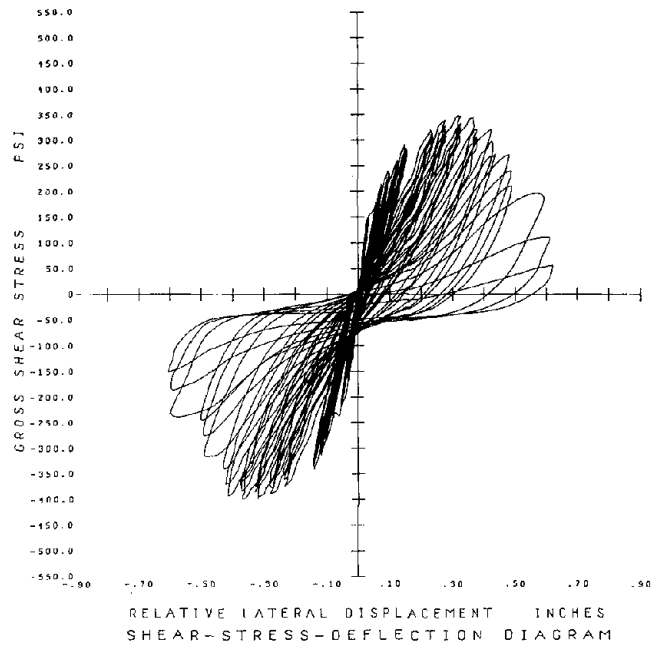


FIG. A.5 CONTINUED HCBL-12-5

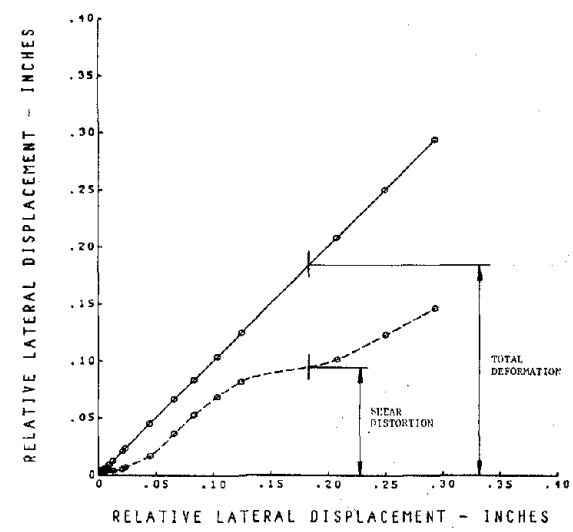
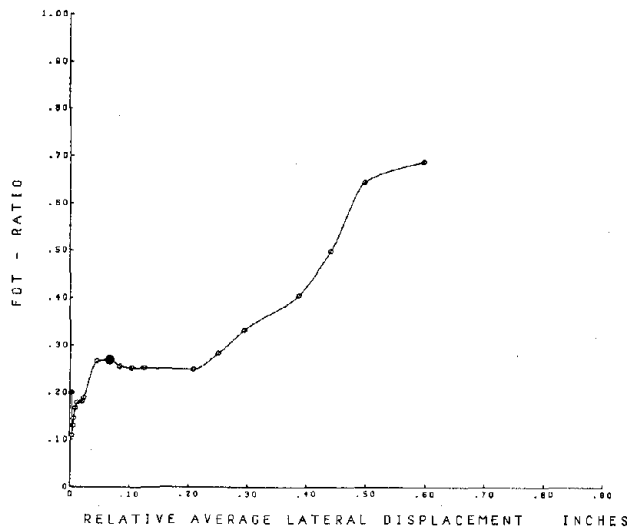
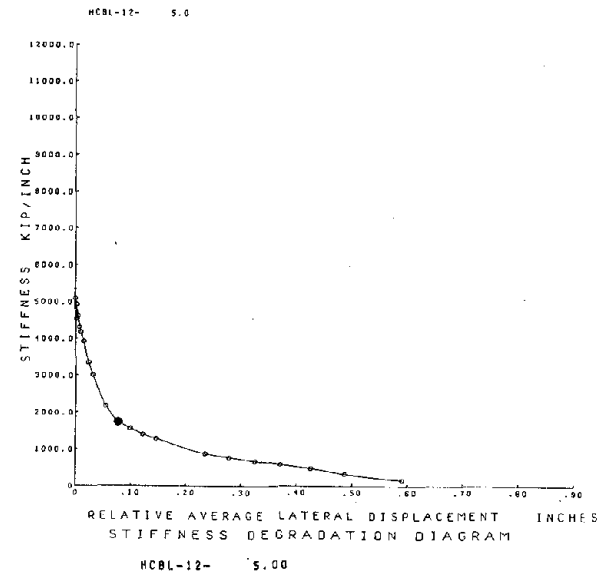
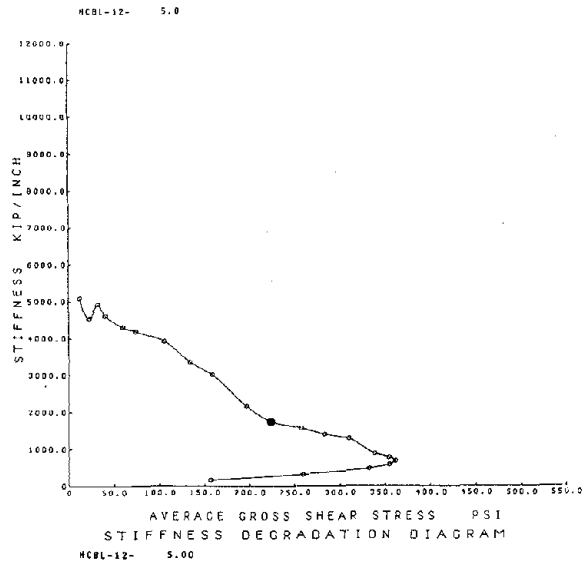
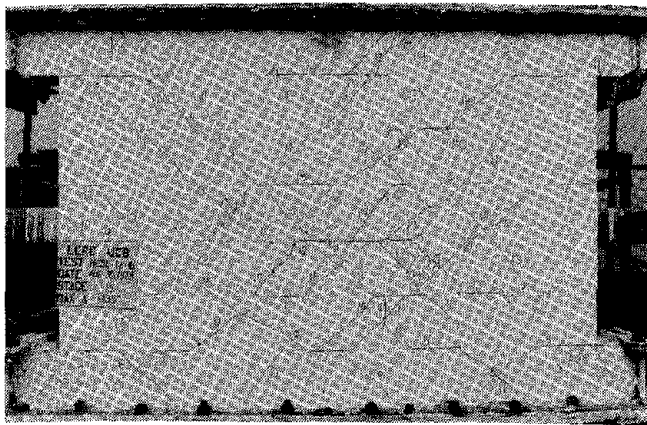
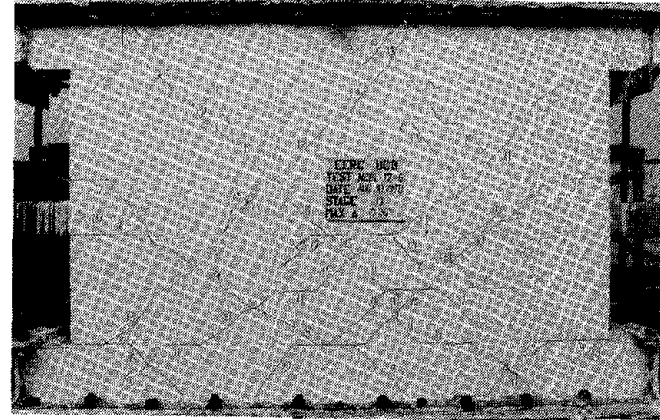


FIG. A.5 CONTINUED HCBL-12-5



●  
Stage Stage  
11 13



Stage Stage  
15 17

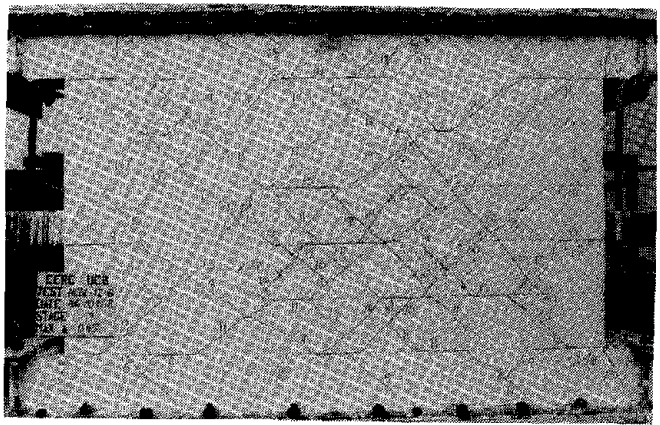
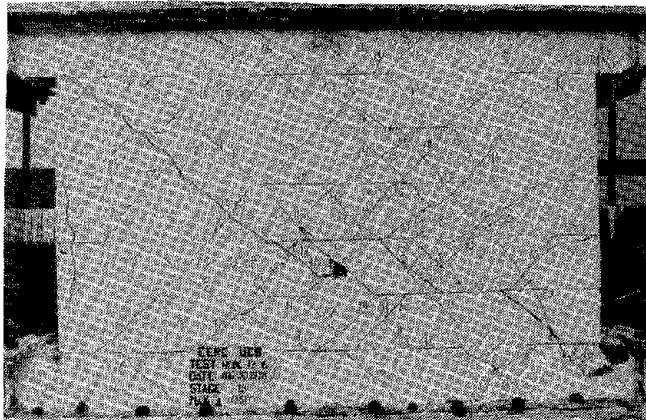


FIG. A.6 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBL-12-6



Stage 18    Stage 19

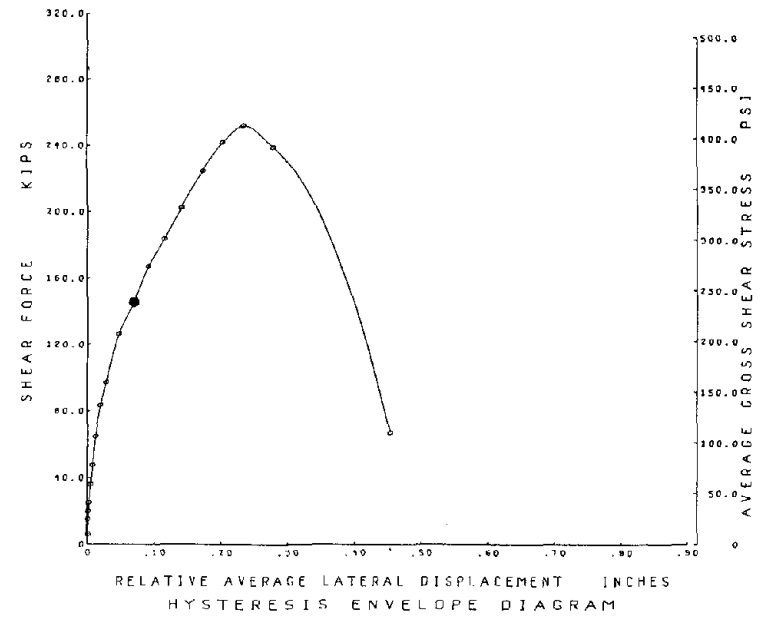
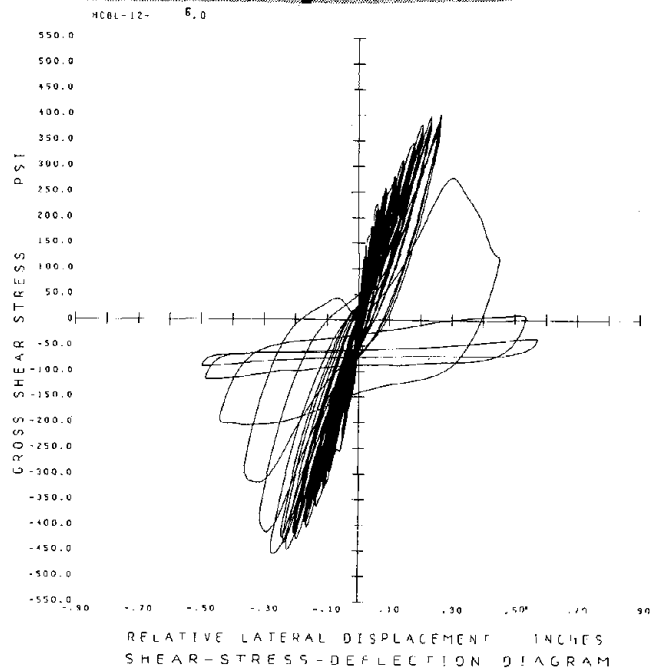
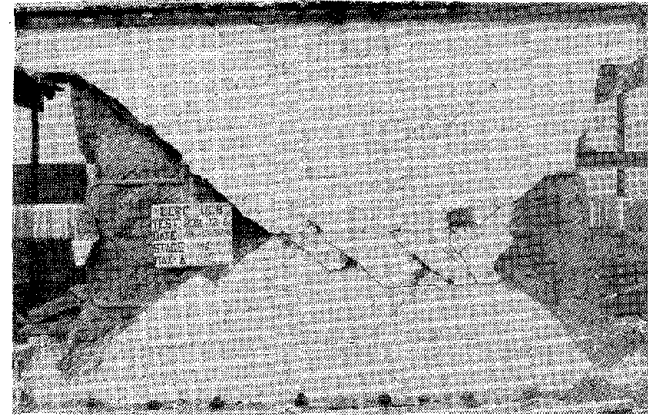


FIG. A.6 CONTINUED HCBL-12-6

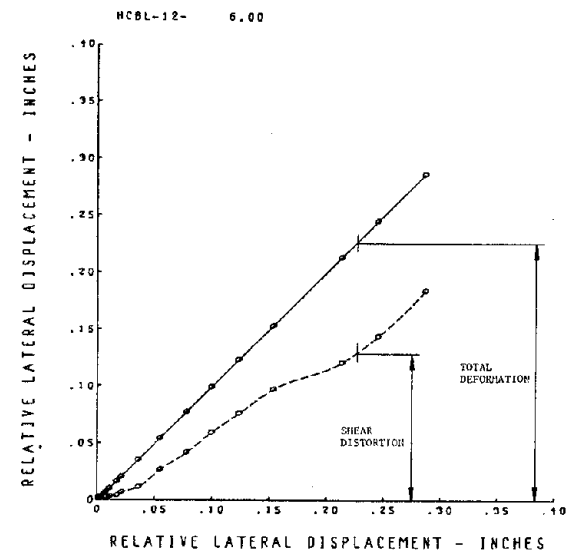
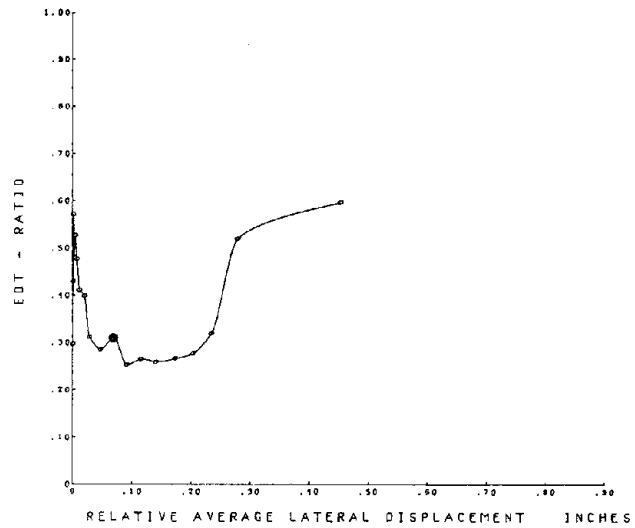
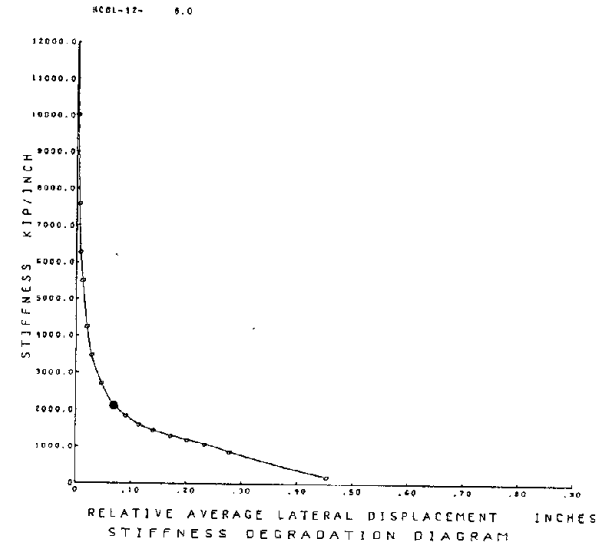
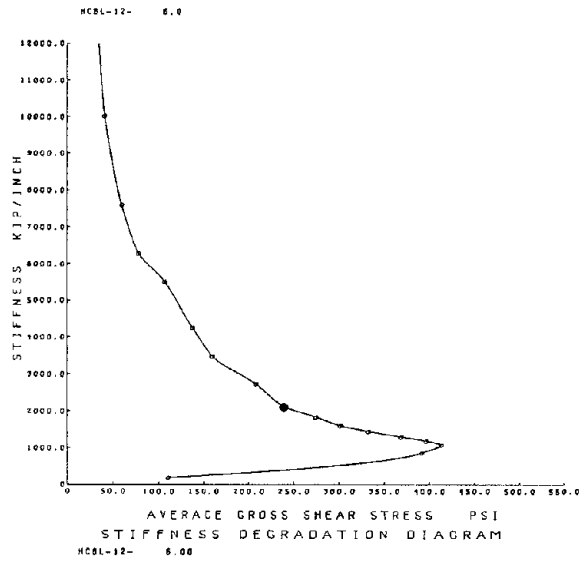
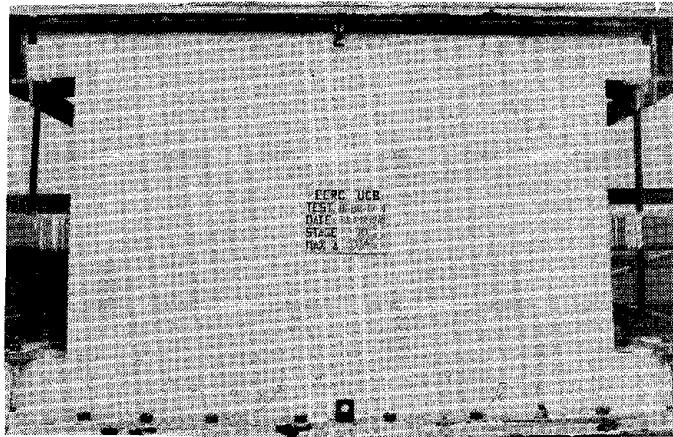
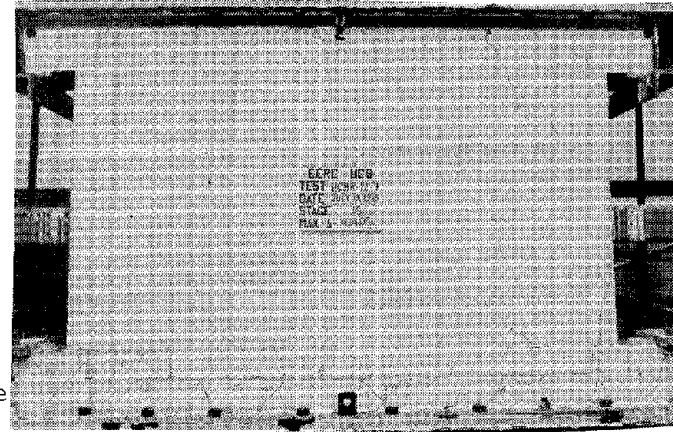


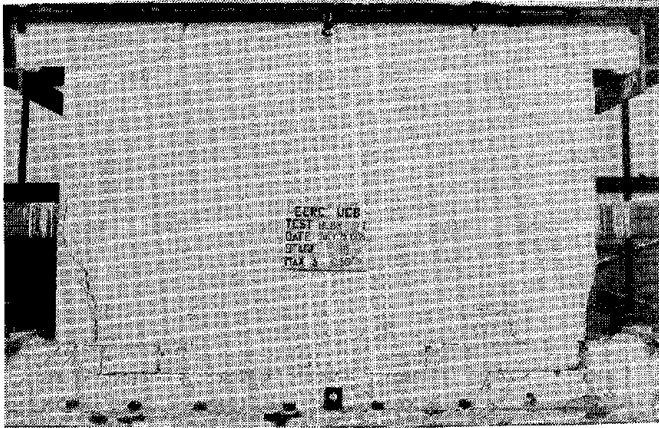
FIG. A.6 CONTINUED HCBL-12-6



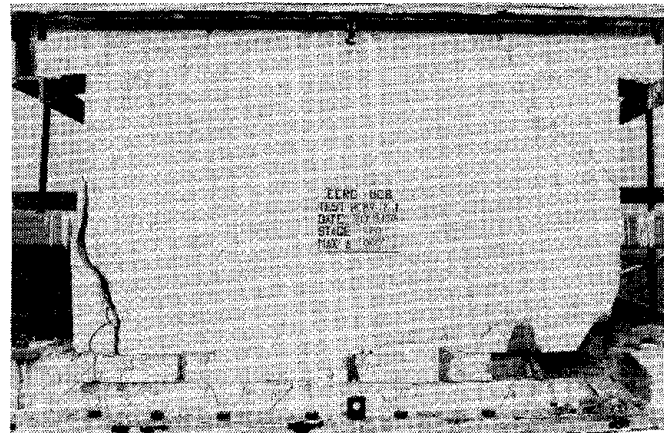
Stage 10



Stage 16

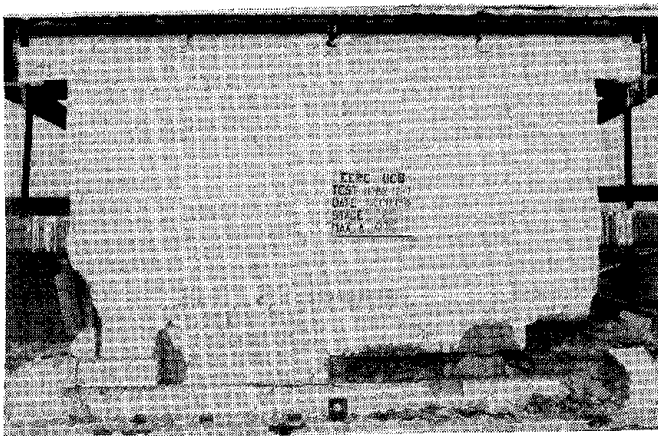


Stage 18



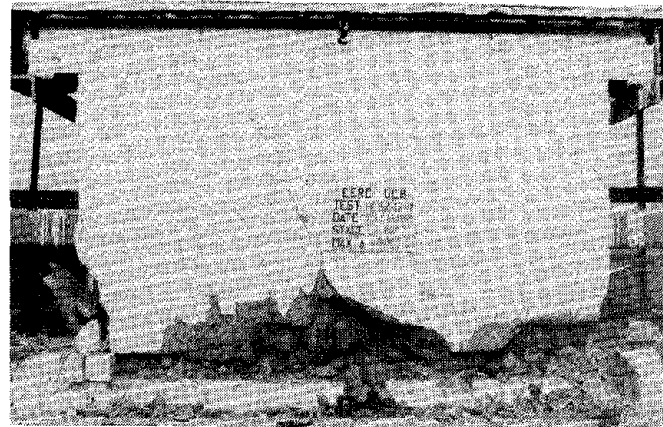
Stage 20

FIG. A.7 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBR-12-1



HCBR-12- 1.0

Stage Stage  
21 22



HCBR-12- 1.0

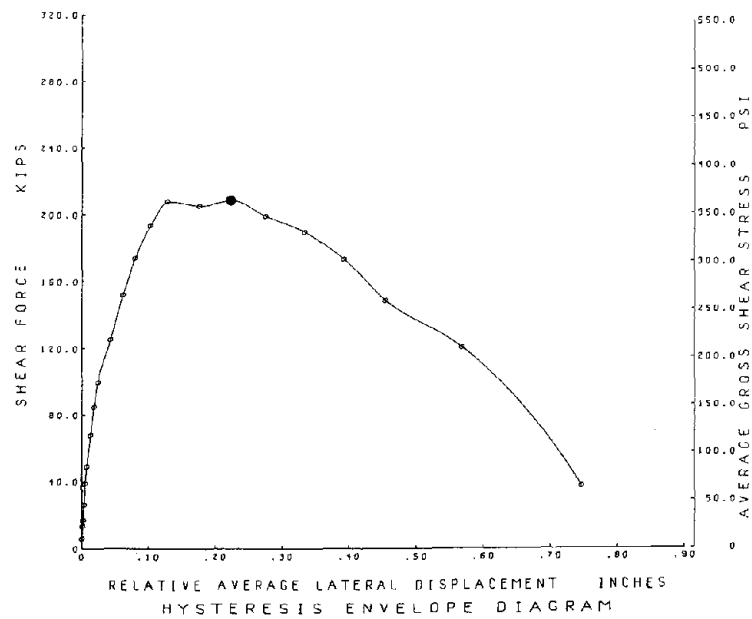
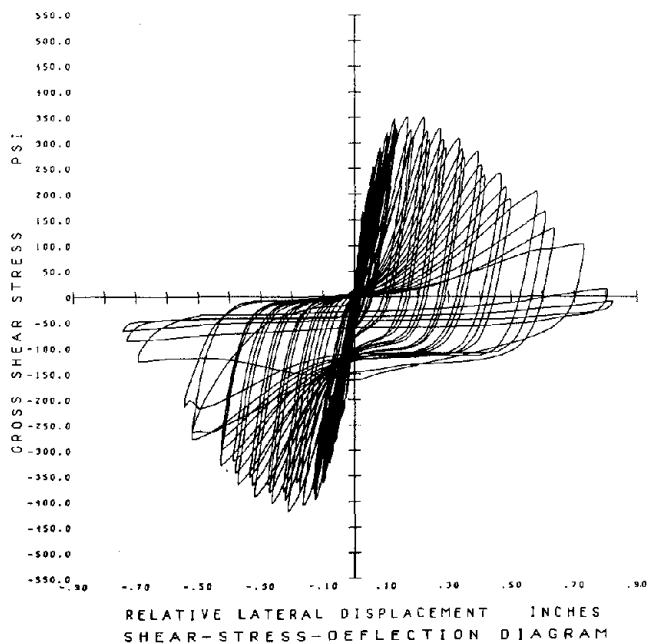


FIG. A.7 CONTINUED HCBR-12-1

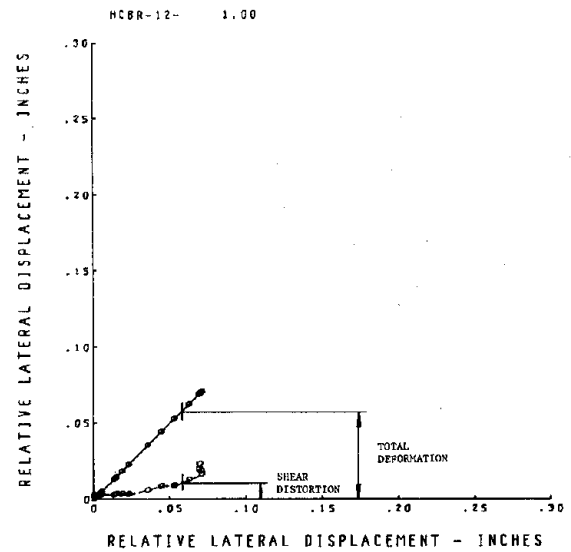
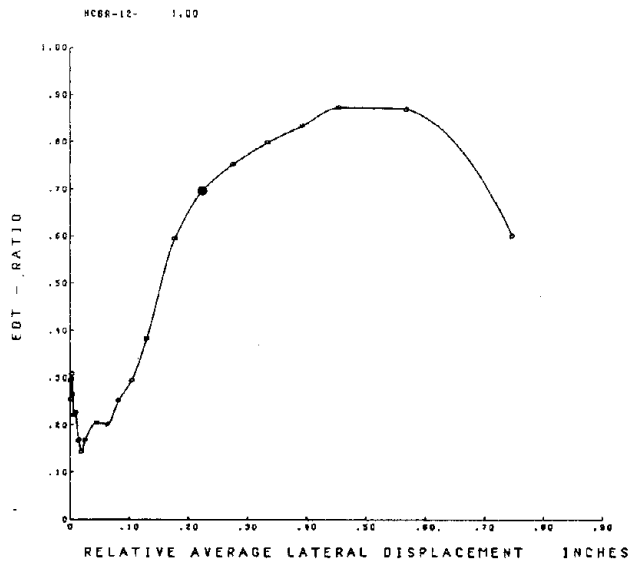
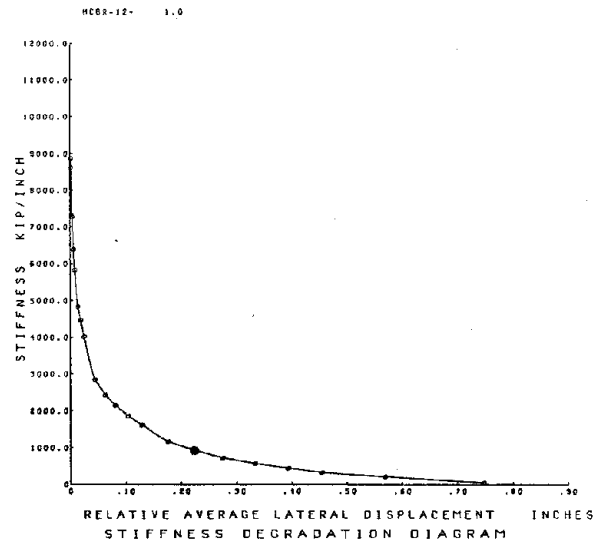
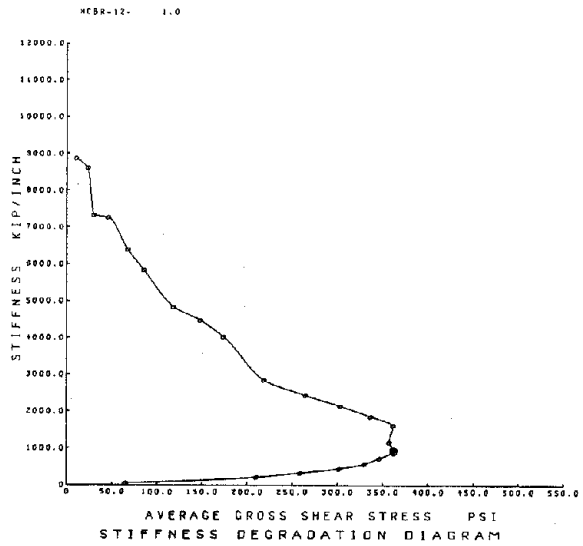
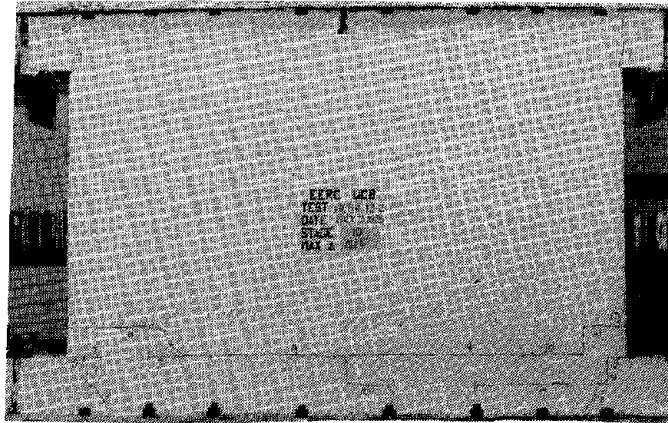
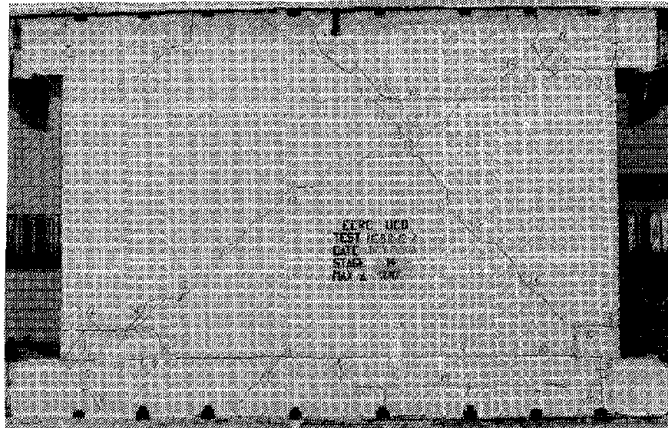
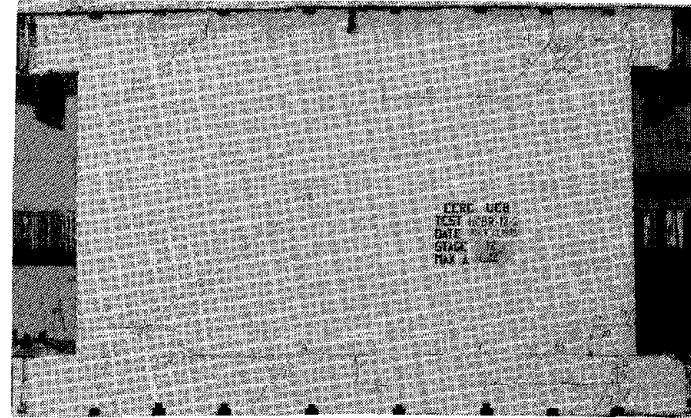


FIG. A.7 CONTINUED HCBR-12-1





Stage Stage  
10 12



Stage Stage  
14 15

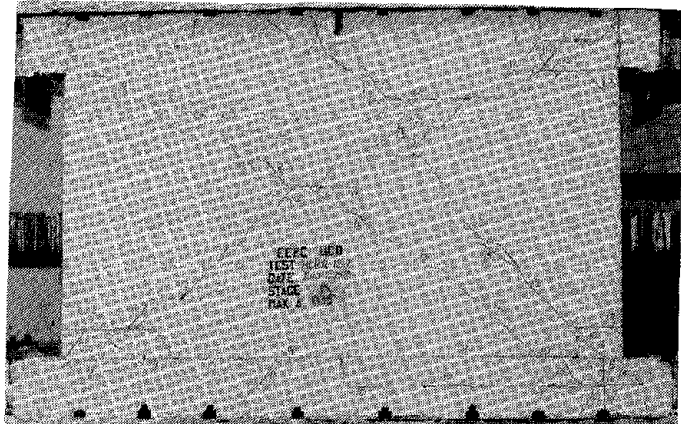


FIG. A.8 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBR-12-2



Stage Stage  
17 19

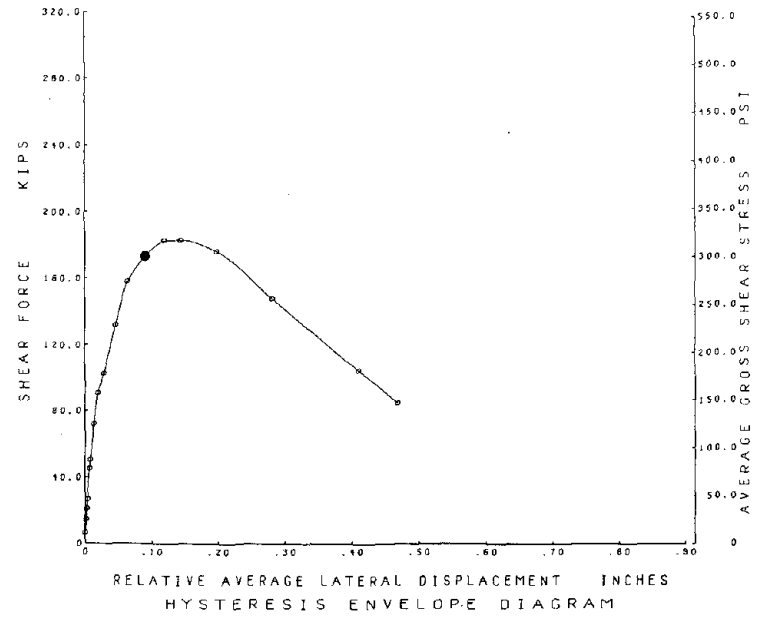
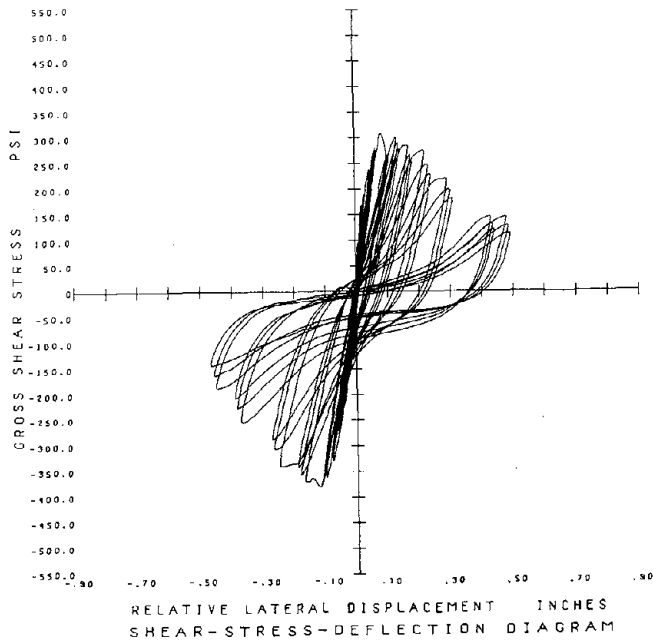


FIG. A.8 CONTINUED HCBR-12-2

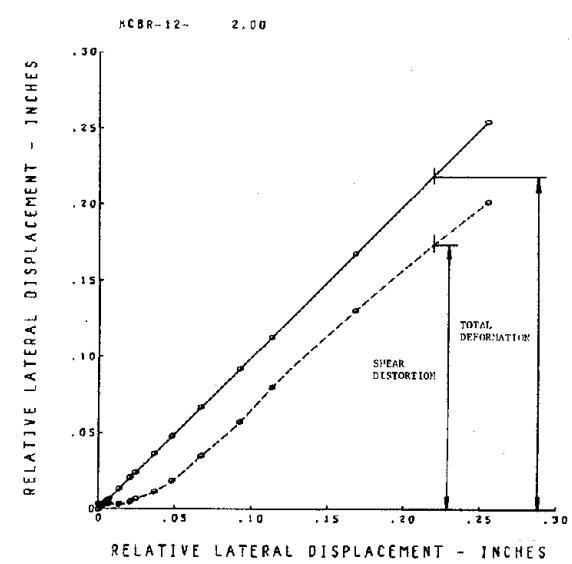
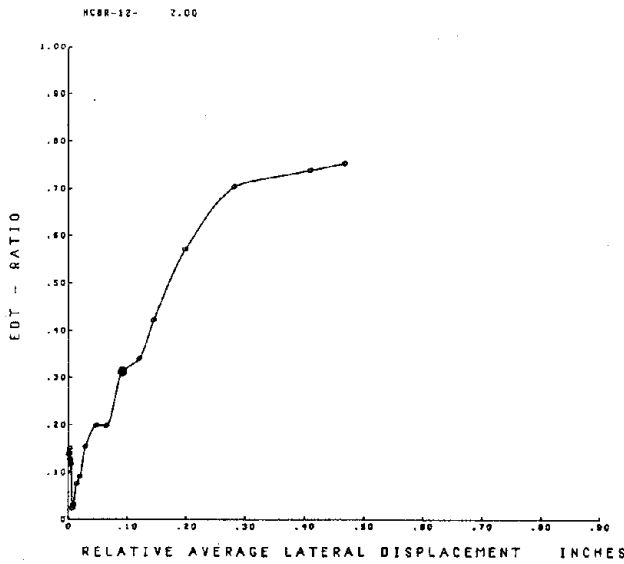
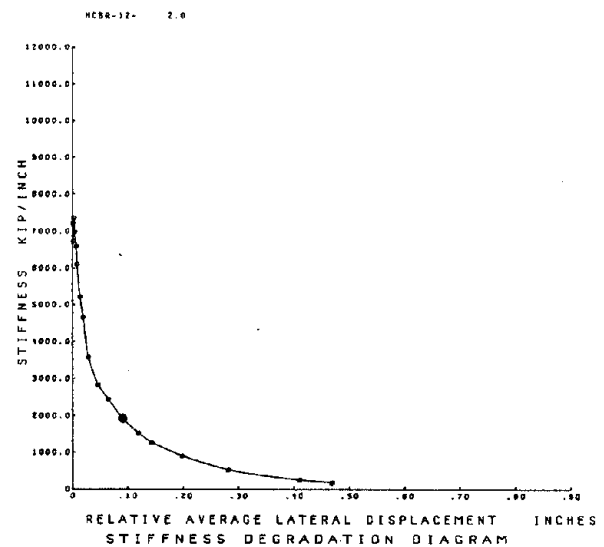
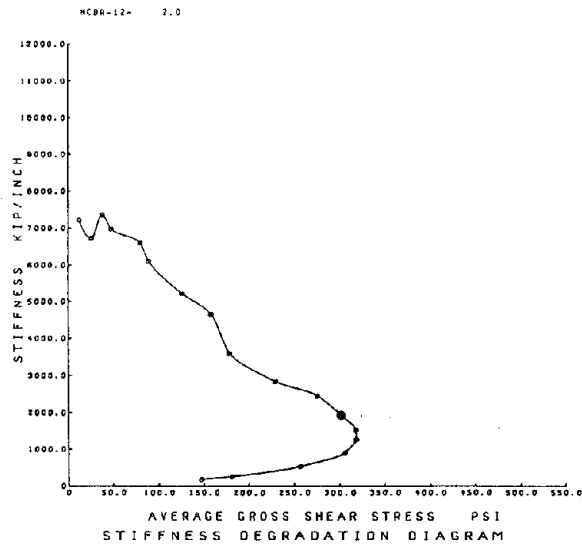
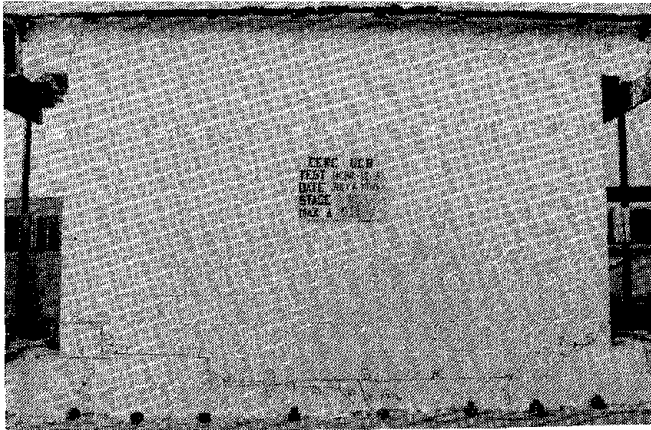
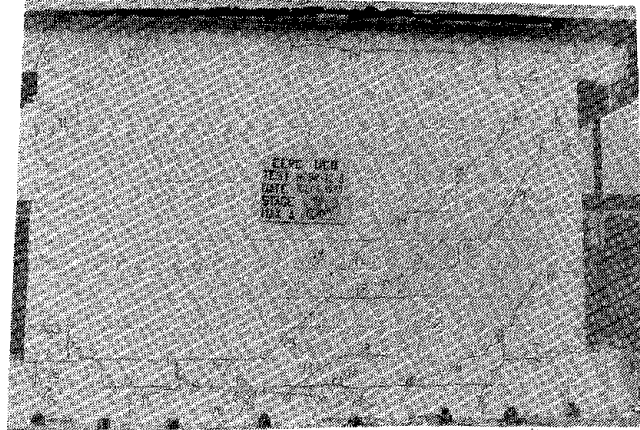


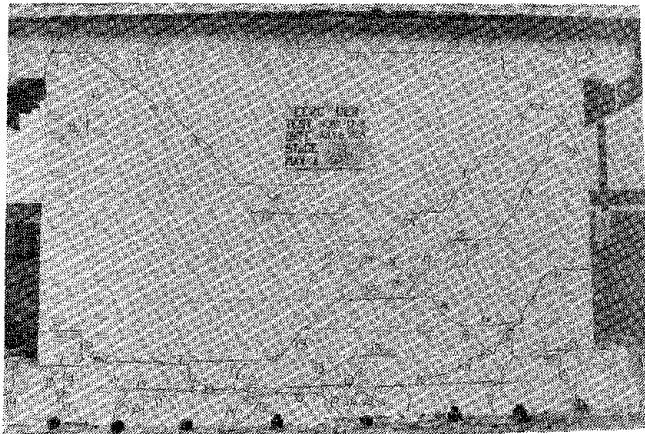
FIG. A.8 CONTINUED HCBR-12-2



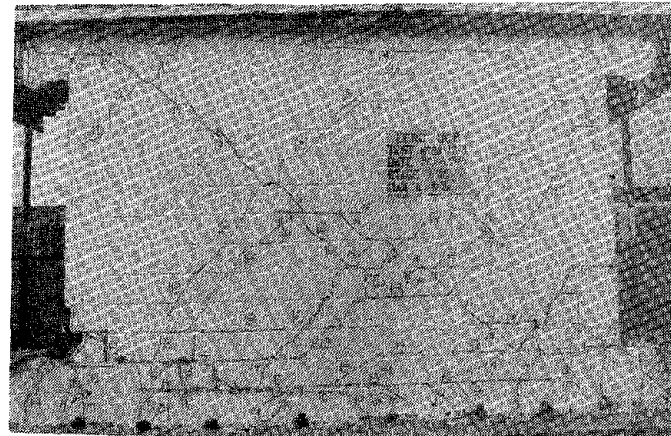
Stage 10



Stage 14

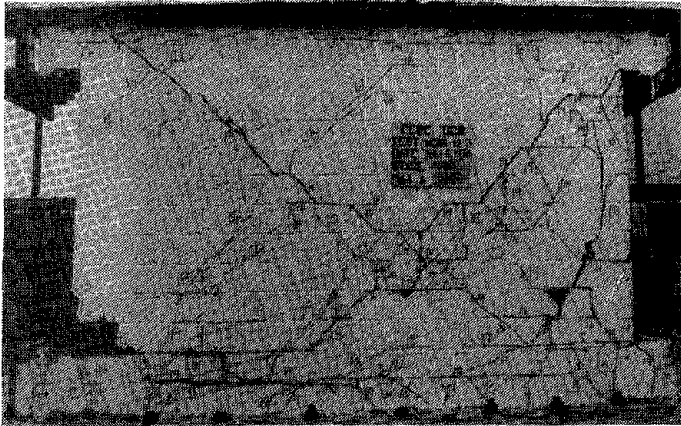


Stage 15



Stage 18

FIG. A.9 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBR-12-3



Stage Stage  
20 21

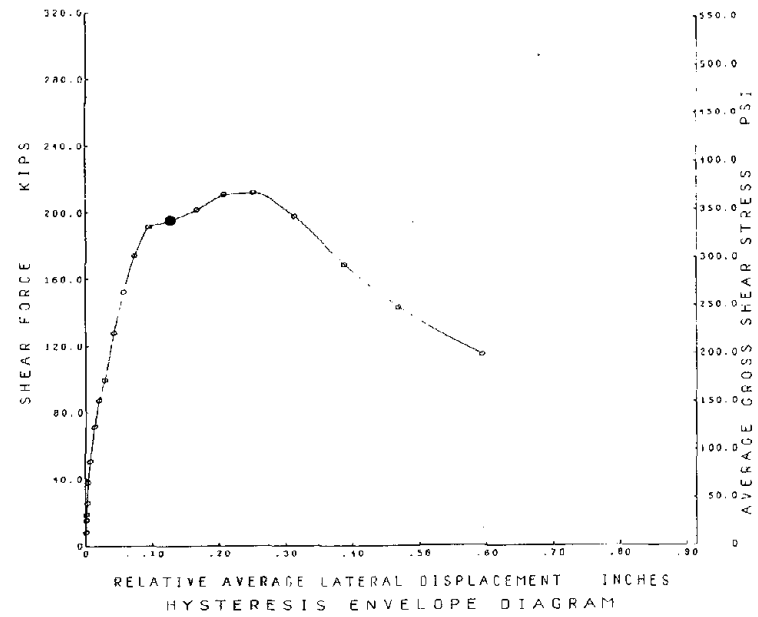
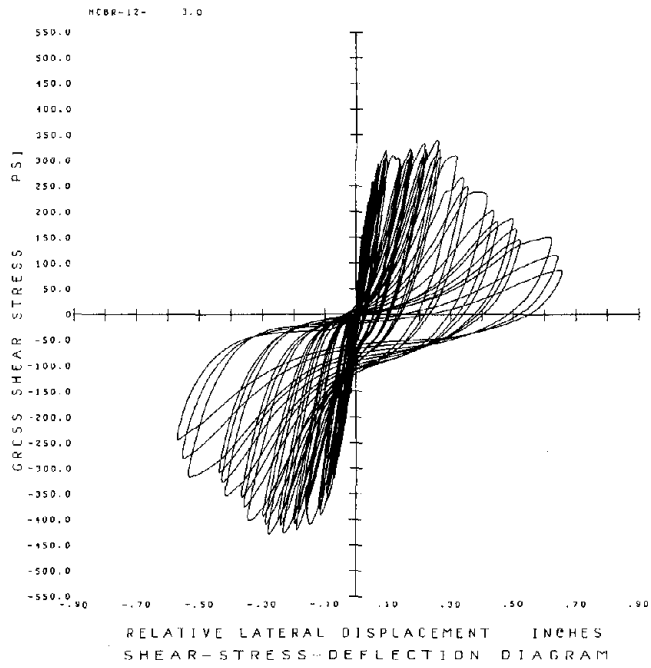


FIG. A.9 CONTINUED HCBR-12-3

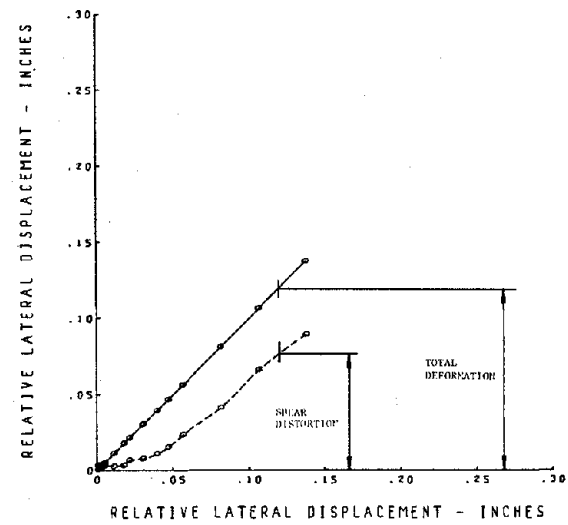
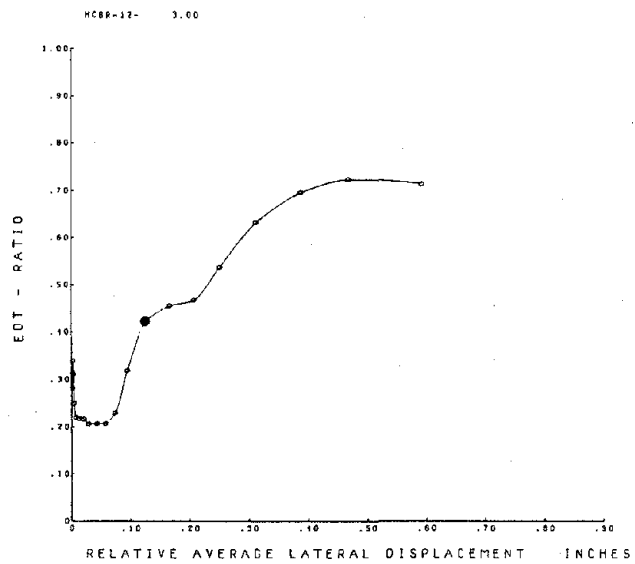
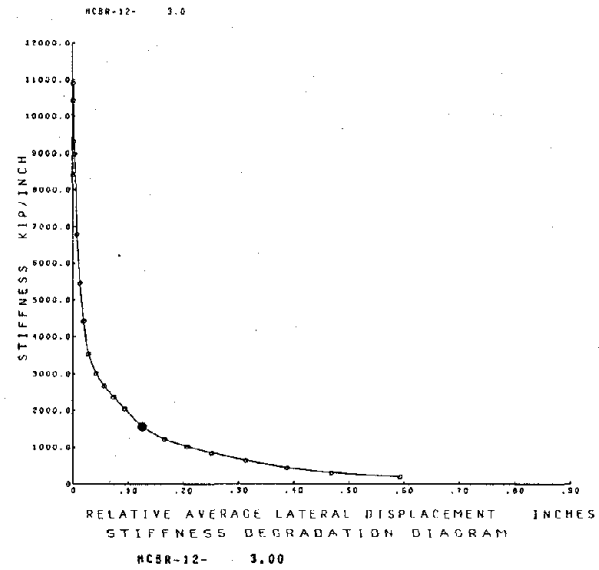
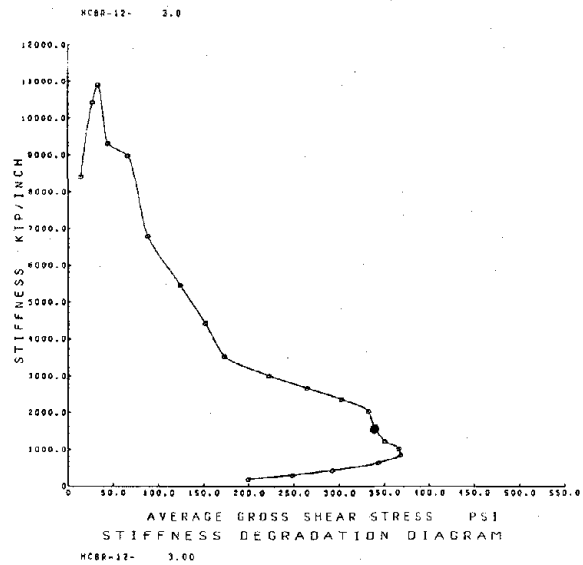
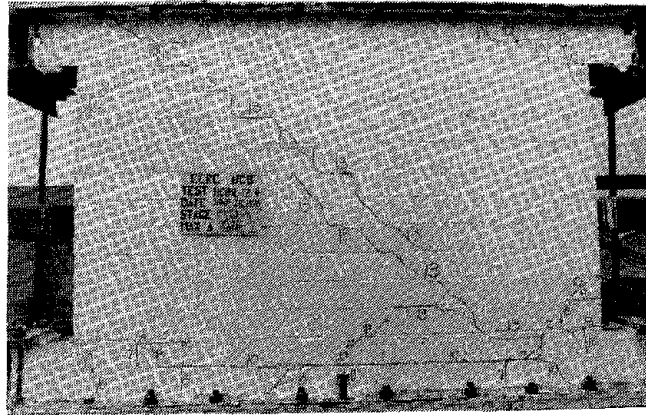
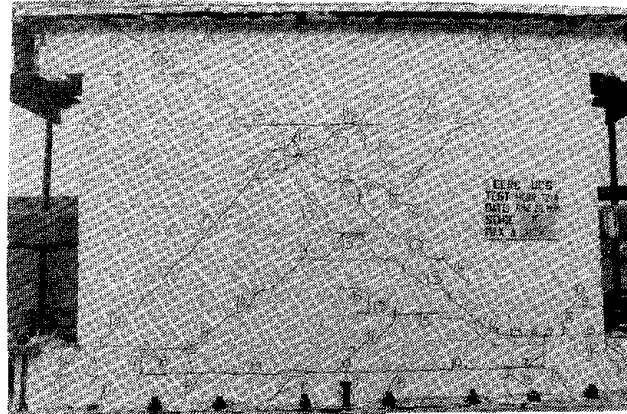
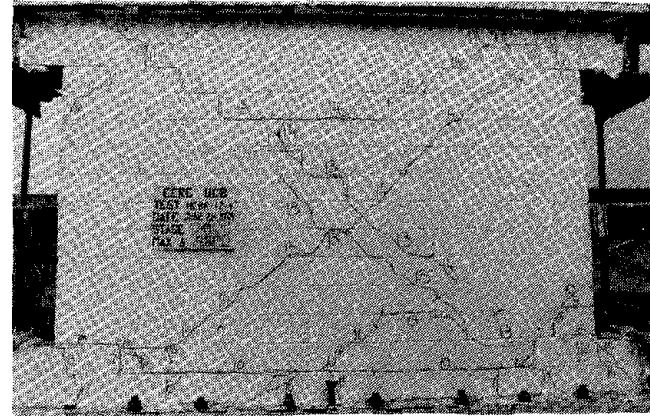


FIG. A.9 CONTINUED HCBR-12-3



●  
Stage Stage  
13 14



Stage Stage  
16 18

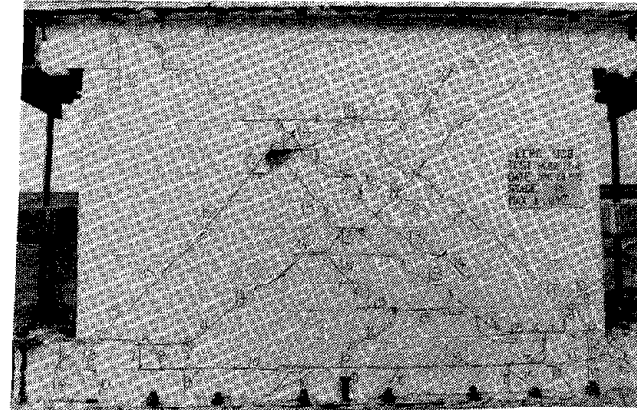
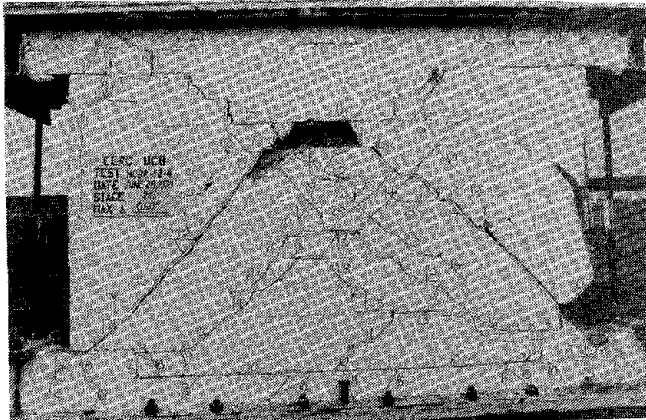


FIG. A.10 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBR-12-4



Stage Stage  
20 21

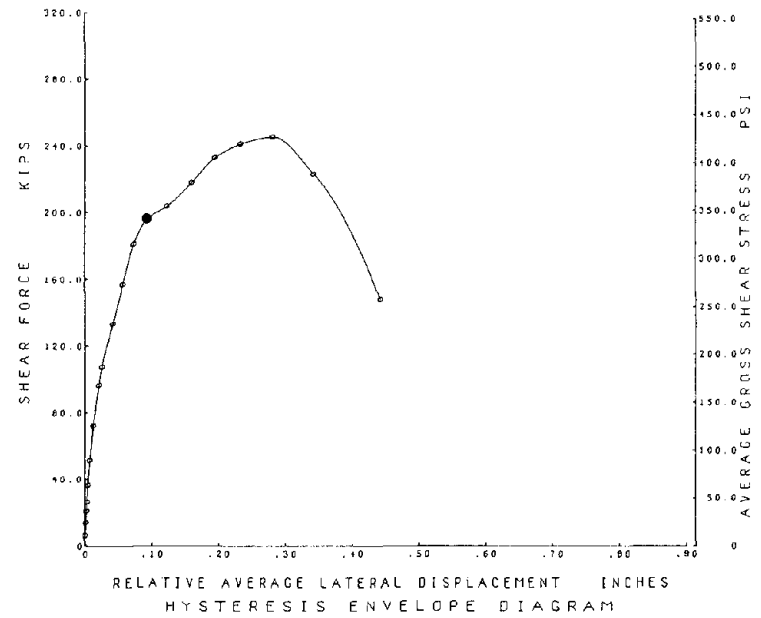
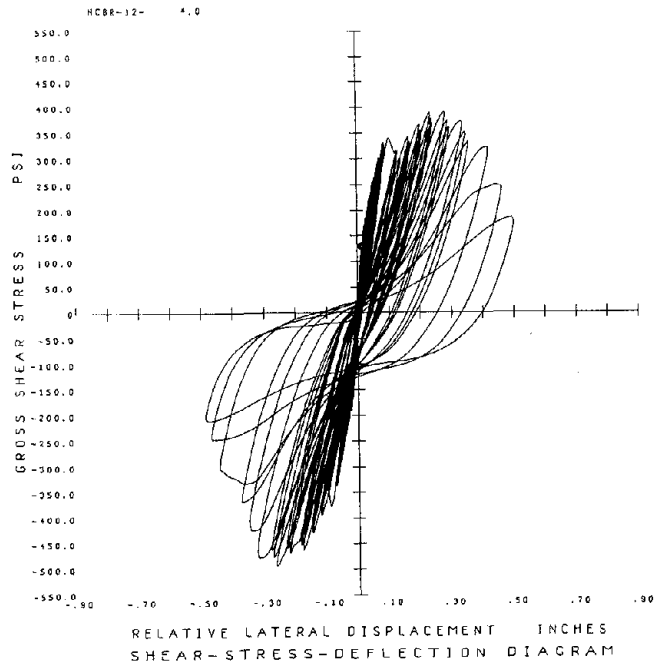
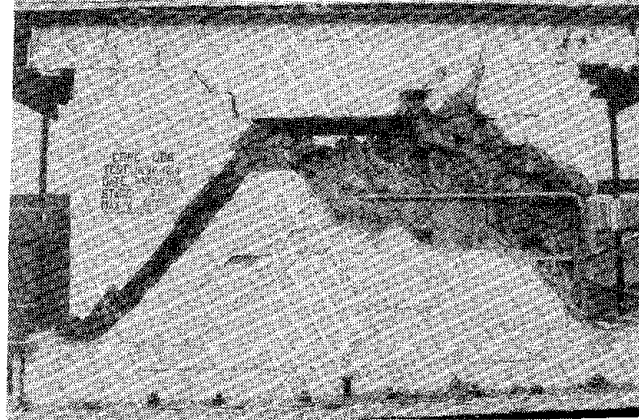


FIG. A.10 CONTINUED HCBR-12-4



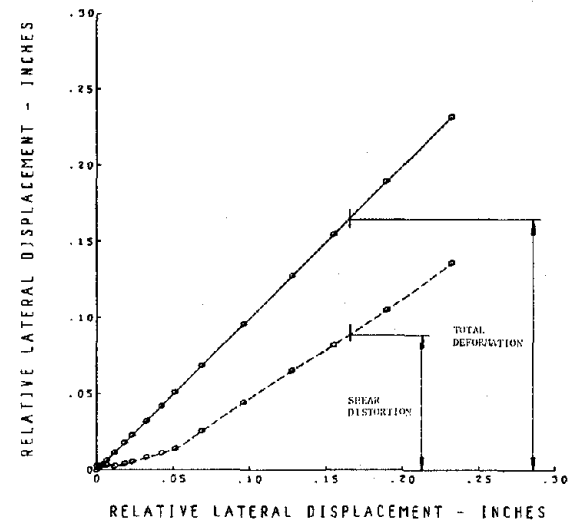
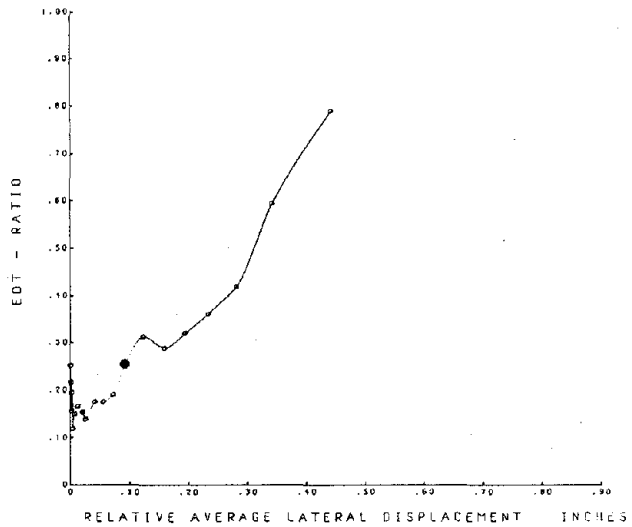
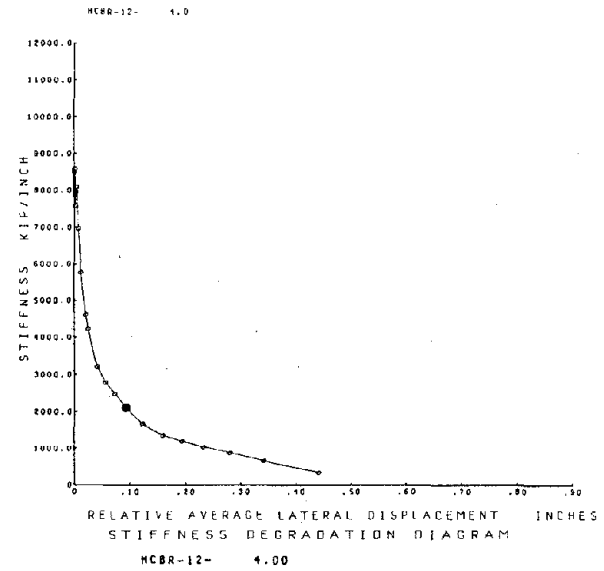
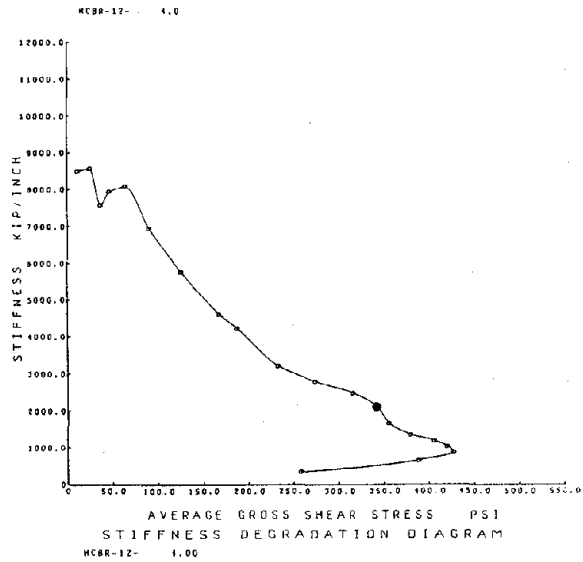
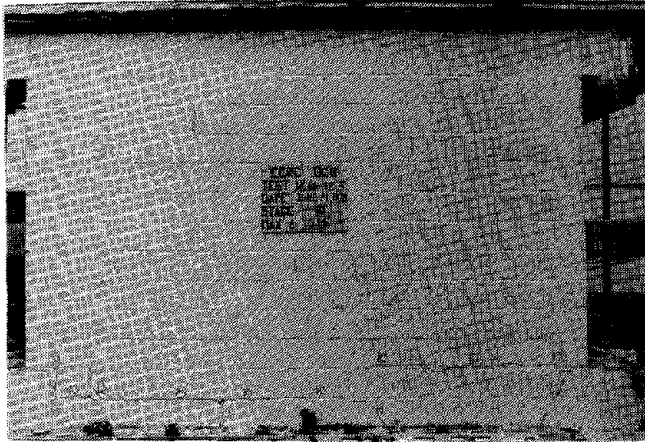
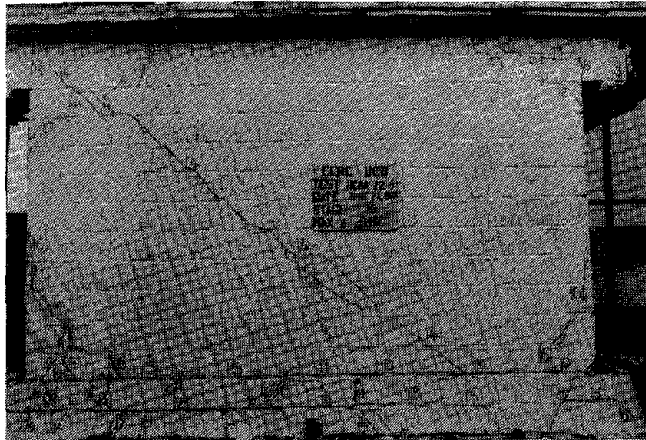
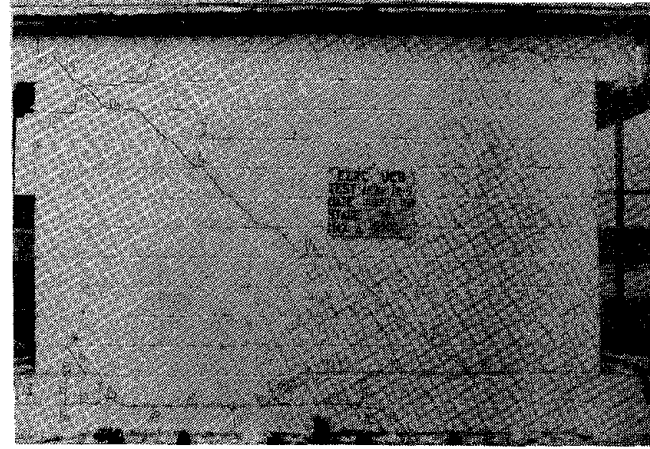


FIG. A.10 CONTINUED HCBR-12-4



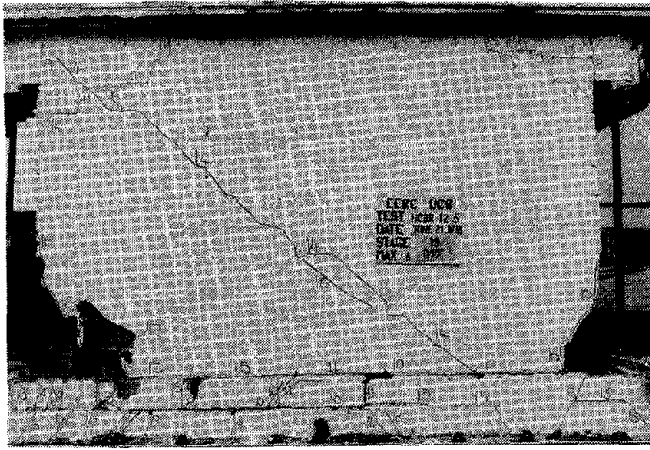
Stage Stage  
10 14



Stage Stage  
16 18



FIG. A.11 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBR-12-5



Stage Stage  
19 20

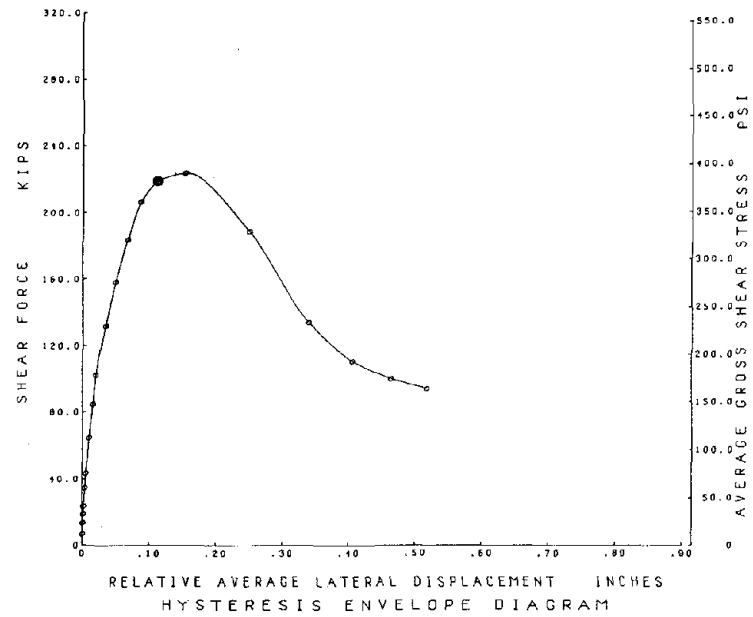
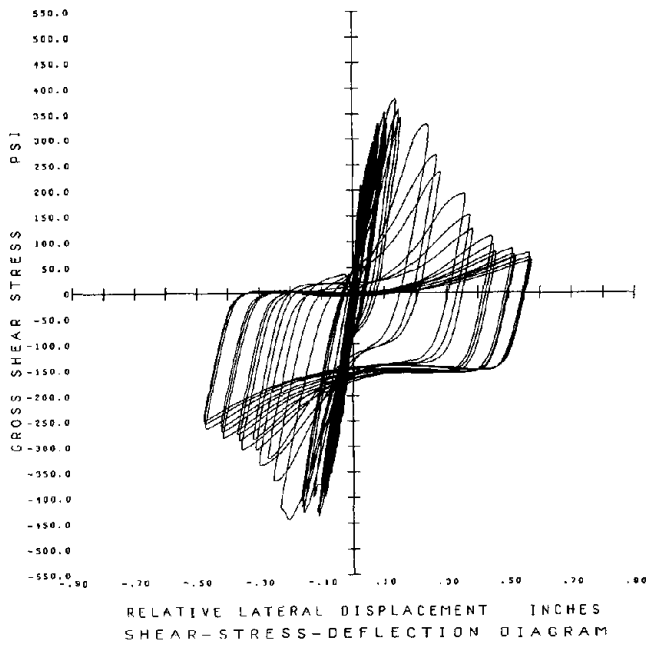


FIG. A.11 CONTINUED HCBR-12-5

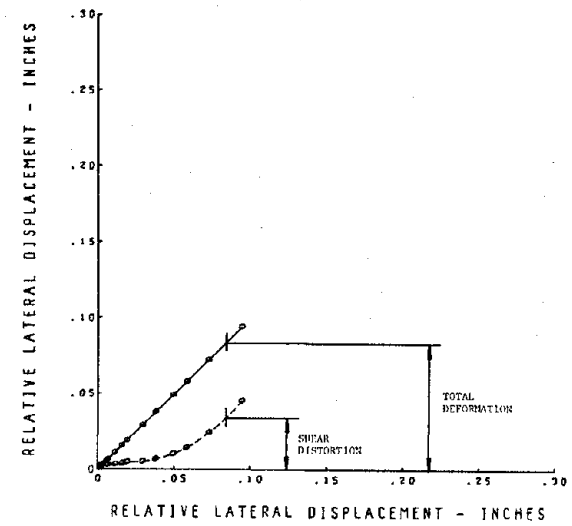
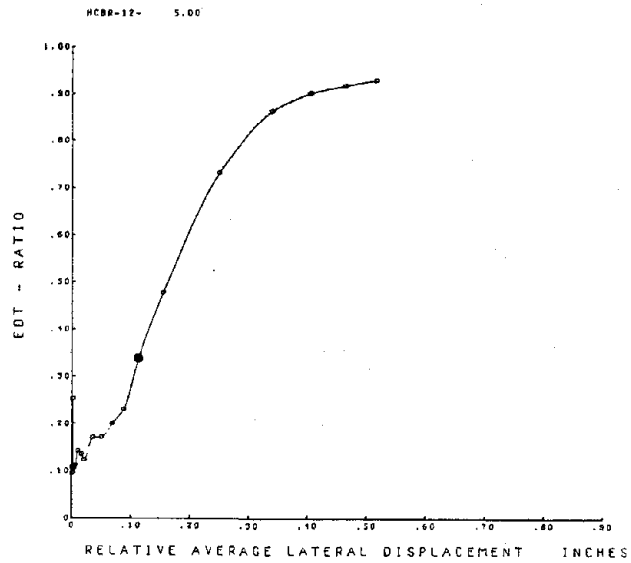
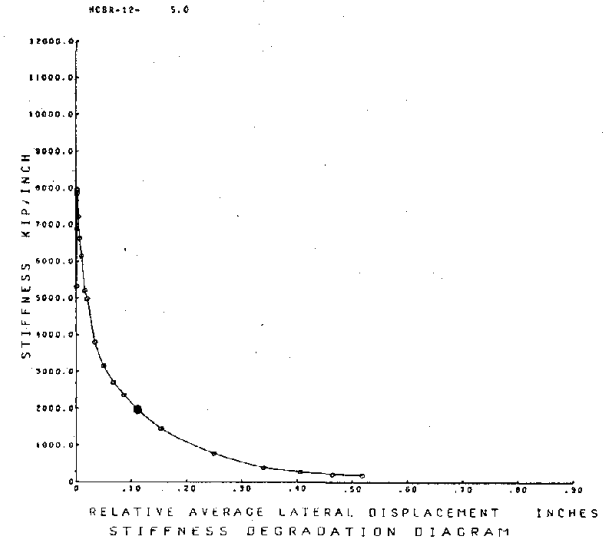
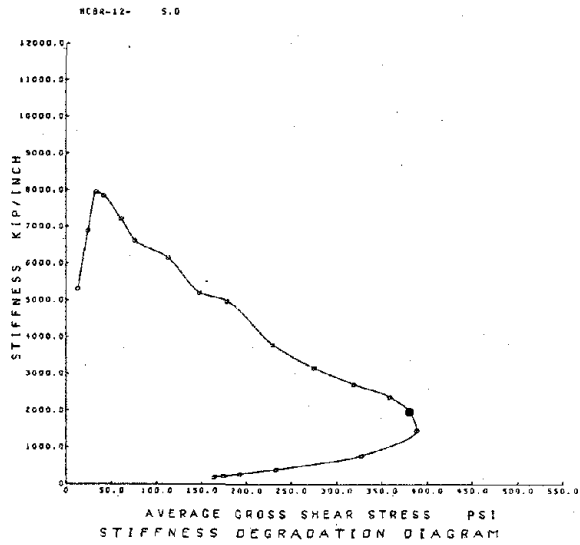
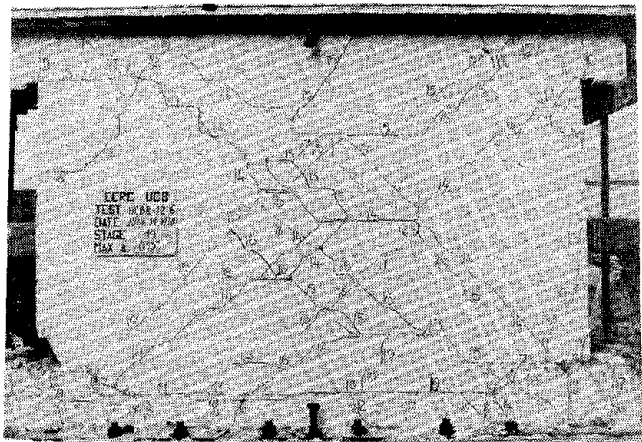
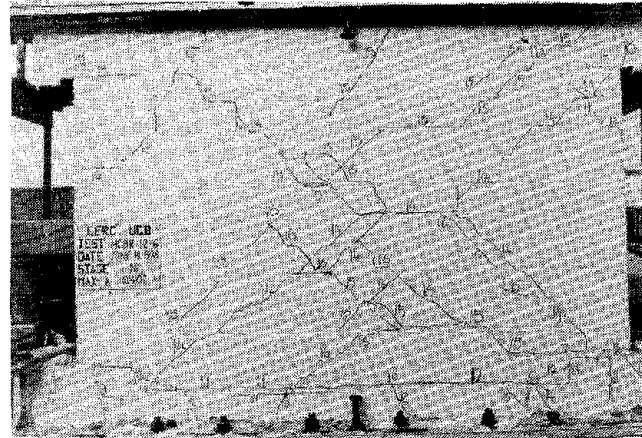


FIG. A.11 CONTINUED HCBR-12-5



●  
Stage Stage  
14 16



Stage Stage  
19 20

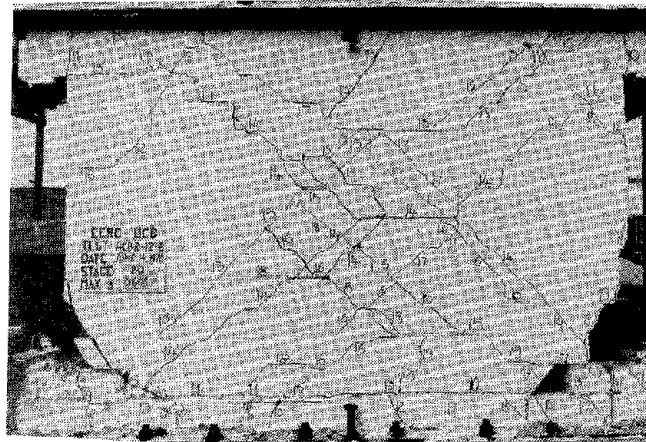
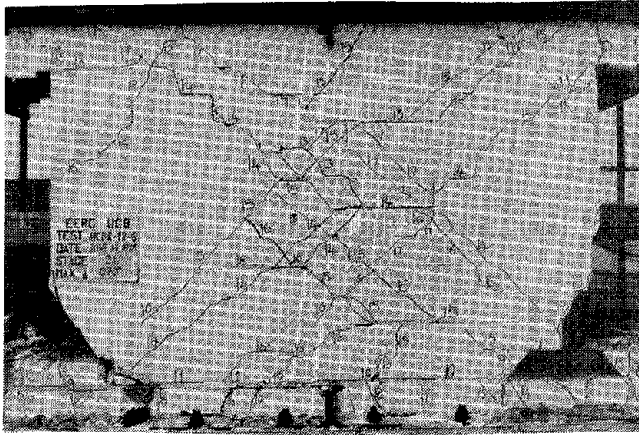
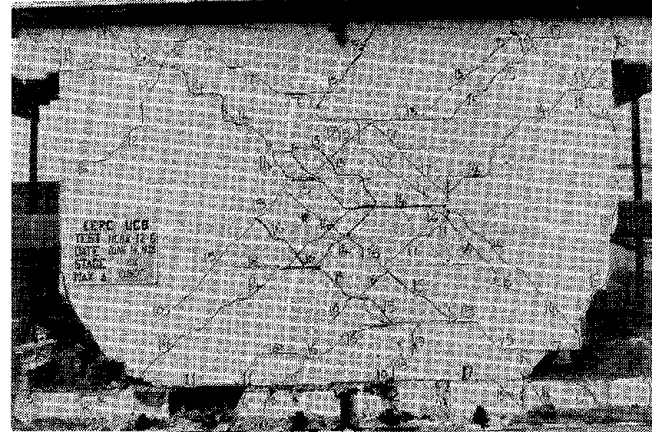


FIG. A.12 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST HCBR-12-6



HCBR-12- 9.0

Stage Stage  
21 22



HCBR-12- 6.0

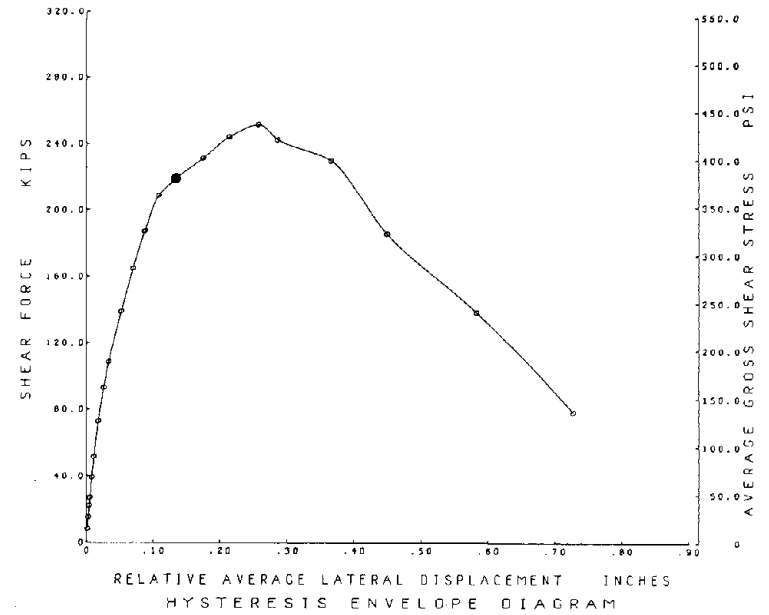
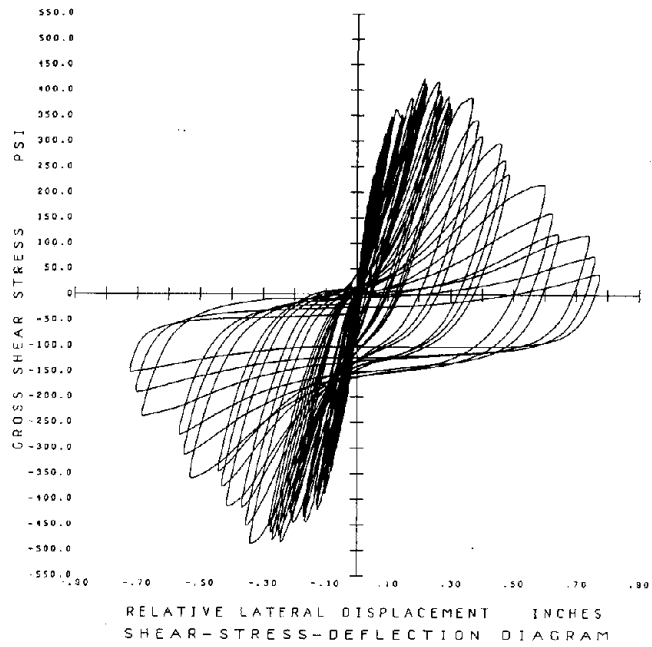


FIG. A.12 CONTINUED HCBR-12-6

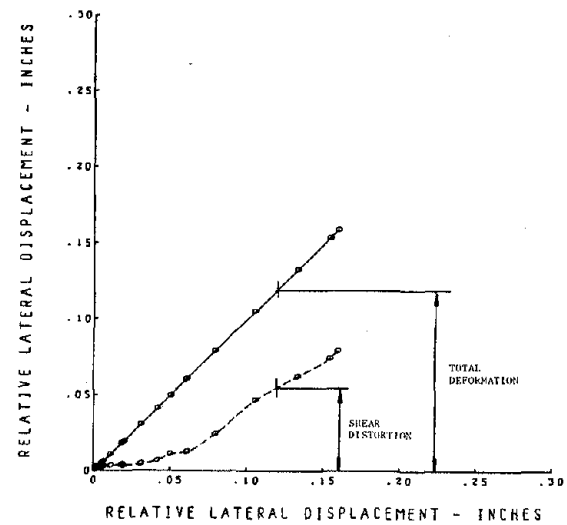
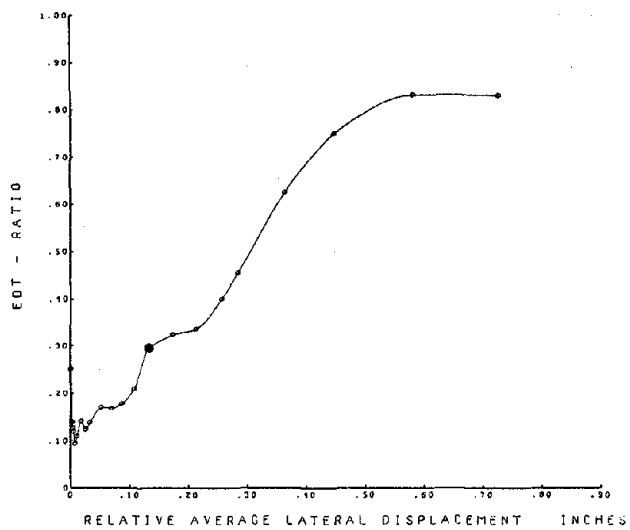
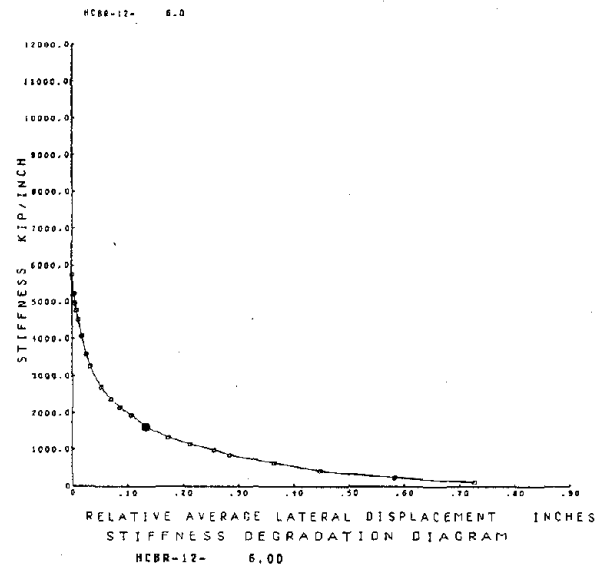
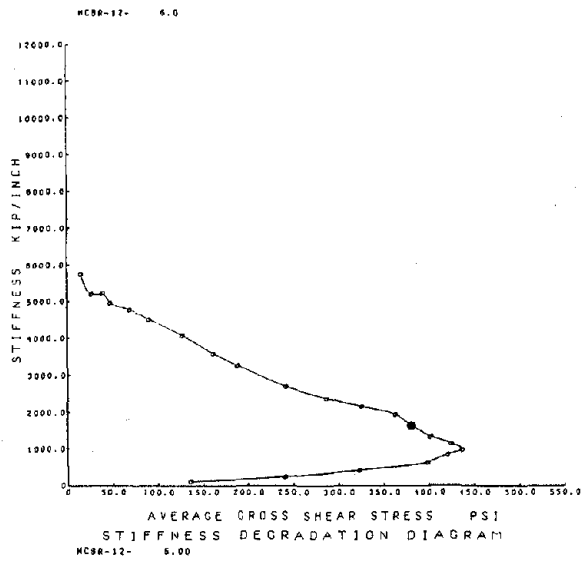
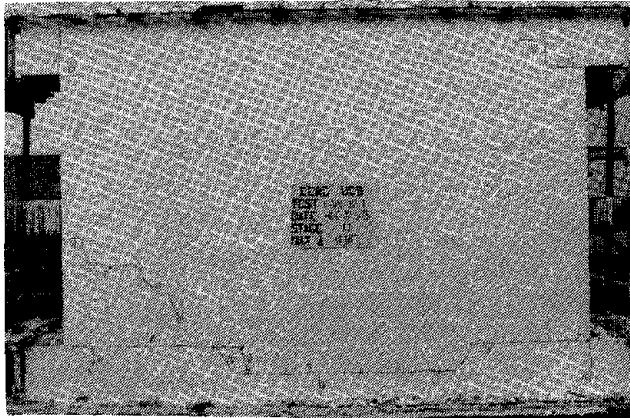
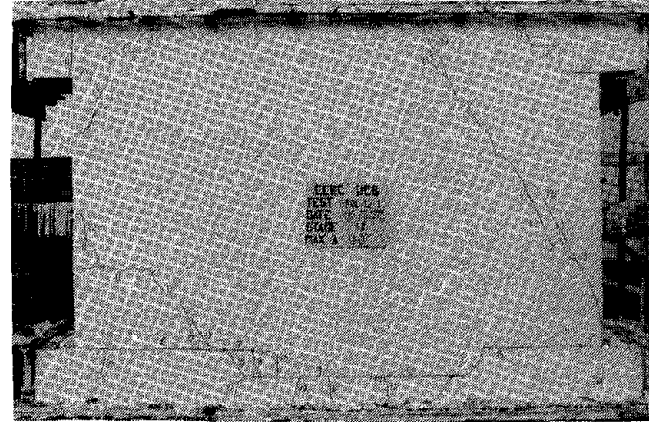


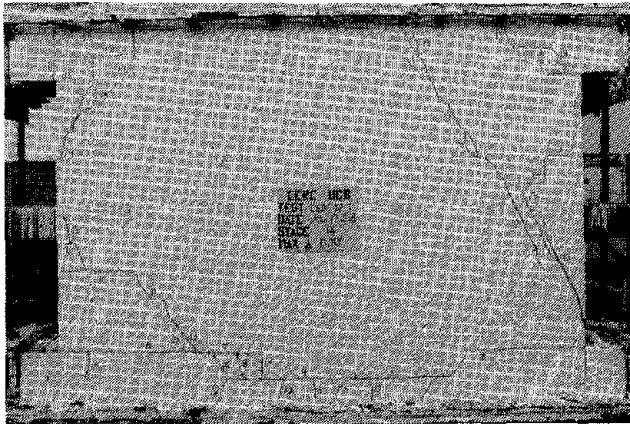
FIG. A.12 CONTINUED HCBR-12-6



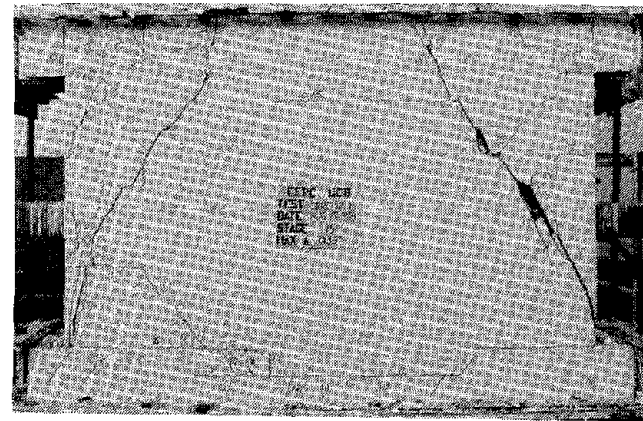
Stage 11



Stage 13



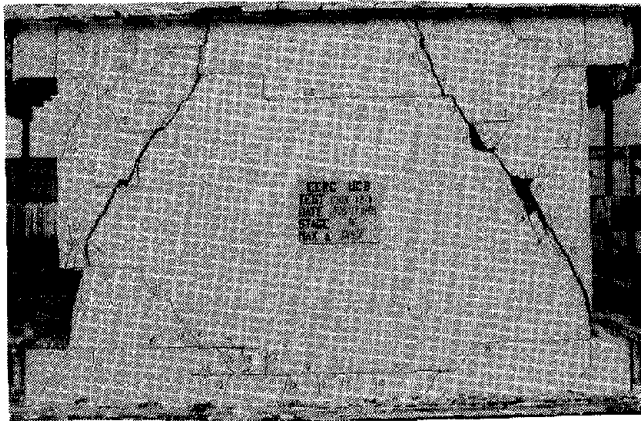
Stage 14



Stage 15

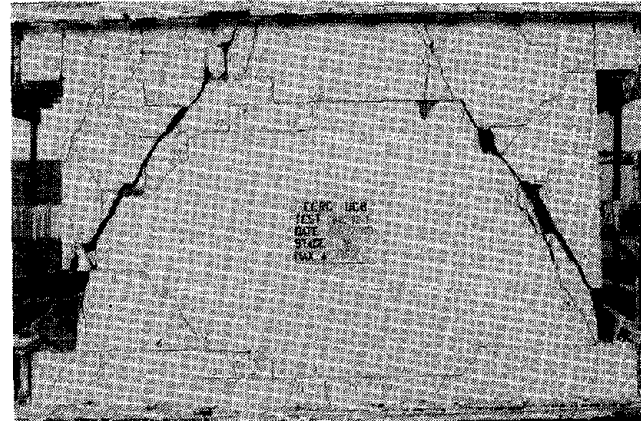
FIG. A.13 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST CBRC-12-1





CBRC-12- 1.0

Stage Stage  
16 18



CBRC-12- 1.0

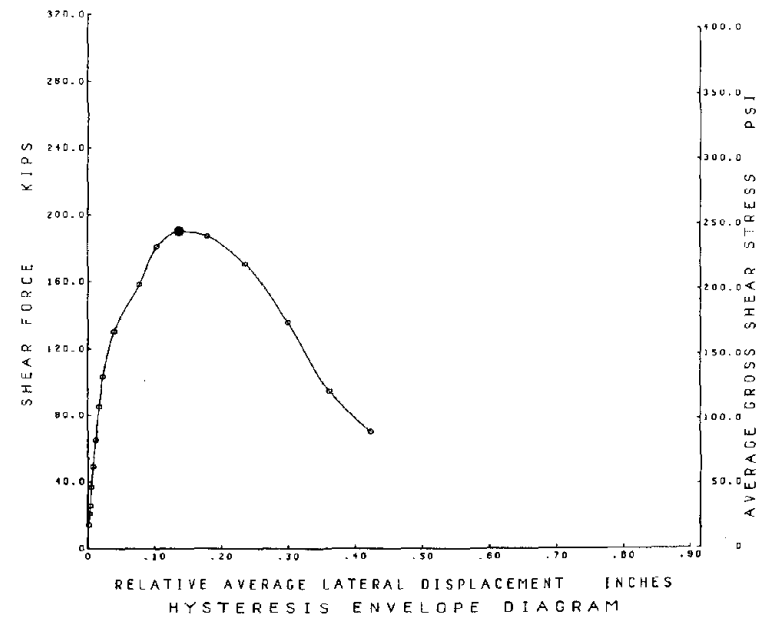
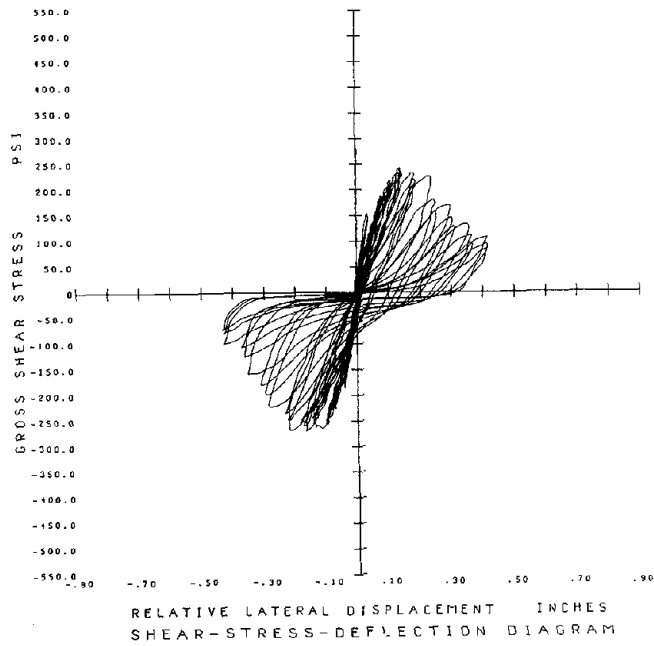


FIG. A.13 CONTINUED CBRC-12-1

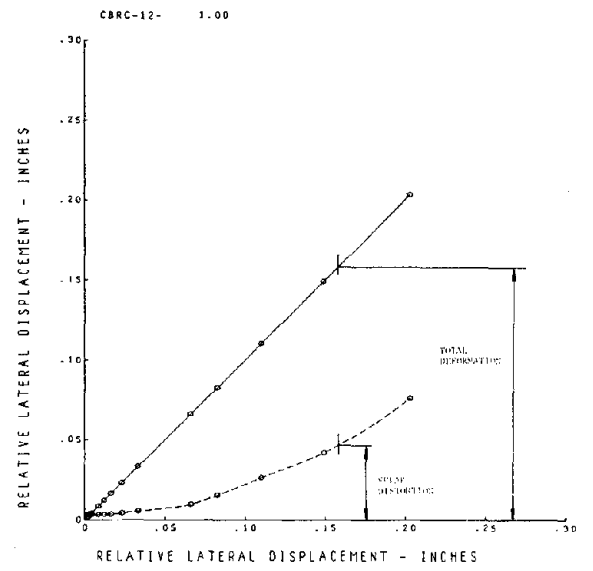
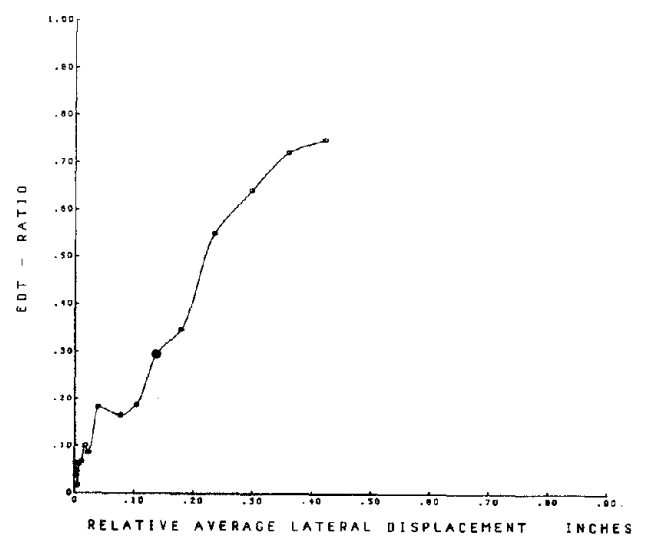
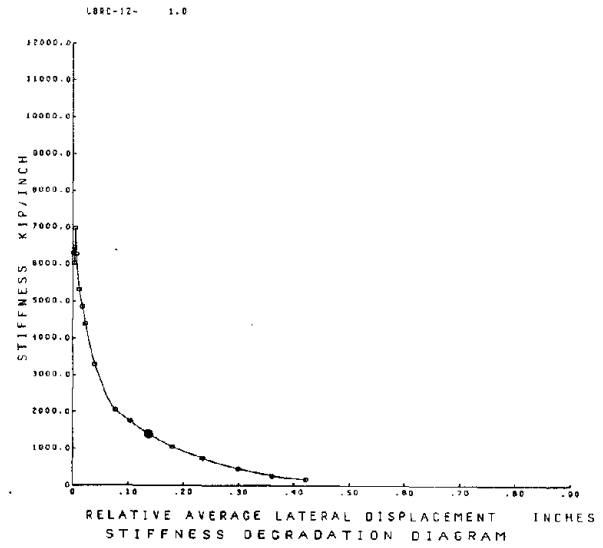
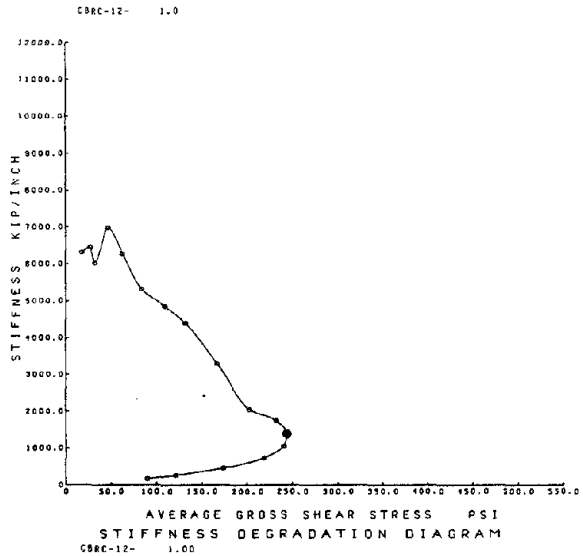
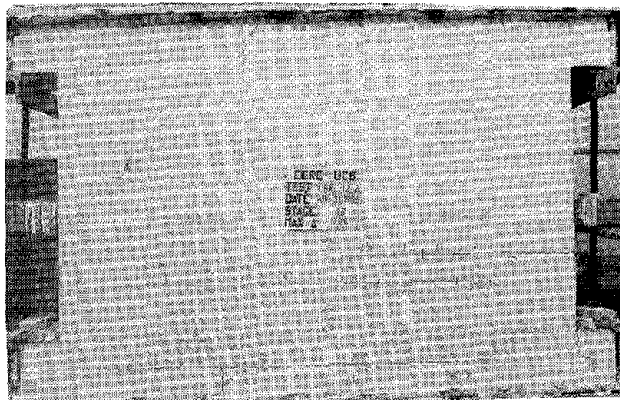


FIG. A.13 CONTINUED CBRC-12-1



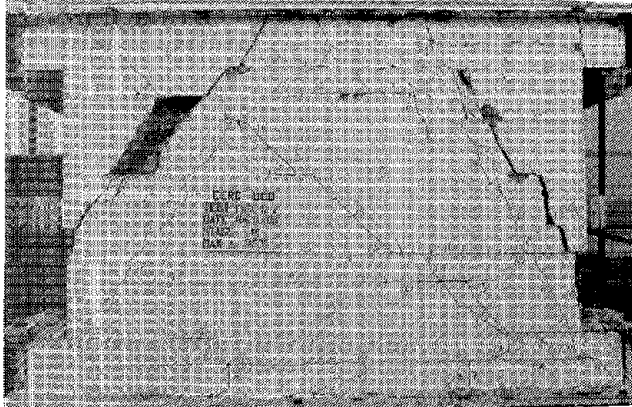
●  
Stage Stage  
13 14



Stage Stage  
16 17



FIG. A.14 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST CBRC-12-2



Stage Stage  
18 19

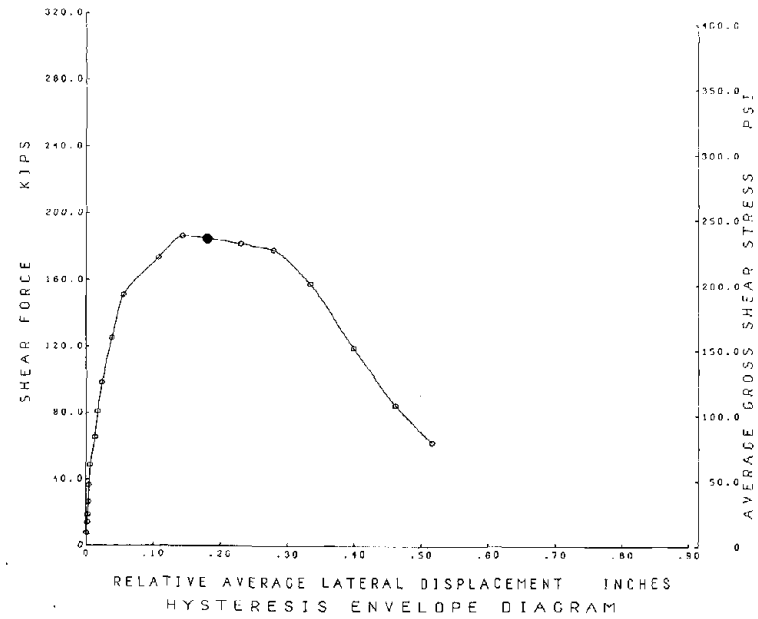
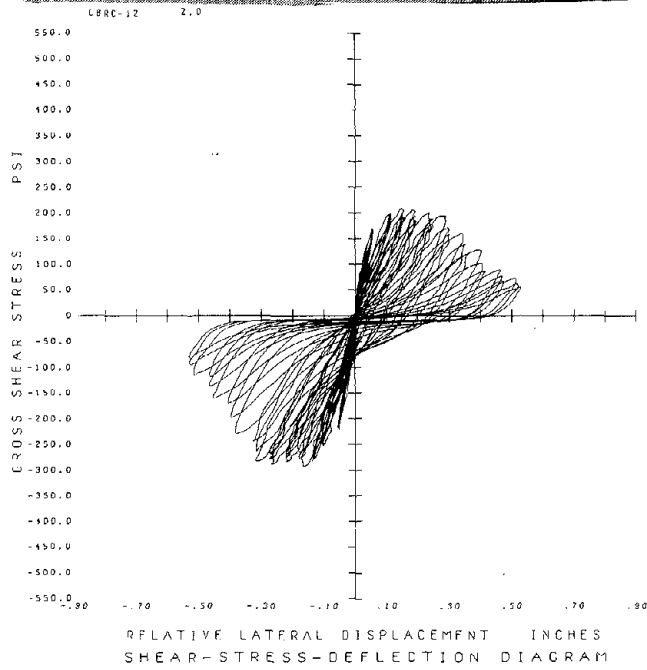


FIG. A.14 CONTINUED CBRC-12-2

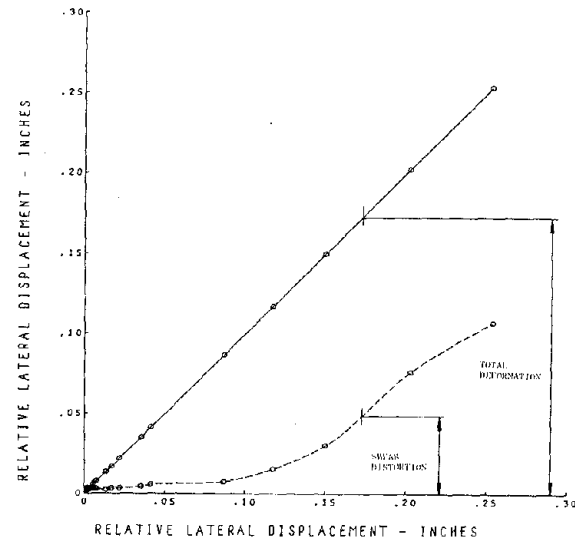
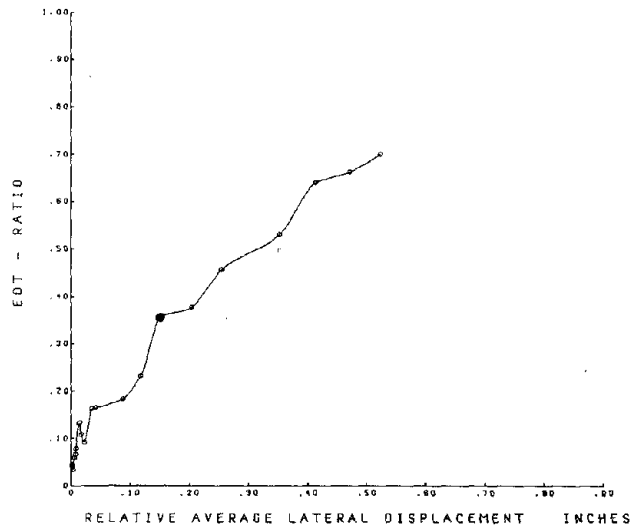
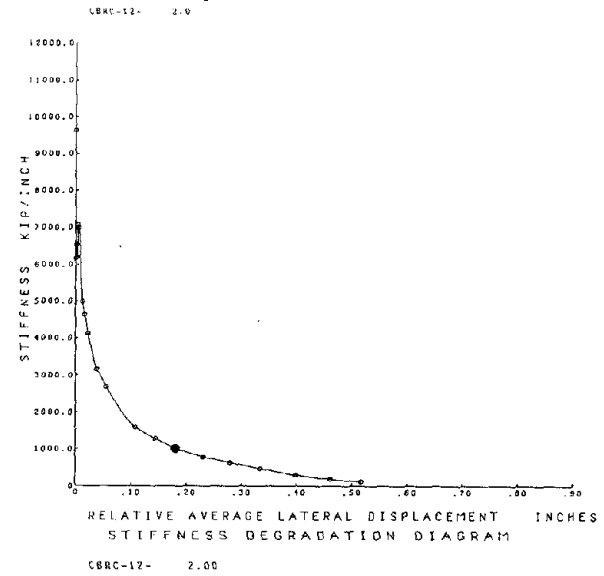
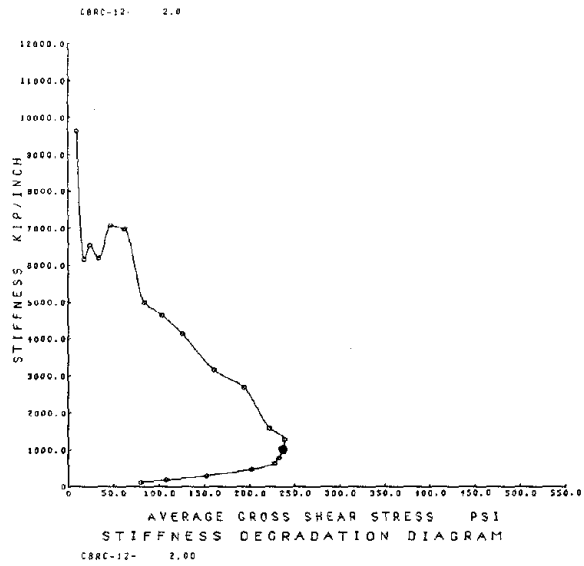
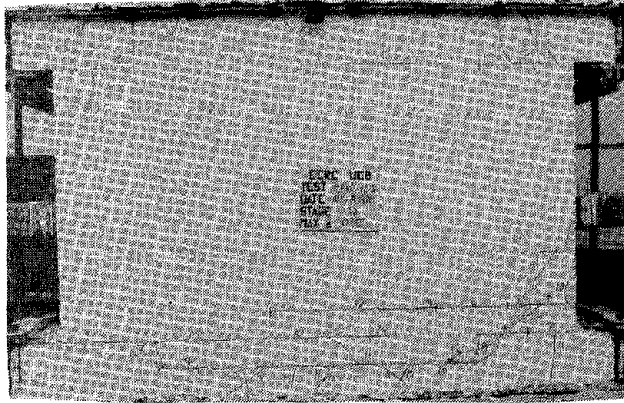
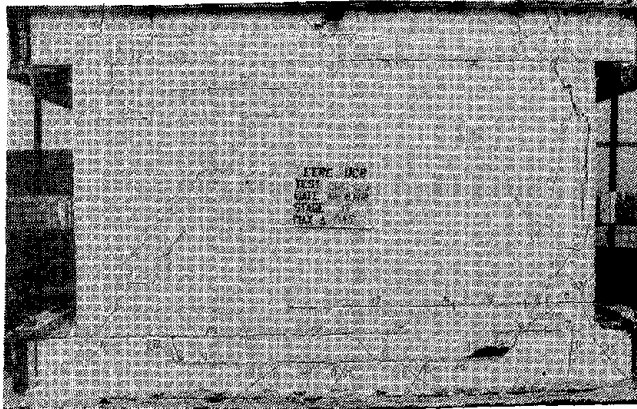
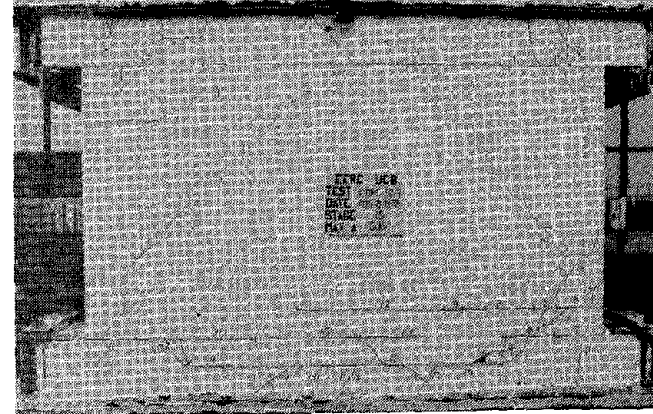


FIG. A.14 CONTINUED CBRC-12-2



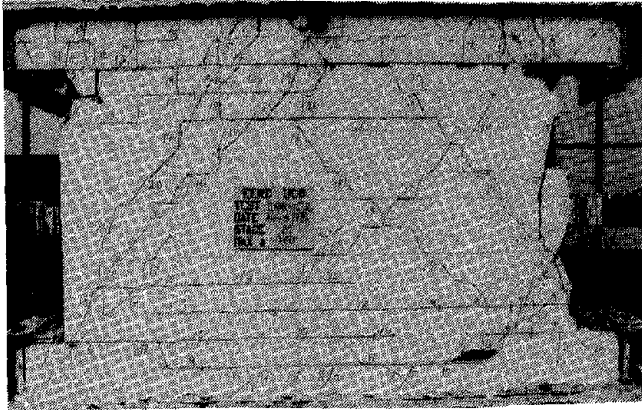
Stage Stage  
14 15



Stage Stage  
17 18

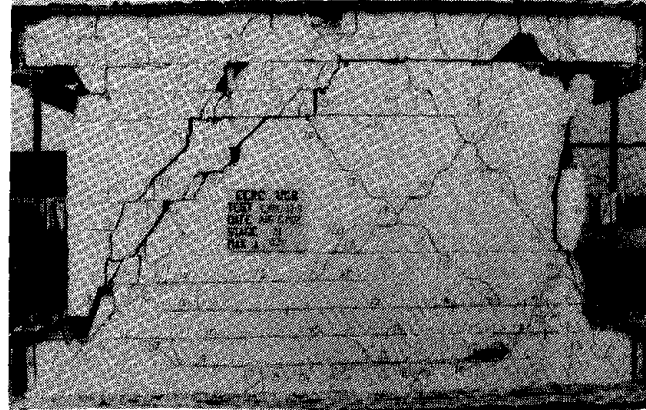


FIG. A.15 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST CBRC-12-3



CBRC-12- 3.0

Stage Stage  
20 21



CBRC-12- 3.0

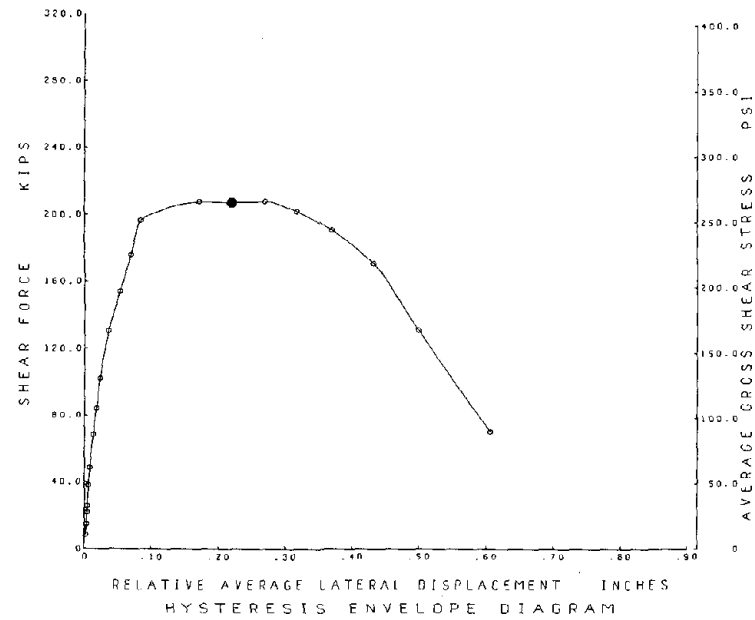
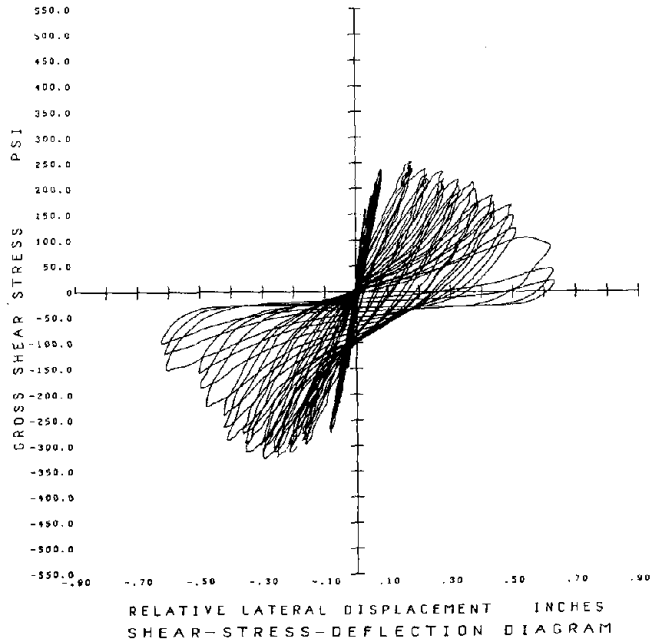


FIG. A.15 CONTINUED CBRC-12-3

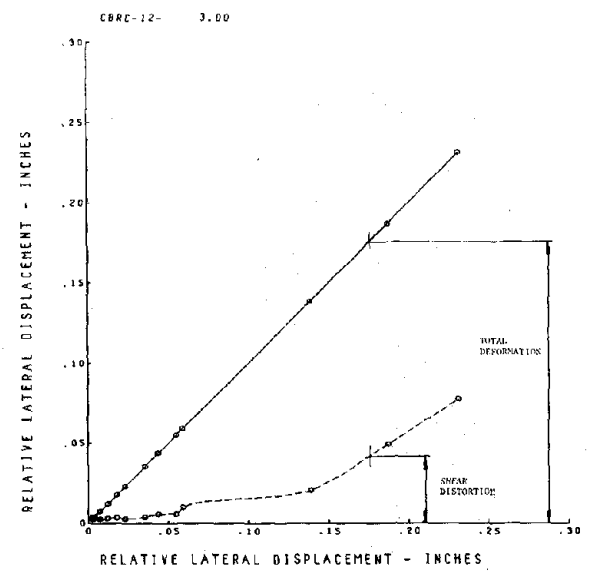
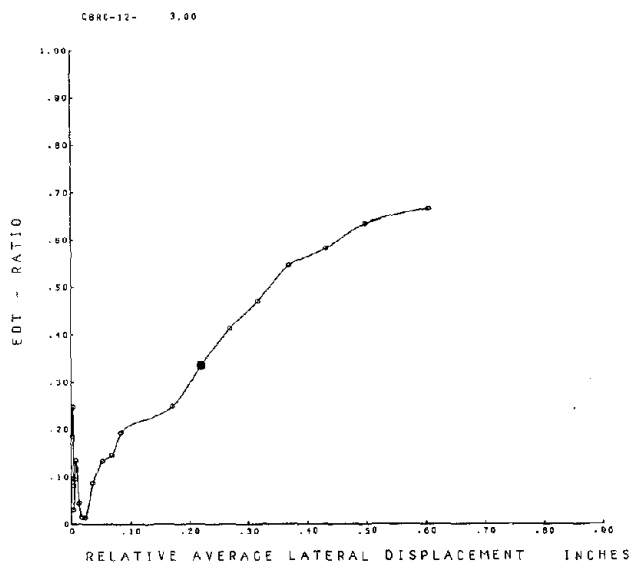
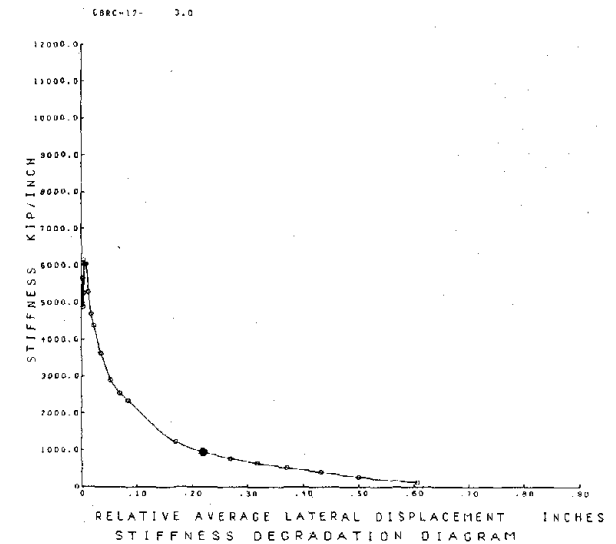
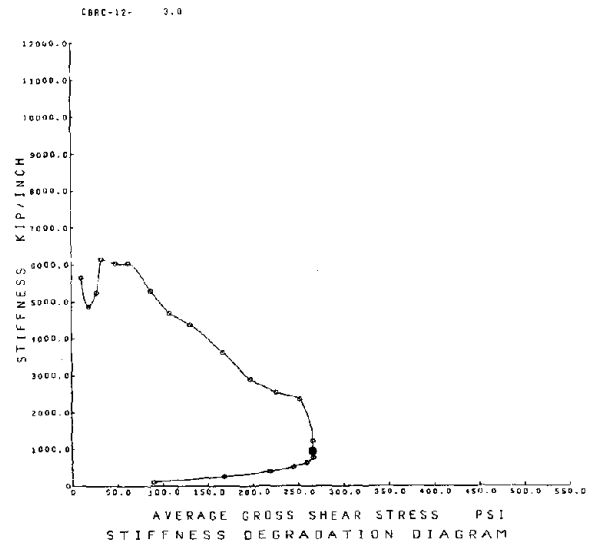
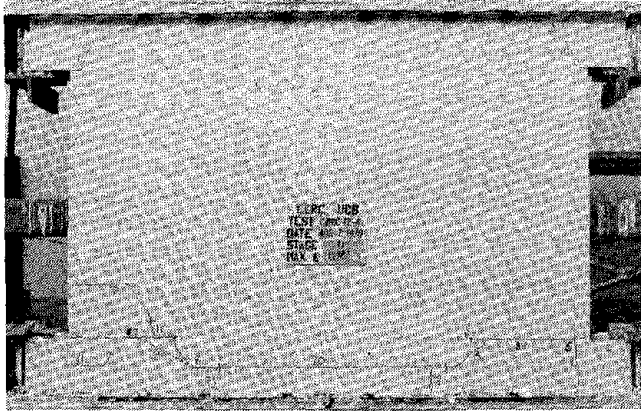
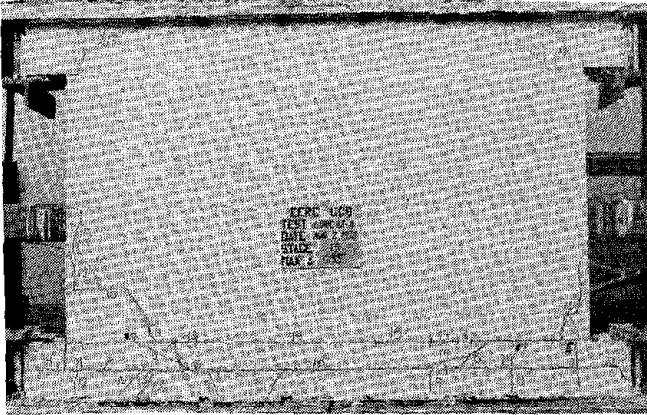
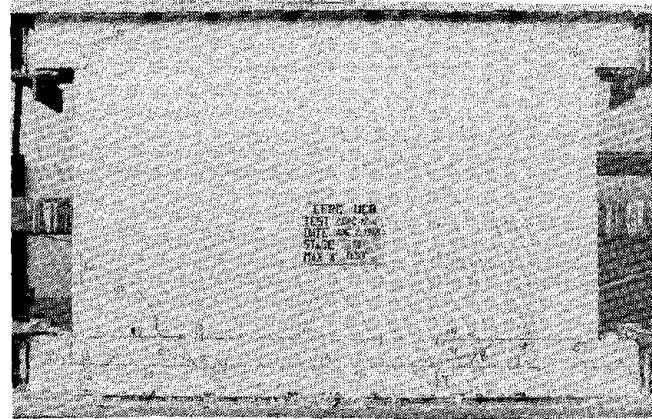


FIG. A.15 CONTINUED CBRC-12-3





Stage Stage  
11 14



Stage Stage  
15 16

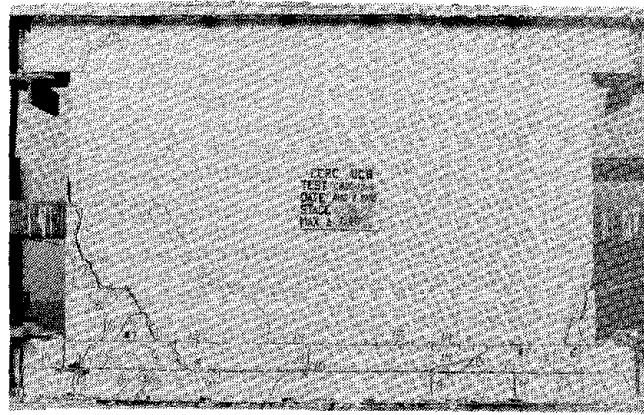
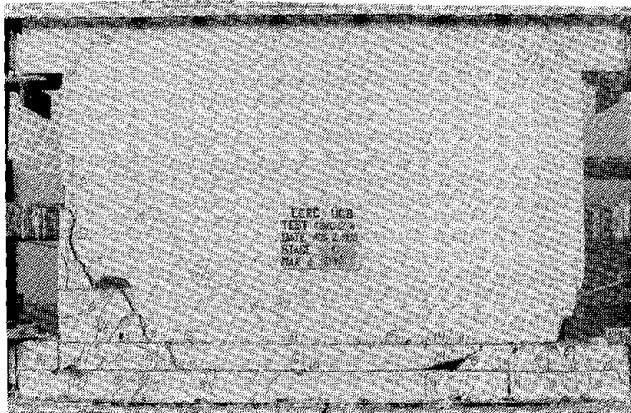
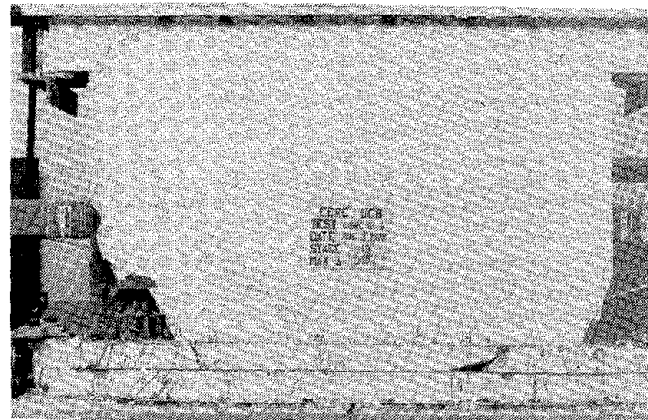


FIG. A.16 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST CBRC-12-4



CBRC-12- 1.0



CBRC-12- 1.0

Stage Stage  
17 19

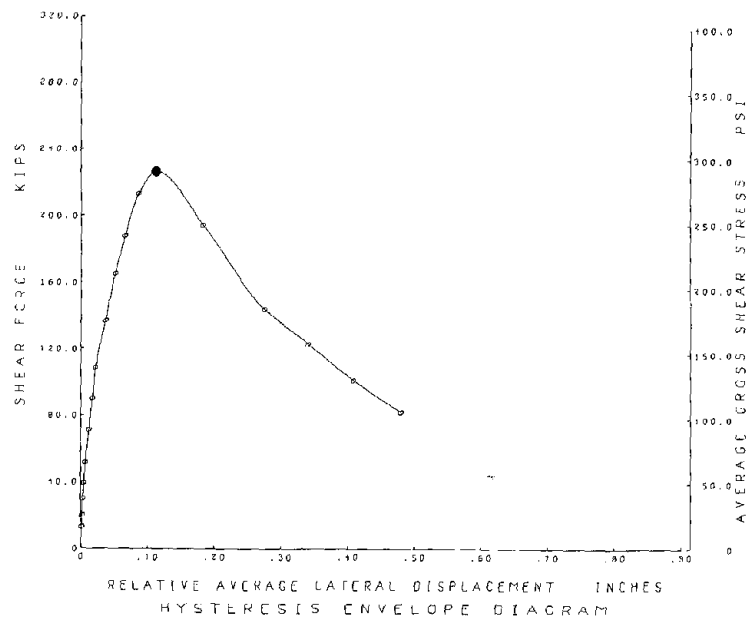
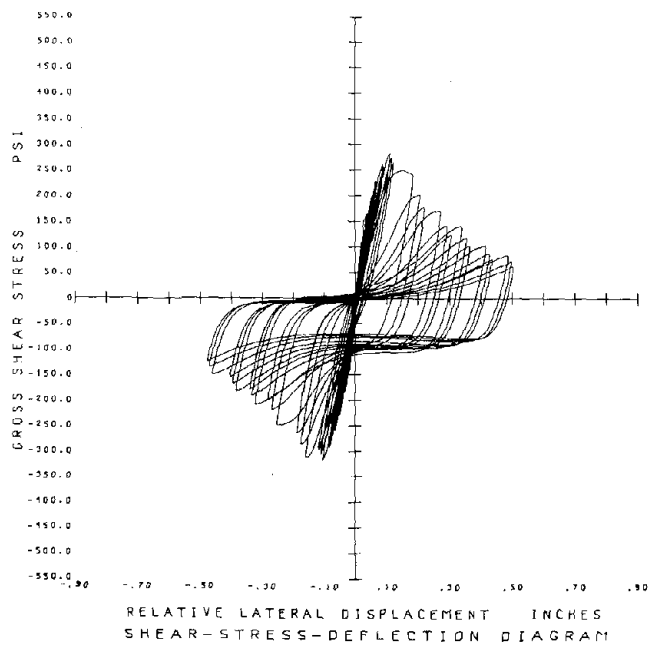


FIG. A.16 CONTINUED CBRC-12-4

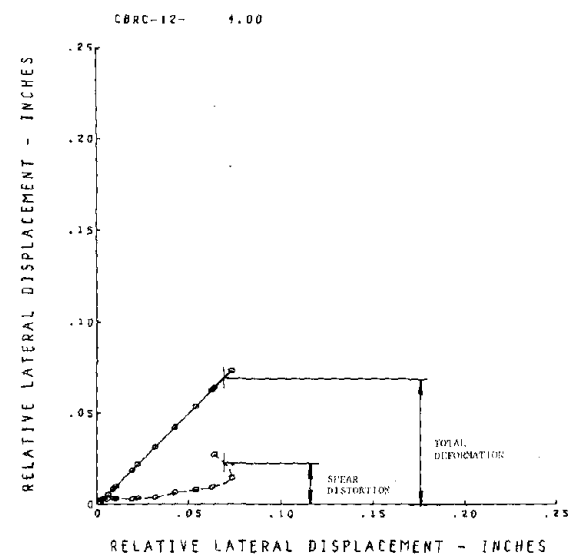
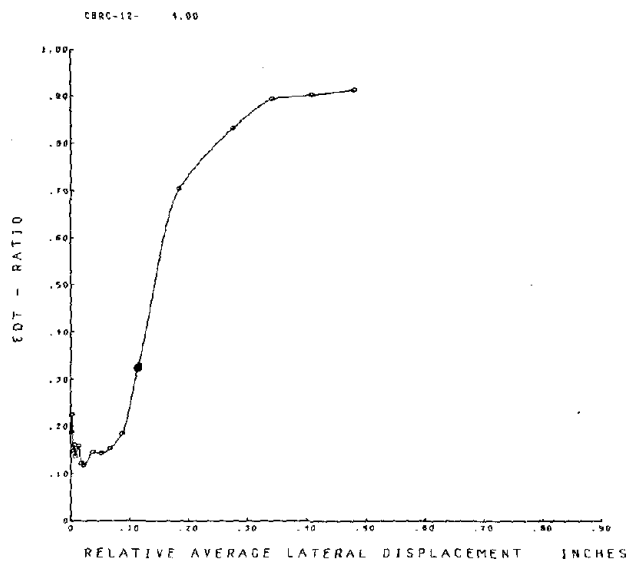
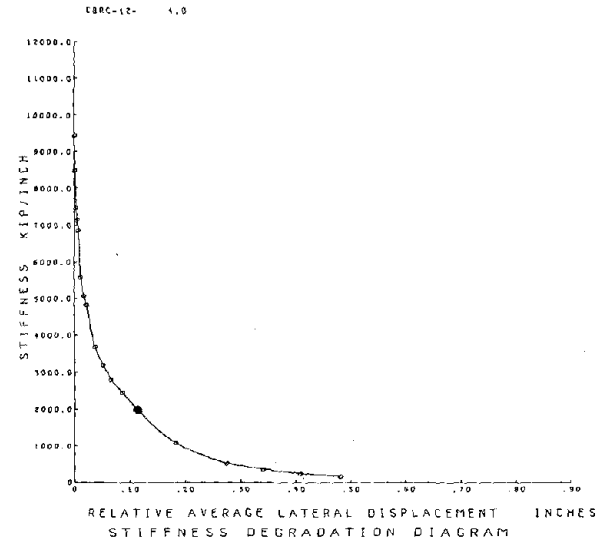
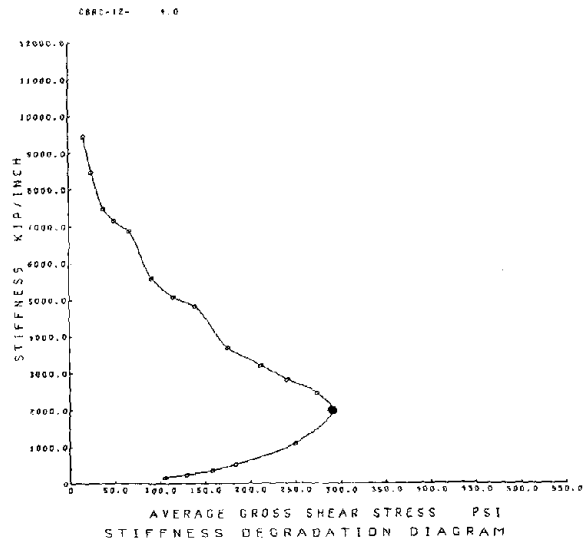
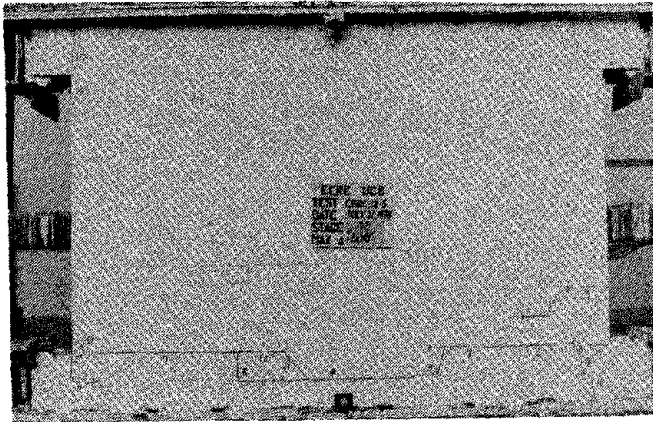


FIG. A.16 CONTINUED CBRC-12-4



Stage 11



Stage 13

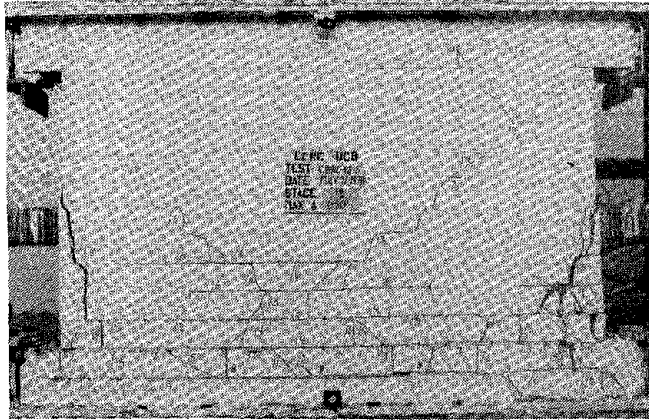


Stage 15



Stage 16

FIG. A.17 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST CBRC-12-5



Stage Stage  
18 19

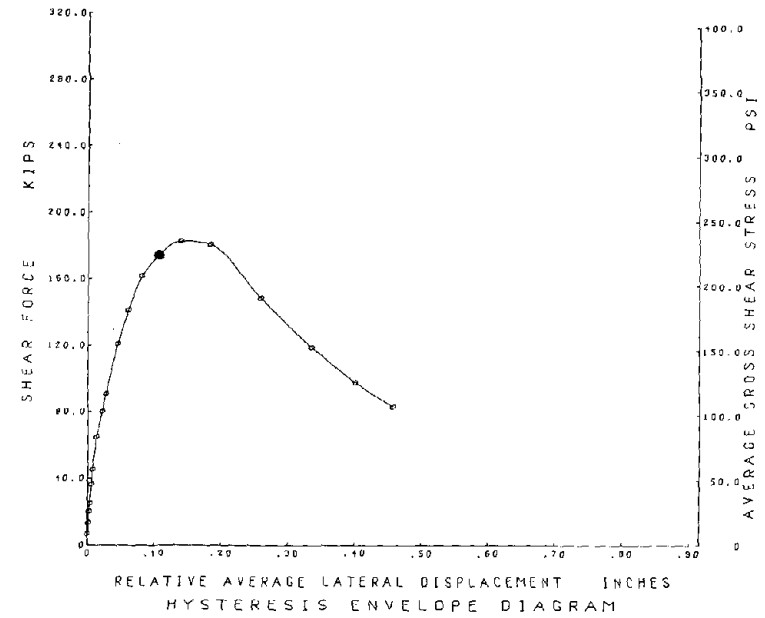
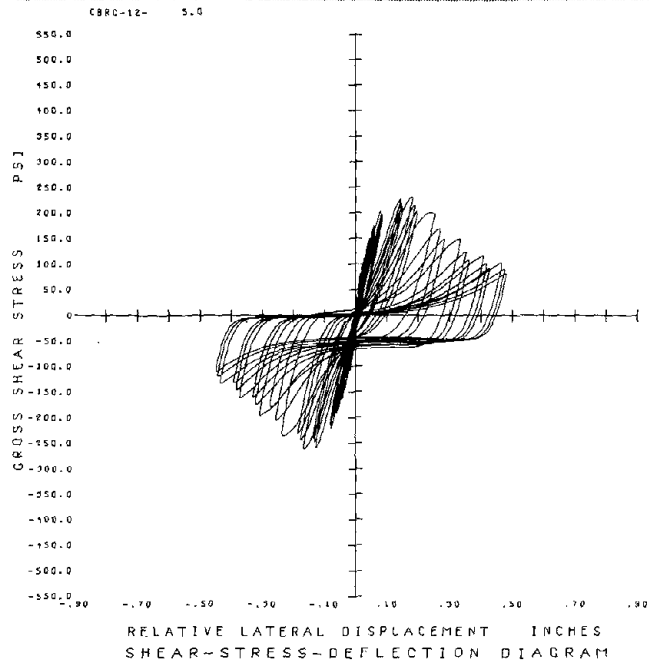


FIG. A.17 CONTINUED CBRC-12-5

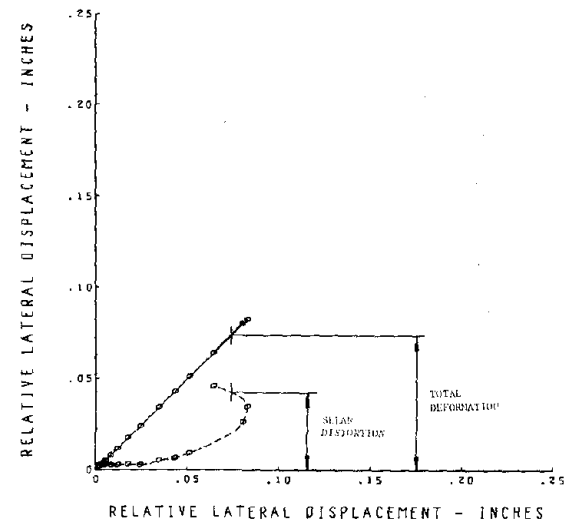
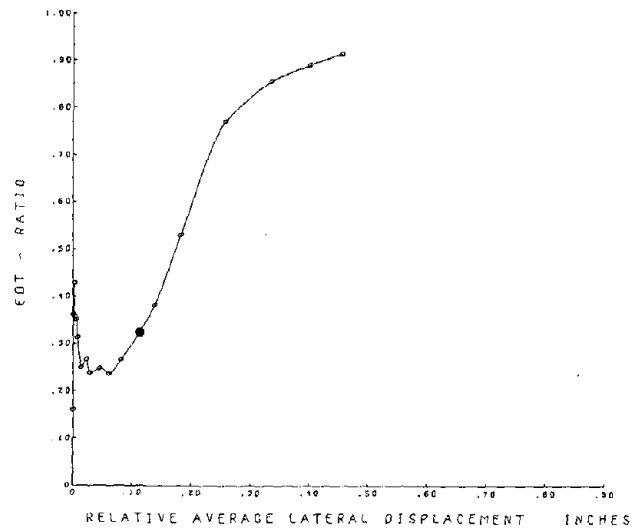
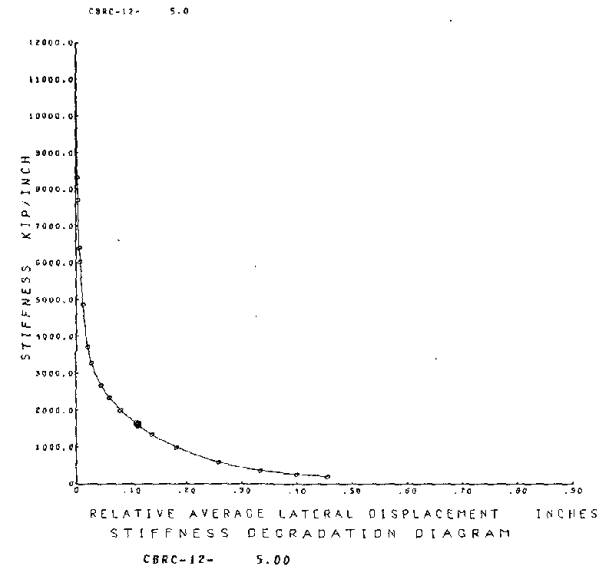
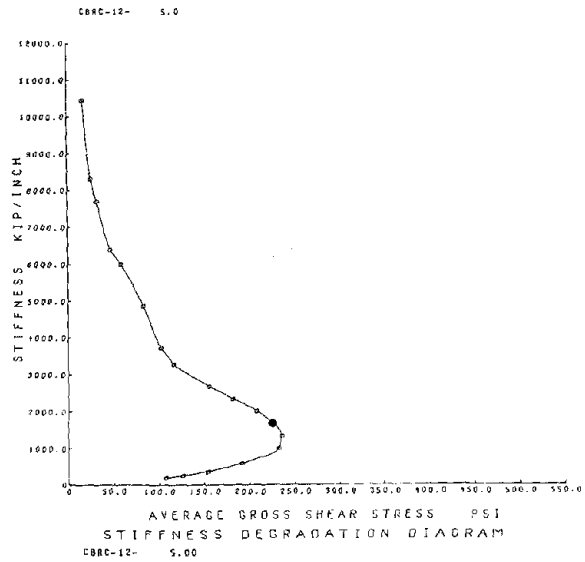
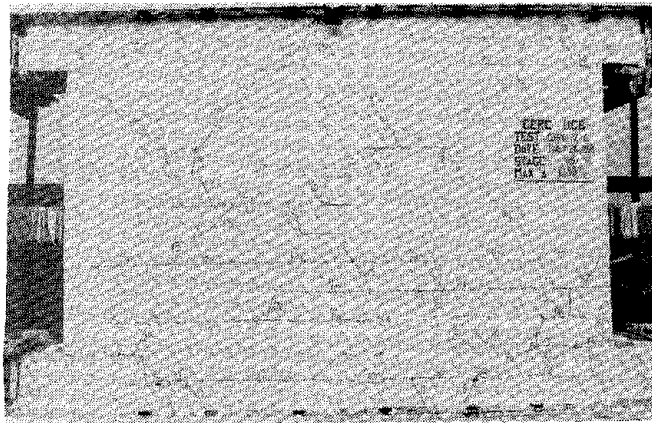
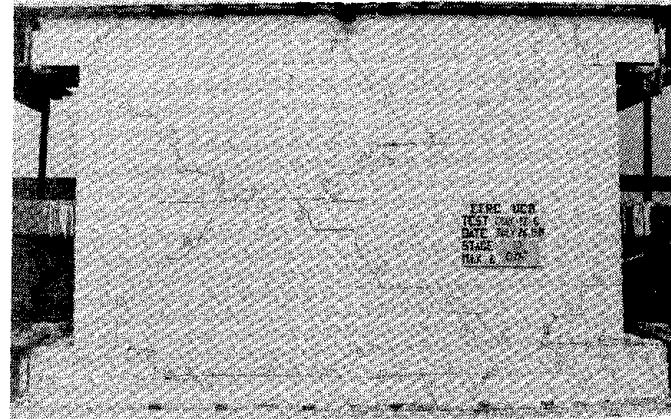


FIG. A.17 CONTINUED CBRC-12-5



●  
Stage Stage  
12 13



Stage Stage  
15 16

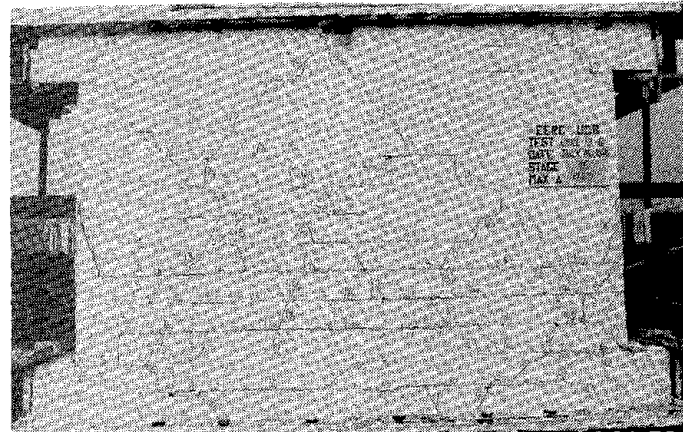
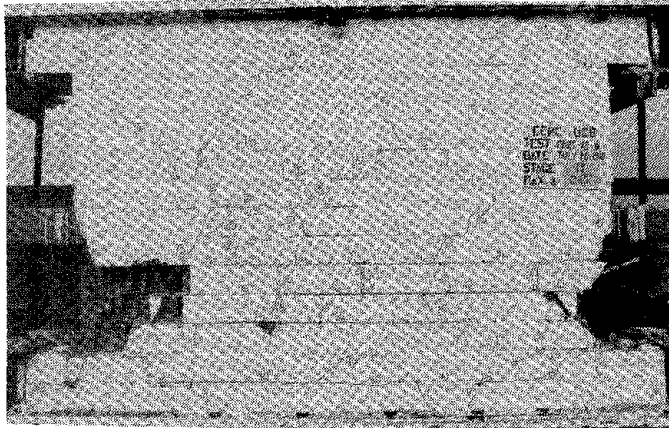


FIG. A.18 SUCCESSIVE CRACK FORMATION AND EXPERIMENTAL RESULTS TEST CBRC-12-6



Stage Stage  
17 18

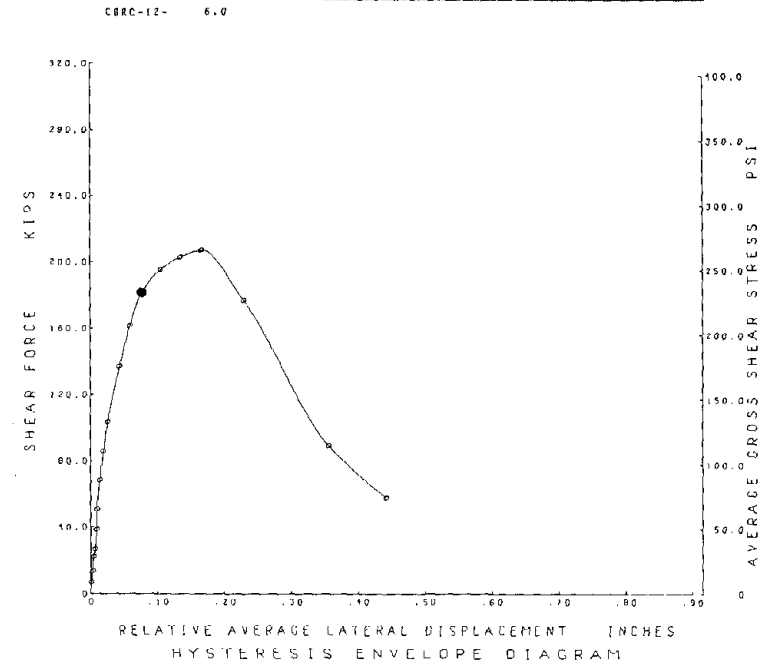
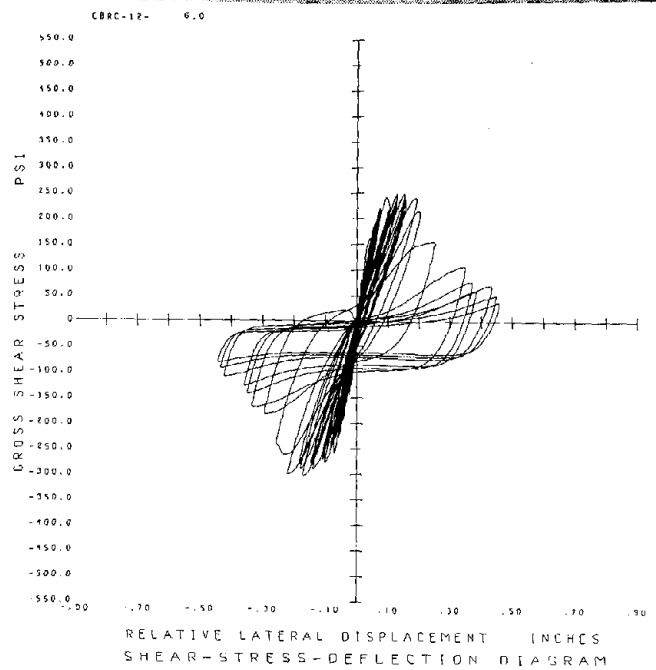
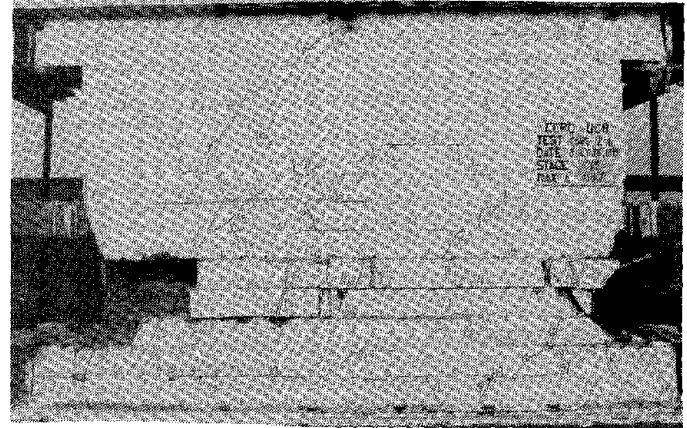


FIG. A.18 CONTINUED CBRC-12-6



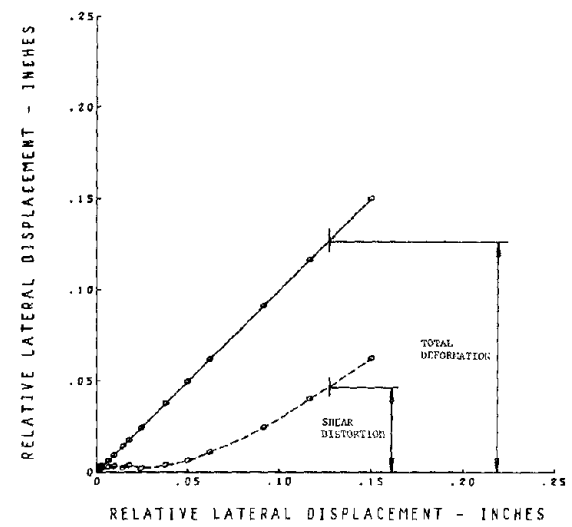
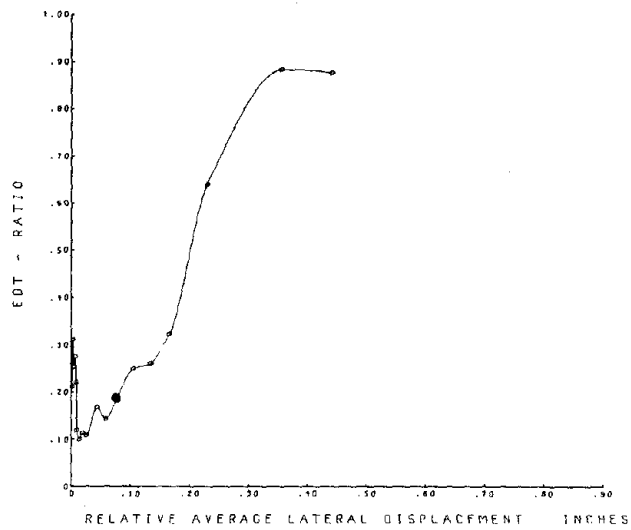
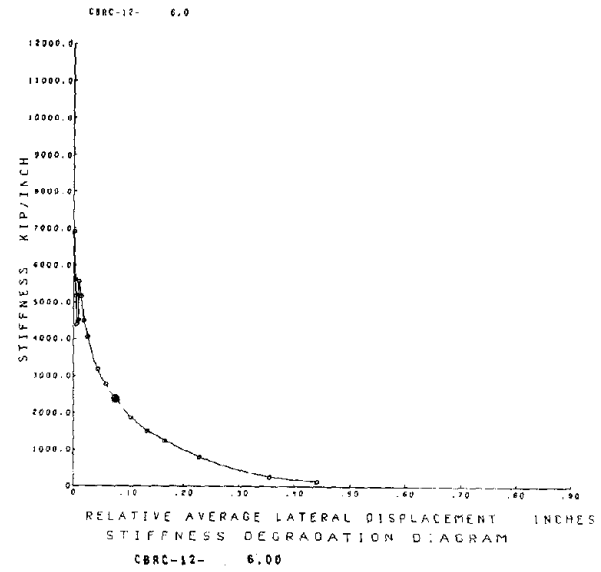
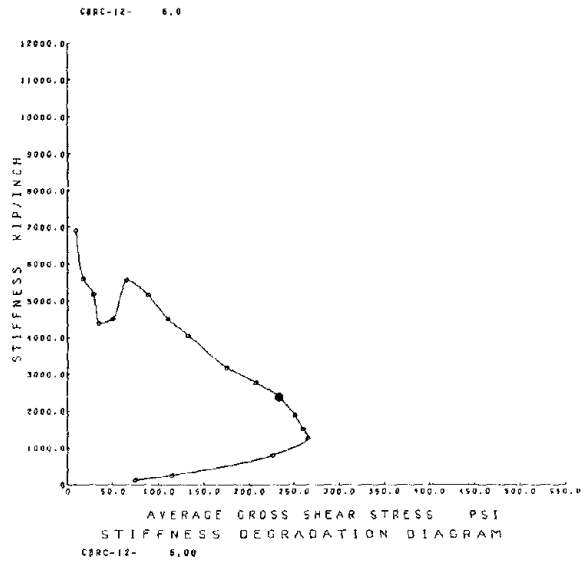
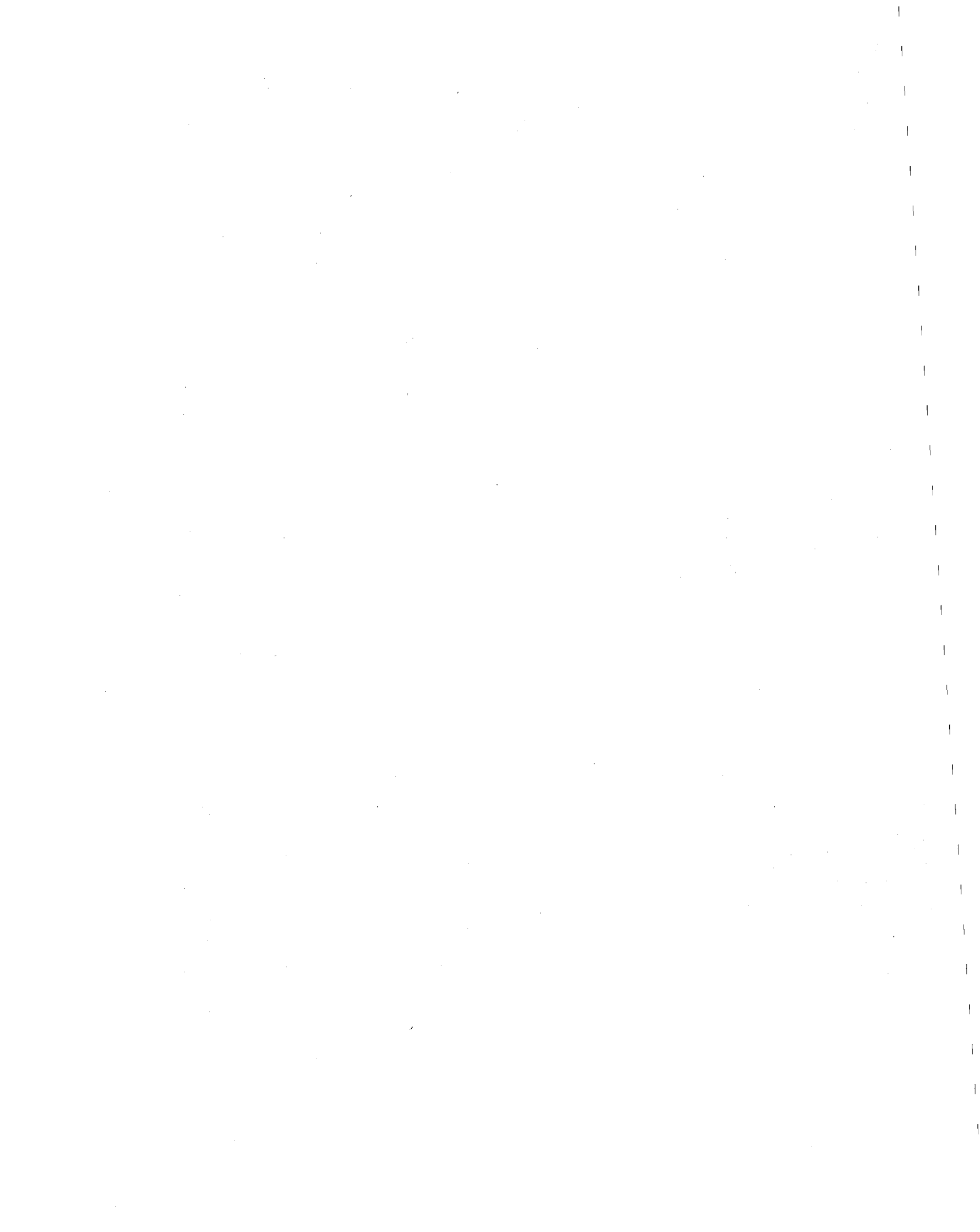


FIG. A.18 CONTINUED CBRC-12-6



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- EERC 67-1 "Feasibility Study Large-Scale Earthquake Simulator Facility," by J. Penzien, J.G. Bouwkamp, R.W. Clough and D. Rea - 1967 (PB 187 905)A07
- EERC 68-1 Unassigned
- EERC 68-2 "Inelastic Behavior of Beam-to-Column Subassemblages Under Repeated Loading," by V.V. Bertero - 1968 (PB 184 888)A05
- EERC 68-3 "A Graphical Method for Solving the Wave Reflection-Refraction Problem," by H.D. McNiven and Y. Mengi - 1968 (PB 187 943)A03
- EERC 68-4 "Dynamic Properties of McKinley School Buildings," by D. Rea, J.G. Bouwkamp and R.W. Clough - 1968 (PB 187 902)A07
- EERC 68-5 "Characteristics of Rock Motions During Earthquakes," by H.B. Seed, I.M. Idriss and F.W. Kiefer - 1968 (PB 188 338)A03
- EERC 69-1 "Earthquake Engineering Research at Berkeley," - 1969 (PB 187 906)A11
- EERC 69-2 "Nonlinear Seismic Response of Earth Structures," by M. Dibaj and J. Penzien - 1969 (PB 187 904)A08
- EERC 69-3 "Probabilistic Study of the Behavior of Structures During Earthquakes," by R. Ruiz and J. Penzien - 1969 (PB 187 886)A06
- EERC 69-4 "Numerical Solution of Boundary Value Problems in Structural Mechanics by Reduction to an Initial Value Formulation," by N. Distefano and J. Schujman - 1969 (PB 187 942)A02
- EERC 69-5 "Dynamic Programming and the Solution of the Biharmonic Equation," by N. Distefano - 1969 (PB 187 941)A03
- EERC 69-6 "Stochastic Analysis of Offshore Tower Structures," by A.K. Malhotra and J. Penzien - 1969 (PB 187 903)A09
- EERC 69-7 "Rock Motion Accelerograms for High Magnitude Earthquakes," by H.B. Seed and I.M. Idriss - 1969 (PB 187 940)A02
- EERC 69-8 "Structural Dynamics Testing Facilities at the University of California, Berkeley," by R.M. Stephen, J.G. Bouwkamp, R.W. Clough and J. Penzien - 1969 (PB 189 111)A04
- EERC 69-9 "Seismic Response of Soil Deposits Underlain by Sloping Rock Boundaries," by H. Dezfulian and H.B. Seed - 1969 (PB 189 114)A03
- EERC 69-10 "Dynamic Stress Analysis of Axisymmetric Structures Under Arbitrary Loading," by S. Ghosh and E.L. Wilson - 1969 (PB 189 026)A10
- EERC 69-11 "Seismic Behavior of Multistory Frames Designed by Different Philosophies," by J.C. Anderson and V. V. Bertero - 1969 (PB 190 662)A10
- EERC 69-12 "Stiffness Degradation of Reinforcing Concrete Members Subjected to Cyclic Flexural Moments," by V.V. Bertero, B. Bresler and H. Ming Liao - 1969 (PB 202 942)A07
- EERC 69-13 "Response of Non-Uniform Soil Deposits to Travelling Seismic Waves," by H. Dezfulian and H.B. Seed - 1969 (PB 191 023)A03
- EERC 69-14 "Damping Capacity of a Model Steel Structure," by D. Rea, R.W. Clough and J.G. Bouwkamp - 1969 (PB 190 663)A06
- EERC 69-15 "Influence of Local Soil Conditions on Building Damage Potential during Earthquakes," by H.B. Seed and I.M. Idriss - 1969 (PB 191 036)A03
- EERC 69-16 "The Behavior of Sands Under Seismic Loading Conditions," by M.L. Silver and H.B. Seed - 1969 (AD 714 982)A07
- EERC 70-1 "Earthquake Response of Gravity Dams," by A.K. Chopra - 1970 (AD 709 640)A03
- EERC 70-2 "Relationships between Soil Conditions and Building Damage in the Caracas Earthquake of July 29, 1967," by H.B. Seed, I.M. Idriss and H. Dezfulian - 1970 (PB 195 762)A05
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