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REPORT NO. 3.2(b1)

OUT-OF-PLANE DYNAMIC TESTING OF CONCRETE MASONRY WALLS

VOLUME I: FINAL REPORT

by

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PREFACE

This report presents the results of Category 3, Task 3.2(b1) of the U.S. Coordinated Program for Masonry Building Research. The program constitutes the United States part of the United States-Japan coordinated masonry research program conducted under the auspices of The Panel on Wind and Seismic Effects of the U.S.-Japan Natural Resources Development Program (UJNR).

This report is based on work supported by the National Science Foundation under Grant No. ECE 8701512. Program Director: Dr. A.J. Eggenberger.

The program was conducted at Agbabian Associates for the University of Southern California.

Material, labor, and crane needed for construction and lifting of test wall panels were donated by the Concrete Masonry Association of California and Nevada and the Masonry Institute of America.

Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the authors and do not necessarily reflect the views of the National Science Foundation, the United States Government, and/or the Masonry Industry.

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Concrete blocks were donated by the Concrete Masonry Association of California and Nevada. Construction and materials for the test wall panels, laboratory testing of materials, crane rental and disposal of the test wall panels were all contributed by the Masonry Institute of America. The coordination by the Masonry Institute of America was extremely helpful to the research team throughout the development and testing conducted under this program.

James Amrhein, Executive Director, Michael Merrigan, Staff Engineer, of the Masonry Institute of America, and Stuart Beavers, Executive Director of Concrete Masonry Association of California and Nevada, provided an extremely valuable contribution during all phases of this program.

The enthusiasm of Mr. Bill Grindle of Blocklite and the high quality of construction provided by R.E. Williams and Son contributed heavily to the success of this program.

Dr. A.J. Eggenberger served as Technical Director of this project for the National Science Foundation and maintained scientific and technical liaison with the principal investigator throughout all phases of the research program. His contribution and support is greatly appreciated.

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FOREWORD

This report is one of a two volume final report prepared for the National Science Foundation under Grant No. ECE 8701512. The two volumes provide results and discussions of the research effort. These results are discussed and evaluated.

Volume I provides an introduction, a description of experimental program, a summary, and discussion of test results.

Volume II provides more detailed selected test results processed during this program. The plots presented were filtered and presented as time histories. A video tape is also available for this research effort.

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SECTION 1

INTRODUCTION

1.1 BACKGROUND

Category 3, Task 3.2(b1) is part of an overall TCCMAR (Technical Coordinating Committee for Masonry Research) research program that comprises ten categories of tasks which form an integrated, interactive, step-by-step research program. The purpose of this program is to develop improved methods for the seismic design and analysis of reinforced masonry buildings. The Category 3 research has four tasks (i.e., Tasks 3.1(a), 3.1(b), 3.2(a), and 3.2(b)), and these tasks predominantly involve experimental studies of reinforced masonry walls.

This program was directed towards "dynamic out-of-plane testing of reinforced concrete block masonry walls." Two parallel programs were also conducted under Task 3 out-of-plane testing. The first program focused on "montonic and cyclic response of concrete block masonry walls." This program was conducted at Drexel University and represents the static version of the work reported herein. The second program focused on static and dynamic testing of clay block masonry walls. The second program was conducted by Computech Engineering as Task 3.2(b2). The three programs were closely coordinated from inception. Test panels, test setups, applied loads, boundary conditions, and material properties were closely selected to allow for a comparison of the responses from the three test programs.

The University of Southern California was awarded a grant (ECE 8701512) by NSF to study the "Out-of-Plane Dynamic Testing of Concrete Block Masonry Walls (Task 3.2 b1)." A test program was developed during this study to evaluate the dynamic response of these panels. The test wall panels were 4-1/2 in. and 6 in. thick for height-to-thickness ratios (H/t) of 53 and 43, respectively. Input motions were applied at the base of the panel and at the roof level. Base motions were obtained from actual earthquake records while roof level diaphragm motions were obtained from both actual roof diaphragm instrument records, in addition to three-dimensional analyses of typical buildings. The 4-1/2 in. wall panel thickness was selected to provide tests for panels with H/t ratios up to 53 using a test setup with a maximum height capacity of 20 ft.

The tests were conducted at Agbabian Associates Test Facilities in El Segundo, California. The scope of this program was directed towards (1) developing a test plan, (2) preparing test setup, (3) constructing panels, (4) conducting dynamic testing, (5) providing preliminary data processing of test results, and (6) summarizing test observations in a final report.

Additional funds were authorized for Computech Engineering to develop a mathematical model to simulate the response of the test wall panels. Computech will utilize the test data generated by this program for analytical/experimental correlation. The results of these efforts will be evaluated and integrated with the remainder of the TCCMAR program in Task 10 effort.

The results of this test program indicate the following: (1) these walls will be dynamically stable during earthquakes, (2) all panels responded elastically to Sequences 1 through 6 which represent typical earthquake motions in various seismic zones of the United States, (3) the first two wall panels began to go into inelastic range only after an earthquake shaking (M9), which represents a somewhat larger seismic motion than that associated with Zone 4, was applied to these panels, (4) the partially grouted wall panels had less mass and were less affected by Motion M9 and sustained 30 earthquake shakings without going into inelastic range, (5) the response of wall panels with and without reinforcing bar lap splices was identical, and (6) all wall panels exhibited considerable ductility.

Most important finding of the present research program is that test results indicate that tall slender reinforced masonry walls, constructed with adequate quality control, can safely sustain a large number of moderate and severe earthquakes. The slenderness and reduced mass of these walls result in a more ductile lighter wall that can sustain severe shaking without the risk of instability or sudden brittle failure.

This report provides an overview of the test program, a description of test specimens, material properties, test setup as well as summary of test results and discussion of these results.

1.2 OBJECTIVES

The objectives of the research program (Grant No. ECE 8701512) were to (1) modify test facility at Agbabian Associates to accommodate two types of wall panel thicknesses and higher vertical loads, (2) design an instrumentation plan and data acquisition system to retrieve dynamic test data, (3) build four wall panels and provide for mortar, grout, and prism laboratory samples, (4) conduct a test program using simulated earthquake motions representing various types of earthquake shaking, (5) convert all test data from analog to digital form, (6) store digital data on tapes for detailed processing in future phases and to conduct preliminary processing of some of the measured data to verify the accuracy of test procedures and results, and (7) document the results in a final report.

The specific objectives of the program were to generate test data for (1) verifying analytical models for out-of-plane dynamic response of concrete block masonry walls, (2) supporting the development of strength design procedures for masonry walls, (3) evaluating the seismic response of tall slender walls as designed by current building codes, and (4) evaluating a significant number of parameters used in the design and construction of these walls.

SECTION 2

EXPERIMENTAL PROGRAM

2.1 INTRODUCTION

The experimental program is designed to test a wall as part of a reinforced concrete block masonry building subjected to an out-of-plane dynamic seismic environment. A typical full-scale wall segment of a building was selected. Realistic kinematic seismic motions at the top and bottom of the masonry panels (walls) were used. These motions along with panel geometries were used to design a full-scale, dynamic test program. An ensemble of bounding earthquake input motions were used at the base. The diaphragm motions associated with base motions were also included, thus allowing the input motions to be applied in pairs, one at the base and one at the top of the panel. Both soft and stiff roof diaphragm materials were included in earlier developments of the earthquake motion pairs used in this program.

The specific objectives of the test program were to (1) provide data for verification of analytical programs for these walls thus allowing for the expansion of the data base for slender masonry walls beyond the matrix of tested walls, (2) support the development of strength design procedures for masonry walls, (3) evaluate the seismic response and ductility of tall slender masonry walls as designed by current building codes, (4) evaluate a number of significant parameters used in the design and construction of these walls, (5) study the stability of slender concrete block masonry walls under dynamic seismic loading, and (6) assess current strength/deflection limitations on this type of construction.

2.2 TEST SPECIMENS

Four walls were built to a height of 20 ft thus presenting slightly less than three story high walls (Fig. 2-1). The four wall specimens were designed to test the most severe conditions in various seismic zones of the United States. A height-to-thickness ratio (H/t) of 43 was selected for three panels. This H/t ratio represents current trends in slender wall design. The fourth panel was designed for an H/t ratio of 53. This represents an upper bound on current practice. The test matrix is shown in Table 2-1 and reflects the following.

1. The designated ultimate compressive strength of masonry unit being 2500 psi is compatible with the TCCMAR program concrete block masonry unit designated strength.

2. The location of the vertical rebar in the center of the second cell from the end of the wall is desirable since it allows the test panel to represent a segment of a continuous wall (Fig. 2-2).

3. The 6-in. walls had 2-#5 vertical reinforcing bars. The percent of reinforcement is approximately 31% of the balanced reinforcement (ρ_b). This percent was lower than the 0.5 ρ_b recommended by the ACI-SEAOSC Slender Wall Task force (Simpson et al., 1982) to prevent brittle failure.

4. The 4.5-in. wall had 2-#4 vertical reinforcing bars. The percent of reinforcement is approximately 25% of the balanced reinforcement (ρ_b).

5. The sequence of four walls was selected to allow for the variation of only one parameter at a time, thus facilitating the comparison between response of different walls.

6. A uniform distribution of #3 horizontal reinforcing bars at a spacing of approximately 48 in. was selected for all walls (Fig. 2-3).

7. A vertical ledger load of 300 lb/ft is applied to the wall at the diaphragm level. This load has an eccentricity of 3-1/2 in. from the face of the wall. The total eccentricity is thus 3-1/2 in. plus half of the thickness of the wall.

8. The wall panel has pinned end conditions as illustrated in Appendix A.

2.3 SEISMIC INPUT MOTIONS

The selected seismic input motions are intended to simulate the kinematic environment imposed by the building response at the base and the top of the walls as they are shaken in the out-of-plane direction. The motion at the wall base represents the ground motion and the motion at the top represents a compatible roof or floor diaphragm response.

The kinematic motions have been defined by displacement time histories that have been obtained from actual earthquake records or from numerical simulations obtained using actual earthquake ground motion records and typical masonry building characteristics. The kinematic motions have been scaled to represent the full range of seismicity in the United States and include seismic hazard zones of 0.1, 0.2, and 0.4g's.

Table 2-2 lists the first set of earthquake motions used in testing and indicates the testing sequence that was followed, where each wall was subjected to a series of six tests of increasing intensity. Subsequently, additional simulated earthquake motions were also used during the testing as will be discussed in Section 5.2. At this point, it is sufficient to point out that each specimen was eventually subjected to more than ten excitations using the additional simulated seismic motions.

TABLE 2-1. CONCRETE MASONRY WALL TEST MATRIX – DYNAMIC OUT-OF-PLANE

Wall No.	Casting Date	Testing Date	Wall Thickness, in.		H/T Ratio	Lap Splice	Vert. Reinf.		Grouting	
			Nominal	Actual			Minimum	High	Partial	Full
1	9/4/87	3/29/88 to 4/29/88	6	5-5/8	43	No		2-#5		X
2	2/24/88	5/22/88 to 6/24/88	4.5	4-1/2	53	No		2-#4		X
3	9/4/87	7/13/88 to 8/12/88	6	5-5/8	43	No		2-#5	X	
4	5/2/88 5/4/88	8/26/88 to 8/30/88	6	5-5/8	43	Yes		2-#5	X	

Concrete masonry wall width 39.5 in.

Vertical ledger load 300 lb/ft

Designated ultimate compressive strength of unit, 2500 psi

Horizontal steel 6-#3 in each wall (approximately 48 in. spacing)

TABLE 2-2. SEISMIC INPUT MOTIONS AND TESTING SEQUENCE

Test Sequence	Seismic Zone, g's	Top Diaphragm Response	Earthquake Record
1	0.1	Flexible	Hollister-Glorietta Warehouse Morgan Hill, 1984
2	0.1	Stiff	Saratoga-W. Valley College Gym Morgan Hill, 1984
3	0.2	Flexible	El Centro, 1940, S00E
4	0.2	Stiff	Castaic, 1971, N69W
5	0.4	Flexible	El Centro, 1940, S00E
6	0.4	Stiff	Castaic, 1971, N69W

FIGURE 2-1. THREE-STORY HIGH SLENDER REINFORCED CONCRETE MASONRY PANELS. ONE PANEL IS BEING LIFTED BY A CRANE FOR PLACEMENT ON THE SHAKING TABLE

FIGURE 2-2. PLACEMENT OF VERTICAL REINFORCING BARS IN THE CENTER OF THE SECOND CELL FROM THE END OF THE PANEL

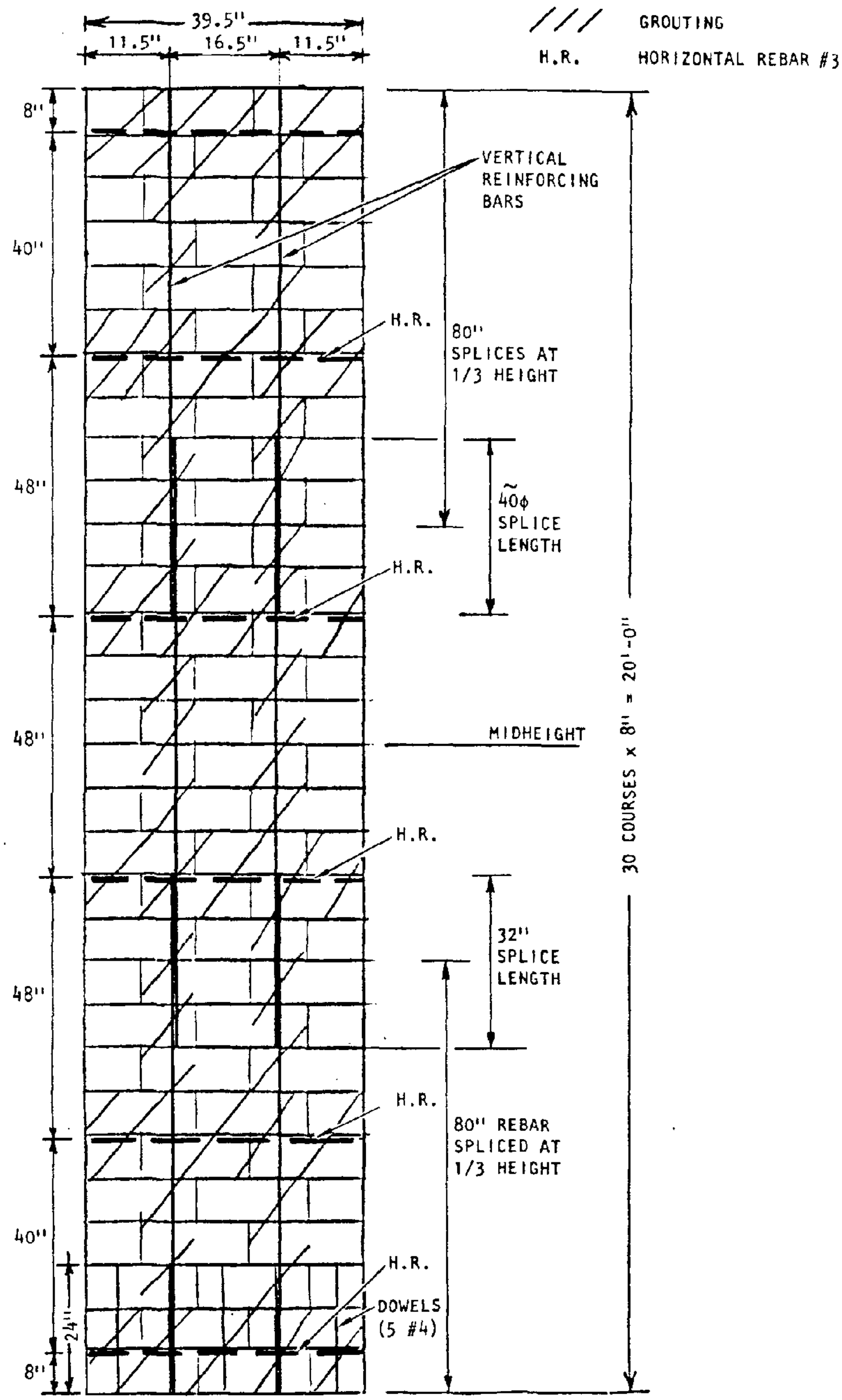


FIGURE 2-3. HORIZONTAL REBAR LAYOUT IN A TYPICAL WALL PANEL. THE PANEL DEPICTED HERE IS SPECIMEN NO. 4 WITH LAP SPLICES

SECTION 3

MATERIALS

3.1 GENERAL

The materials used in the construction of the wall panels are commercially available and are typical of those commonly used in building construction in the United States. Specific materials were selected based on the compatibility with the current national experimental program conducted by the Technical Coordination Committee on Masonry Building Research (TCCMAR) as well as compatibility with reinforced masonry wall construction in seismic areas.

3.2 CONCRETE MASONRY UNITS

The masonry units used in this program complied with ASTM Standard C90-75 (ASTM, 1984, Sec. 4) Grade N block with a net compressive value of 2000 psi minimum, manufactured by Blocklite, California, (Fig. 3-1.). Two types of 6 in. nominal hollow two-core masonry blocks were used. The full double corner block has nominal dimensions of 6 in. by 8 in. x 16 in., and the half sash block 6 in. by 8 in. by 8 in. Dimensions and block properties, averaged for three specimens, are shown in Figure 3-2 and in Table 3-1.

Another type of concrete masonry block was manufactured, as a special order, by Blocklite, California (Fig. 3-3). Both full and one-half block units were provided with a width of 4-1/2". This block was used for construction of Wall #2 and its nominal dimensions are shown in Table 3-1.

Strength properties of walls were obtained according to ASTM C 140-75 (ASTM 1984, Sec. 4). Mortar cylinders, grout prisms, grout cores, and concrete block prisms were taken for each wall (Fig. 3-4) and results are given in Tables 3-2 through 3-11.

Half-block prisms were salvaged from Wall #1 after completion of testing. Grout cores were taken out of these samples and prism strength was also obtained. The actual density of tested walls was checked by taking samples of these walls (Fig. 3-5).

3.3 REINFORCEMENT

Vertical steel reinforcement consisted of Grade 60 No. 4 and No. 5 bars conforming to ASTM A615-849 (ASTM 1984, Sec. 1). Horizontal steel reinforcement consisted of Grade 60 No. 3 bars conforming to the same specifications. Tension properties of vertical reinforcing steel are given in Table 3-12.

TABLE 3-1. COMPRESSION TESTS ON CONCRETE MASONRY UNITS

Number	Date Tested	Nominal Size of Specimen, in.	Total Load, lb	Gross Area, sq in.	Net Area, sq in.	Gross, psi	Net, psi
1	11-10-87	4-1/2 x 8 x 16*	77,800	70.31	32.67	1,110	2,380
2	11-10-87	4-1/2 x 8 x 16	79,000	70.31	32.67	1,120	2,420
3	11-10-87	4-1/2 x 8 x 16	79,300	70.31	32.67	1,130	2,430
Used in Wall #2				Average		1,120	2,410
1	11-10-87	6 x 8 x 16†	135,000	87.89	38.55	1,540	3,500
2	11-10-87	6 x 8 x 16	123,500	87.89	38.55	1,410	3,200
3	11-10-87	6 x 8 x 16	120,200	87.89	38.55	1,370	3,120
Used in Walls # 1, 3, and 4				Average		1,440	3,270
4	12-3-87	6 x 8 x 16	169,500	87.89	45.37	1,930	3,740
5	12-3-87	6 x 8 x 16	173,000	87.89	45.37	1,970	3,810
6	12-3-87	6 x 8 x 16	172,500	87.89	45.37	1,960	3,800
Used in Walls # 1, 3, and 4				Average		1,950	3,780

*Size 4-1/2 x 7-5/8 x 15-5/8

†Size 5-5/8 x 7-5/8 x 15-5/8

TABLE 3-2. COMPRESSION TEST OF MORTAR CYLINDERS

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lb	Test Value, psi	Area sq. in. in./in.	Wall No.
1	9/3/87	9/10/87	7	Cylinder	2 x 4	10,900	3,470	3.14	1&3
2	9/3/87	10/1/87	28	Cylinder	2 x 4	16,100	5,130	3.14	1&3
3	9/3/87	10/1/87	28	Cylinder	2 x 4	15,800	5,030	3.14	1&3
Specimens 2 and 3 Average 28 Days							5,080		
4	9/2/87	9/30/87	28	Cylinder	2 x 4	15,900	5,060	3.14	1&3
5	9/4/87	9/11/87	7	Cylinder	2 x 4	8,500	2,710	3.14	1&3
6	9/4/87	10/2/87	28	Cylinder	2 x 4	14,400	4,590	3.14	1&3
7	9/4/87	10/2/87	28	Cylinder	2 x 4	15,100	4,810	3.14	1&3
Specimens 6 and 7 Average 28 Days							4,700		

Mortar Mix: 18 shovels sand
2 shovels lime
1 sack cement

TABLE 3-3. COMPRESSION TEST OF GROUT PRISMS AND GROUT CORES

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lbs	Test Value, psi	Square Area. in. in.	Wall No.	Corrected* psi
1	9/4/87	9/11/87	7	Prism	3 x 3 x 6	40,300	4,480	9	1&3	N/A
2	9/4/87	10/2/87	28	Prism	3 x 3 x 6	48,100	5,340	9	1&3	N/A
3	9/4/87	10/2/87	28	Prism	3 x 3 x 6	47,500	5,280	9	1&3	N/A
					Average 28 days		5,310			
1	9/4/87	5/26/88	235	Cylinder Core †	2.77 x 5.16 H/D = 1.83 Corr Fact = 0.99	20,400	3,380	6.03	1&3	3,350
2	9/4/87	5/26/88	235	Cylinder Core	2.77 x 5.77 H/D = 2.08 Corr Fact = 1.00	21,800	3,620	6.03	1&3	3,620
3	9/4/87	5/26/88	235	Cylinder Core	2.77 x 5.61 H/D = 2.03 Corr Fact = 1.00	24,800	4,110	6.03	1&3	4,110
								Average		3,693

Test: Dry

***Corrected for H/D ratio**

†Samples were saw cut from wall on 5-18-88 and then cored from the grouted cells.

Grout Mix: 20 shovels sand
15 shovels pea gravel
1 sack cement
18 oz additive

TABLE 3-4. COMPRESSION TESTS ON BLOCK PRISM WALLS 1 AND 3

Prism No.	Date Cast	Date Tested	Age Days	Dimension, in.	Height, in.	Gross Area, sq in.	Net Area, sq in.	Max. Load, lbs	Compr. Stress, psi	H/D	Corr. Factor	F'm
1	9/4/87	10/2/87	28	15-1/2 x 5-5/8	24-1/8	87.19	87.19	167,400	1,920	4.2g	1.32	2,530
2	9/4/87	10/2/87	28	15-1/2 x 5-5/8	24-1/8	87.19	87.19	148,300	1,700	4.2g	1.32	2,240
3	9/4/87	10/2/87	28	15-1/2 x 5-5/8	24-1/8	87.19	87.19	148,900	1,710	4.2g	1.32	2,260
4	9/4/87	10/2/87	28	15-1/2 x 5-5/8	24-1/8	87.19	87.19	131,400	1,510	4.2g	1.32	1,990
5	9/4/87	10/2/87	28	15-1/2 x 5-5/8	24-1/8	87.19	87.19	130,200	1,490	4.2g	1.32	1,970
6	9/4/87	10/2/87	28	15-1/2 x 5-5/8	24-1/8	87.19	87.19	123,500	1,420	4.2g	1.32	1,870

TABLE 3-5. COMPRESSION TESTS ON BLOCK PRISMS

Prism No.	Date Cast	Date Tested	Age Days	Dimension, in.	Height, in.	Gross Area, sq in.	Net Area, sq in.	Max. Load, lbs	Compr. Stress, psi	H/D	Wall No.
1	9/4/87	5/26/88	235	5.625 x 8.0	7.75	45	45	149,750	3,330	1.38	1
2	9/4/87	5/26/88	235	5.625 x 8.0	7.75	45	45	151,500	3,370	1.38	
3	9/4/87	5/26/88	235	5.625 x 8.0	7.75	45	45	144,500	3,210	1.38	

Samples taken from 1/3 height of Wall #1 by saw cutting outside half of block wall.

TABLE 3-6. COMPRESSION TEST OF MORTAR CYLINDERS

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lb	Test Value, psi	Area sq. in. in./in.	Wall No.
1	2/25/88	3/3/88	7	Cylinder	2 x 4	13,400	4,270	3.14	2
2	2/24/88	3/23/88	28	Cylinder	2 x 4	16,000	5,100	3.14	
3	2/24/88	3/23/88	28	Cylinder	2 x 4	15,400	4,900	3.14	
					Average 28 Days		5,000		
4	2/24/88	5/26/88	93	Cylinder	2 x 4	19,700	6,270	3.14	

Mortar Mix: 18 shovels sand
2 shovels lime
1 sack cement

TABLE 3-7. COMPRESSION TEST OF GROUT PRISMS

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lb	Test Value, psi	Area sq. in. in./in.	Wall No.
1	2/24/88	3/2/88	7	Prism	3 x 3 x 6	27,600	3,070	9.00	2
2	2/25/88	3/24/88	28	Prism	3 x 3 x 6	21,300	2,370	9.00	
3	2/25/88	3/24/88	28	Prism	3 x 3 x 6	22,200	2,470	9.00	
					Average 28 Days		2,420		
4	2/24/88	5/26/88	93	Prism	3 x 3 x 6	45,800	5,090	9.00	

Grout Mix: 20 shovels sand
15 shovels pea gravel
1 sack cement
18 oz. admixture

TABLE 3-8. COMPRESSION TESTS ON BLOCK PRISMS

Prism No.	Date Cast	Date Tested	Age Days	Dimension, in.	Height, in.	Gross Area, sq in.	Net Area, sq in.	Max. Load, lbs	Compr. Stress, psi	H/D	Corr. Factor	F'm	Wall No.
1	2/25/88	3/24/88	28	7.5 x 4.5 (Half Block)	24	33.75	33.75	91,500	2,710	5.3	1.37	3,710	2
2	2/25/88	3/24/88	28	7.5 x 4.5 (Half Block)	24	33.75	33.75	88,000	2,610	5.3	1.37	3,580	
3	2/25/88	3/24/88	28	7.5 x 4.5 (Half Block)	24	33.75	33.75	89,750	2,660	5.3	1.37	3,640	
								Half Block Average 28 Days				3,640	
1	2/25/88	3/24/88	28	15.5 x 4.5 (Full Block)	23.75	69.75	69.75	140,400	2,010	5.2	1.37	2,750	
2	2/25/88	3/24/88	28	15.5 x 4.5 (Full Block)	23.75	69.75	69.75	157,200	2,250	5.2	1.37	3,080	
								Full Block Average 28 Days				2,920	

TABLE 3-9. COMPRESSION TEST OF MORTAR CYLINDERS

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lb	Test Value, psi	Area sq. in. in./in.	Wall No.
1	5/3/88	5/17/88	14	Cylinder	2 x 4	10,200	3,250	3.14	4
2									
3									

Mortar Mix: 18 shovels sand
2 shovels lime
1 sack cement

TABLE 3-10a. COMPRESSION TEST OF GROUT PRISMS

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lb	Test Value, psi	Area sq. in. in./in.	Wall No.
1	5/3/88	5/17/88	14	Prism	3 x 3 x 6	38,000	4,220	9.00	4
2	5/3/88	5/31/88	28	Prism	3 x 3 x 6	38,000	4,220	9.00	
3	5/3/88	5/31/88	28	Prism	3 x 3 x 6	38,000	4,220	9.00	

Grout Mix: 9 shovels sand
6 shovels pea gravel
1/2 sack cement

TABLE 3-10b. COMPRESSION TEST OF GROUT CORE CYLINDERS

Specimen No.	Date Cast	Date Tested	Age Days	Specimen Type	Specimen Size, in.	Total Load, lb	Test Value, psi	Area sq. in. in./in.	Wall No.
1	5/3/88	5/17/88	14	Core Cylinder	2.75 x 5.4 H/D = 1.96 Corr F. = 1.0	17,800	3,000	5.94	4
2	5/3/88	5/31/88	28	Core Cylinder	2.77 x 5.38 H/D = 1.94 Corr F. = 1.0	22,000	3,650	6.03	
3	5/3/88	5/31/88	28	Core Cylinder	2.77 x 5.39 H/D = 1.95 Corr F. = 1.00	22,800	3,780	6.03	

Note: Cores obtained from grouted block cell.

TABLE 3-11a. COMPRESSION TESTS ON BLOCK PRISMS

Prism No.	Date Cast	Date Tested	Age Days	Dimension, in.	Height, in.	Gross Area, sq in.	Net Area, sq in.	Max. Load, lbs	Compr. Stress, psi	H/D	Corr. Factor	F'm	Wall No.
1	5/3/88	5/17/88	14	5.625 x 15.5 (Full Block)	24 (3 Course)	87.19	87.19	177,600	2,040	4.27	1.32	2,690	4
2	5/3/88	5/31/88	28	5.625 x 15.5 (Full Block)	24 (3 Course)	87.19	87.19	221,000	2,530	4.27	1.32	3,340	
3	5/3/88	5/31/88	28	5.625 x 15.5 (Full Block)	24 (3 Course)	87.19	87.19	239,500	2,750	4.27	1.32	3,630	
									Average 28 Days			3,490	
1	5/3/88	5/17/88	14	5.625 x 15.5 (Full Block)	16 (2 Course)	87.19	87.19	241,000	2,760	2.84	1.17	3,230	
2	5/3/88	5/31/88	28	5.625 x 15.5 (Full Block)	16 (2 Course)	87.19	87.19	247,500	2,840	2.84	1.17	3,320	
3	5/3/88	5/31/88	28	5.625 x 15.5 (Full Block)	16 (2 Course)	87.19	87.19	256,000	2,940	2.84	1.17	3,440	
									Average 28 Days			3,380	

TABLE 3-11b. COMPRESSION TESTS ON BLOCK PRISMS

Prism No.	Date Cast	Date Tested	Age Days	Dimension, in.	Height, in.	Gross Area, sq in.	Net Area, sq in.	Max. Load, lbs	Compr. Stress, psi	H/D	Corr. Factor	F'm	Wall No.
1	5/3/88	5/17/88	14	5.625 x 7.5 (1/2 Block)	24 (3 Course)	42.19	42.19	105,800	2,510	4.27	1.32	3,310	4
2	5/3/88	5/31/88	28	5.625 x 7.5 (1/2 Block)	24 (3 Course)	42.19	42.19	107,000	2,540	4.27	1.32	3,350	
3	5/3/88	5/31/88	28	5.625 x 7.5 (1/2 Block)	24 (3 Course)	42.19	42.19	109,500	2,600	4.27	1.32	3,430	
									Average 28 Days			3,390	

TABLE 3-12. TENSION TESTS ON STEEL REINFORCING BARS

Rebar No.	Date Tested	ASTM Spec.	Size	Grade	Yield Point, psi	Ultimate Strength, psi	Elongation, in./in.	Bend Test
1	8/28/87	A615	#5	60	62,260	106,770	12	O.K.
2	8/28/87	A615	#4	60	69,000	105,000	13.5	O.K.

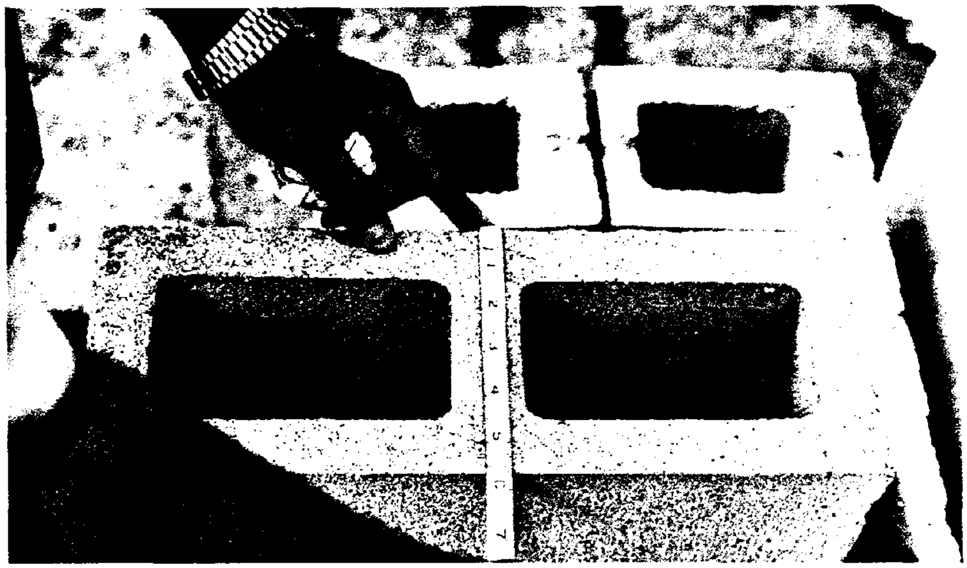
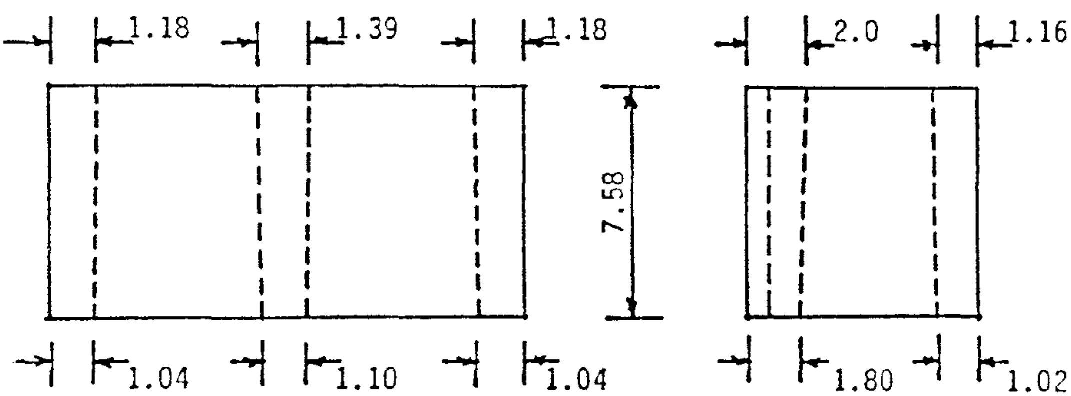
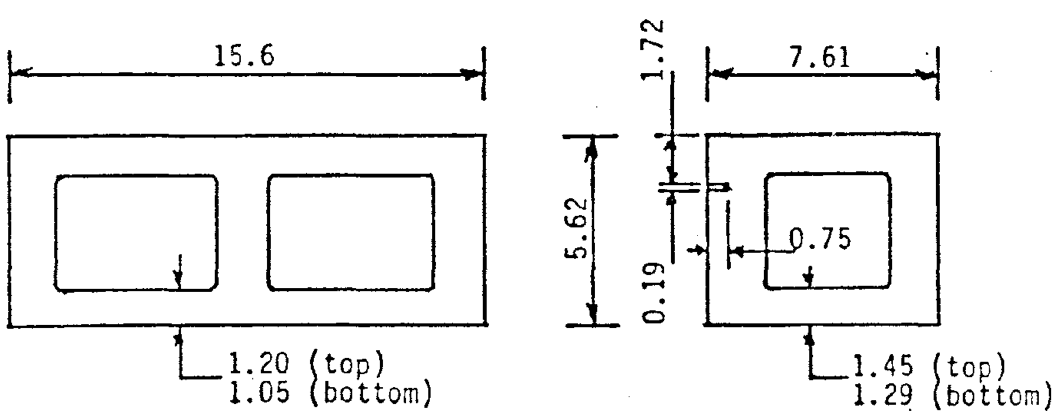


FIGURE 3-1. CONCRETE MASONRY UNITS



(a) Typical elevation



(b) Typical cross-section

FIGURE 3-2. CONFIGURATION OF CONCRETE MASONRY UNITS (NOTE: ALL DIMENSIONS ARE GIVEN IN INCHES)

FIGURE 3-3. SPECIAL CONCRETE MASONRY BLOCK 4-1/2 IN. x 16 IN. x 8 IN. FOR WALL NO. 2

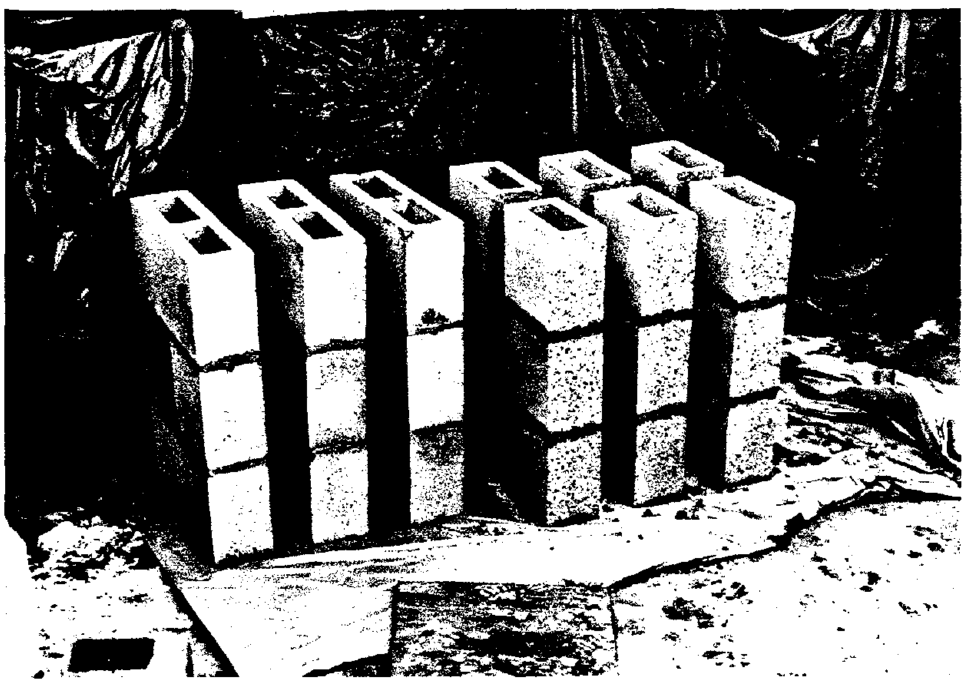


FIGURE 3-4. FULL AND HALF BLOCK MASONRY PRISMS

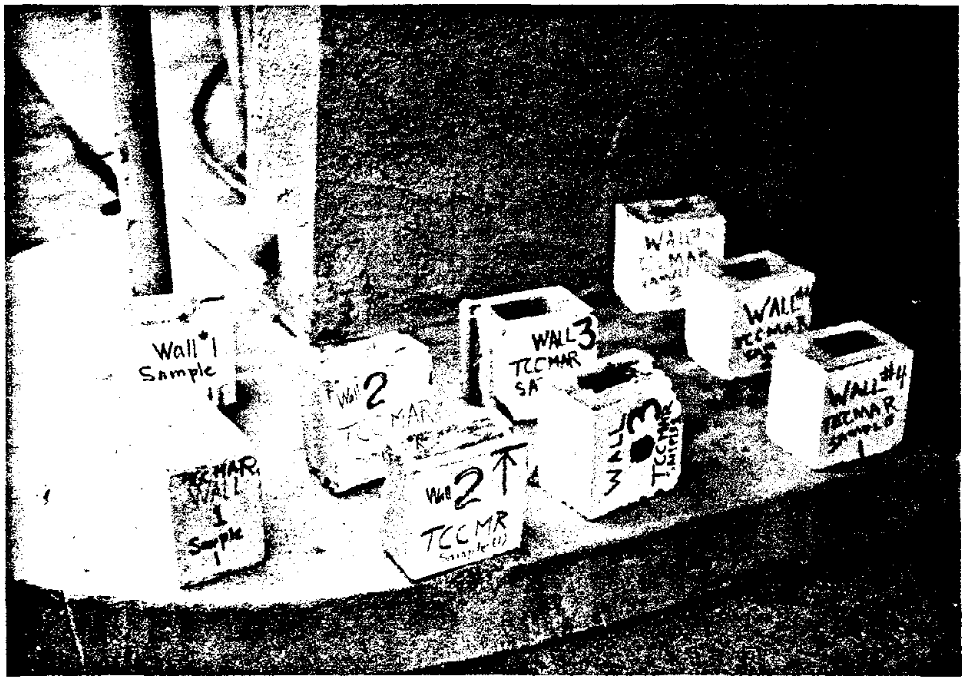


FIGURE 3-5. SAMPLES TAKEN FROM TESTED WALLS TO CHECK ACTUAL DENSITY OF THE WALLS

SECTION 4

TEST FACILITY AND SETUP

4.1 TEST AREA

The test facility at Agbabian Associates (AA) occupies a concrete surface area of 20 ft x 25 ft adjacent to the AA office building in El Segundo, California, (Fig. 4-1). The entire concrete slab has been treated with Thompson's Water Seal, and the slab has a 1% slope to the east to facilitate spill cleaning.

A 32 ft I-beam (WF 33 x 241) is used as the strongback for the vibrators located at the base and near the top as depicted in Figure 4-2. The strongback is cantilever mounted 7 ft deep in 25 yd of reinforced concrete. The fundamental natural frequency of the strongback is above 20 Hz and its deflection at 20 ft elevation for 10 kip load is about 0.05 in. These characteristics satisfy the requirements imposed by the vibratory system on the strongback.

A plywood paneled wall encloses the two open sides of the facility as shown in Figure 4-1.

4.2 ACTUATION SYSTEM

The seismic motion is imparted onto the test specimen by high pressure hydraulic actuators under closed-loop servocontrol. The seismic time history used as the excitation command is sent via a microcomputer to a control panel as a digitally generated analog signal that replicates the actual earthquake displacements. The control panel drives the two hydraulic actuators as independent units. The entire system is essentially the same as the one used in an earlier NSF-funded study of out-of-plane bending of tilt-up concrete walls (Adham, 1987). Various components of the actuation system will be discussed briefly in the following subsections.

4.2.1 TEST CONTROL COMPUTER

The conduct of the testing sequence was controlled by a program on a Digital Equipment Corporation PDP 11/03 computer. The program incorporates safety features which allow for safely stopping the excitation if an anomaly is detected in the system. The seismic excitation signal delivered to the servocontroller is stored in the computer memory and, upon starting, sends the signal to the controller through a 12-bit Digital-to-Analog converter attached to the computer.

4.2.2 HYDRAULIC PRESSURE SYSTEM

The hydraulic power supply consists of a Denison pump which at 1750 RPM and 3000 psi delivers approximately 5 gpm of hydraulic oil to the system. The total fluid flow requirement for each cylinder during a 30-sec excitation is about 6.8 gal with a singular peak flow requirement of 70 gpm.

A 12-gal accumulator is provided to meet the peak flow requirement; one for each actuator. These bladder-type accumulators are charged to a preset pressure of 1500 psi with dry nitrogen. Only when the total flow from both cylinders exceeds the pump capacity does flow occur from the accumulators.

4.2.3 HYDRAULIC ACTUATORS

Electrohydraulic actuators at the base and at the top of the wall provide the seismic excitation to the test specimen. The actuators consist of a cylinder and a piston. The center part of the cylinder has two receiving ports which make the piston move back and forth depending upon the pressurized port. Both actuators have a performance capability of 20,000 lb force output, 16 in. peak-to-peak stroke, a peak velocity of 40 in./sec, and a waveform fidelity up to 10 Hz.

4.2.4 SERVOVALVES

The servovalves used in this system are mounted directly on the actuators and have a pilot valve/slave valve arrangement. The pilot is a voice coil driven valve with its spool forced fitted into the voice coil. The pilot spool is supported by springs at both ends to return it to a neutral position. The hydraulic signal from the pilot valve controls the spool of the slave valve which opens or shuts the main flow passage. The spool and sleeve of the slave valve are similar to the pilot valve, but the spool is not supported by a spring. To close the loop on the hydraulic system a LVDT position sensor is used for displacement feedback.

4.3 MOTION SENSORS

Four types of sensors were available to be used in this program: displacement gages, velocity gages, strain gages, and load cells. Figure 4-3 shows the instrumentation layout on a typical test specimen. Due to the limitation on the number of available recording channels of the FM recorder, the signals from the load cells were not recorded and the instrumentation layout excludes these two load cells.

4.3.1 DISPLACEMENT GAGES

The displacement gages were Celesco DV1 500-ohm string potentiometers, or "pots," with a 30-in. span. Precision resistors were used to form a wheatstone bridge with the pot, the pot forming two active legs of the circuit. The bridge was excited by a DC power supply equipped with a circuit to provide constant current to the bridge.

4.3.2 VELOCITY GAGES

Each displacement gage includes a velocity sensor in the same instrument. These are low inertia tachometers that generate a voltage as a function of the rate of coiling and uncoiling of the string of the string pot. At the peak velocity of 40 in./sec the unit provides 5.28 volts. Inherently self-generating the velocity gages did not need an excitation supply.

4.3.3 LOAD CELLS

The load cells were 25 kip low profile units manufactured by Interfacxe, Inc. The load cells were part of the assembly that connected the actuators to the strongback. As mentioned earlier, due to unavailability of extra recording channels the signal from these cells were not recorded.

4.3.4 STRAIN GAGES

The strain gages used for determining the deformations in the vertical reinforcements are EATON SG129 weldable gages which are directly welded to the rebar. The rated strain level is ±20,000 microinches per inch or 2% strain level. The strain gage with three conductor cables forms one active leg of the gage circuit with the signal conditioner completing the bridge. A 9 volt DC input serves as the excitation to the bridge.

4.4 DATA ACQUISITION SYSTEM

The analog signals generated in the sensors were recorded for future digitization via the data acquisition system. This system consists of frequency modulation (FM) recorders, amplifiers, and anti-aliasing filters. The data acquisition system used in this program is shown in Figures 4-4 and 4-5.

The analog signals were then digitized at the facilities of the University of Southern California and the digitized data was stored on a VAX 11/750 minicomputer.

4.4.1 MAGNETIC TAPE RECORDERS

The frequency modulation (FM) recorder used to record the analog signal was a Honeywell model 7600 magnetic tape recoder/ reproducer. The tape speed was set at 30 in./sec. The FM recorder had 14 channels from which one was reserved for the time code.

Another FM recorder (AMPEX model SP 300) was used to record the two additional channels; for a total of 15 channels. The AMPEX recorder speed was set at 3.75 in./sec, and it was triggered simultaneously with the Honeywell recorder through a special hookup.

4.4.2 AMPLIFIER

The signals from the strain gages and the displacement potentiometer were amplified using Teledyne Philbrick 1701 chopper stabilized amplifiers with the gain set at 100. The velocity signal did not need any amplification.

4.4.3 FILTERS

The driving signal from the test control computer was passed through an anti-aliasing filter (Precision Filters, System 616) with the cutoff frequency set at 10 Hz. The filtering was done to prevent any erroneous signal from end caping the actuators. The signal from the motion sensors were not filtered. If needed, the filtering could be done digitally.

4.4.4 SIGNAL DIGITIZATION

The digitization process of the analog signals were performed at the University of Southern California. A 12-bit A/D converter was used with the sampling rate set at 1000 SPS. The digitized data was stored for further processing on a VAX 11/750 minicomputer.

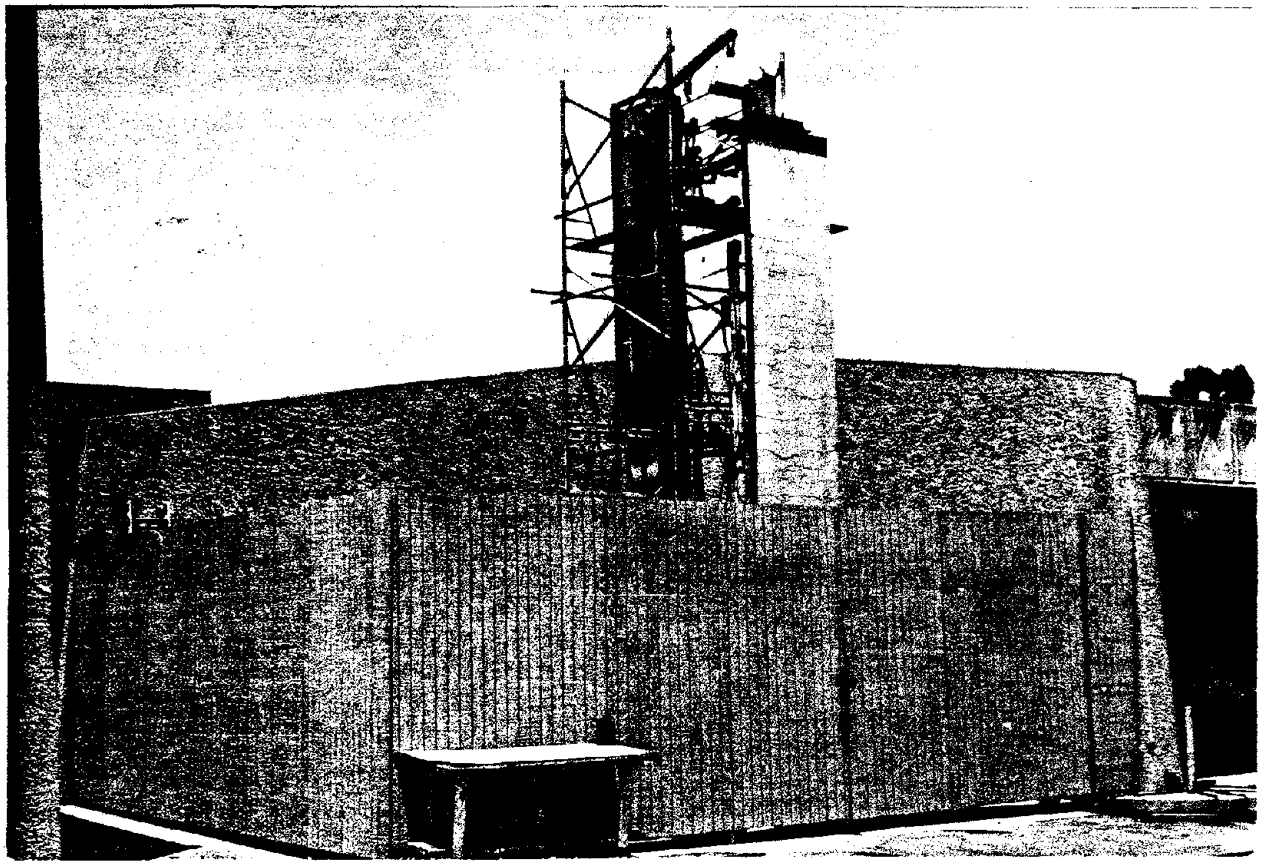


FIGURE 4-1. TEST FACILITY

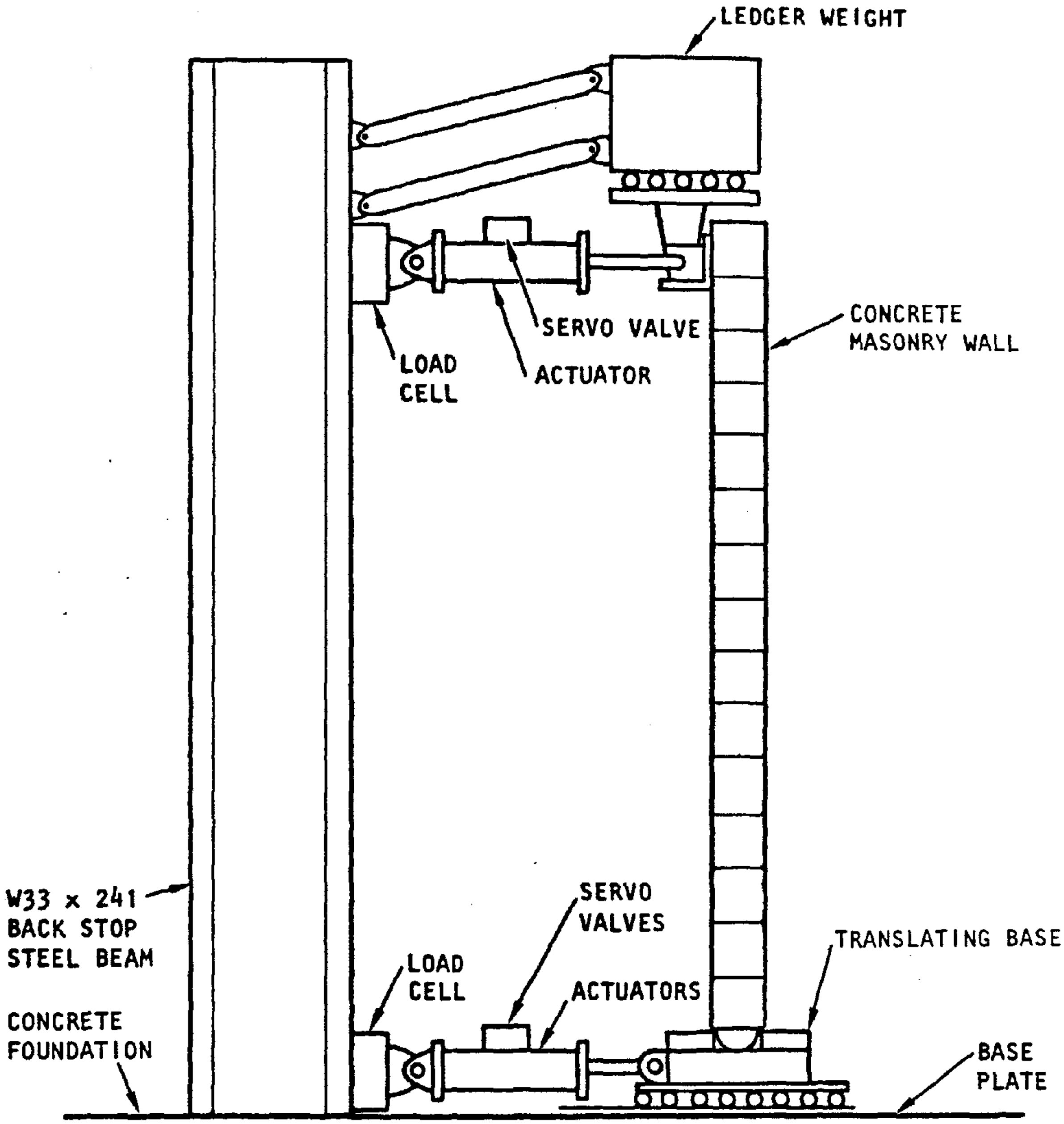


FIGURE 4-2. TEST SETUP FOR DYNAMIC TESTING OF WALLS

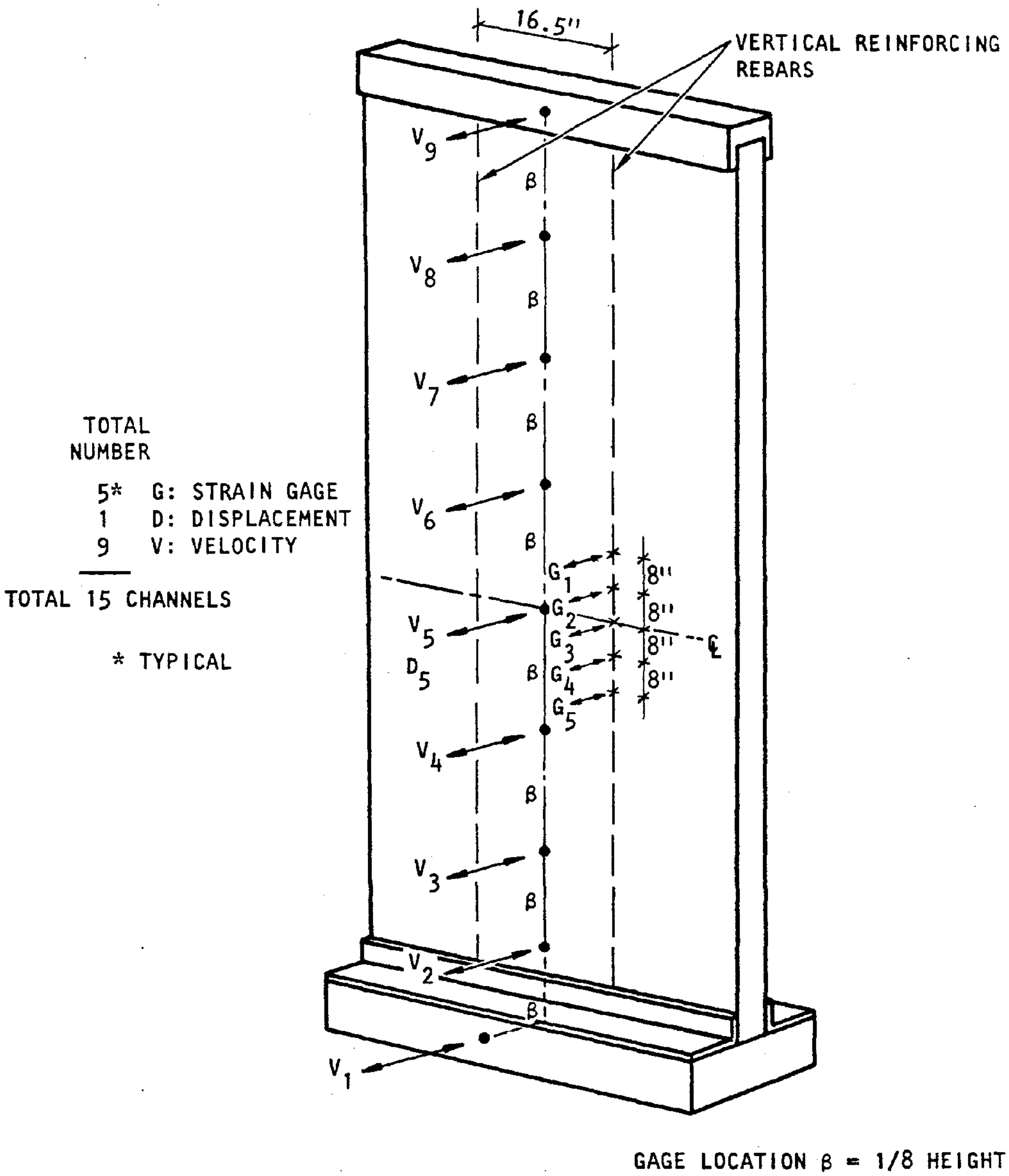


FIGURE 4-3. TYPICAL WALL INSTRUMENTATION FOR DYNAMIC TESTING

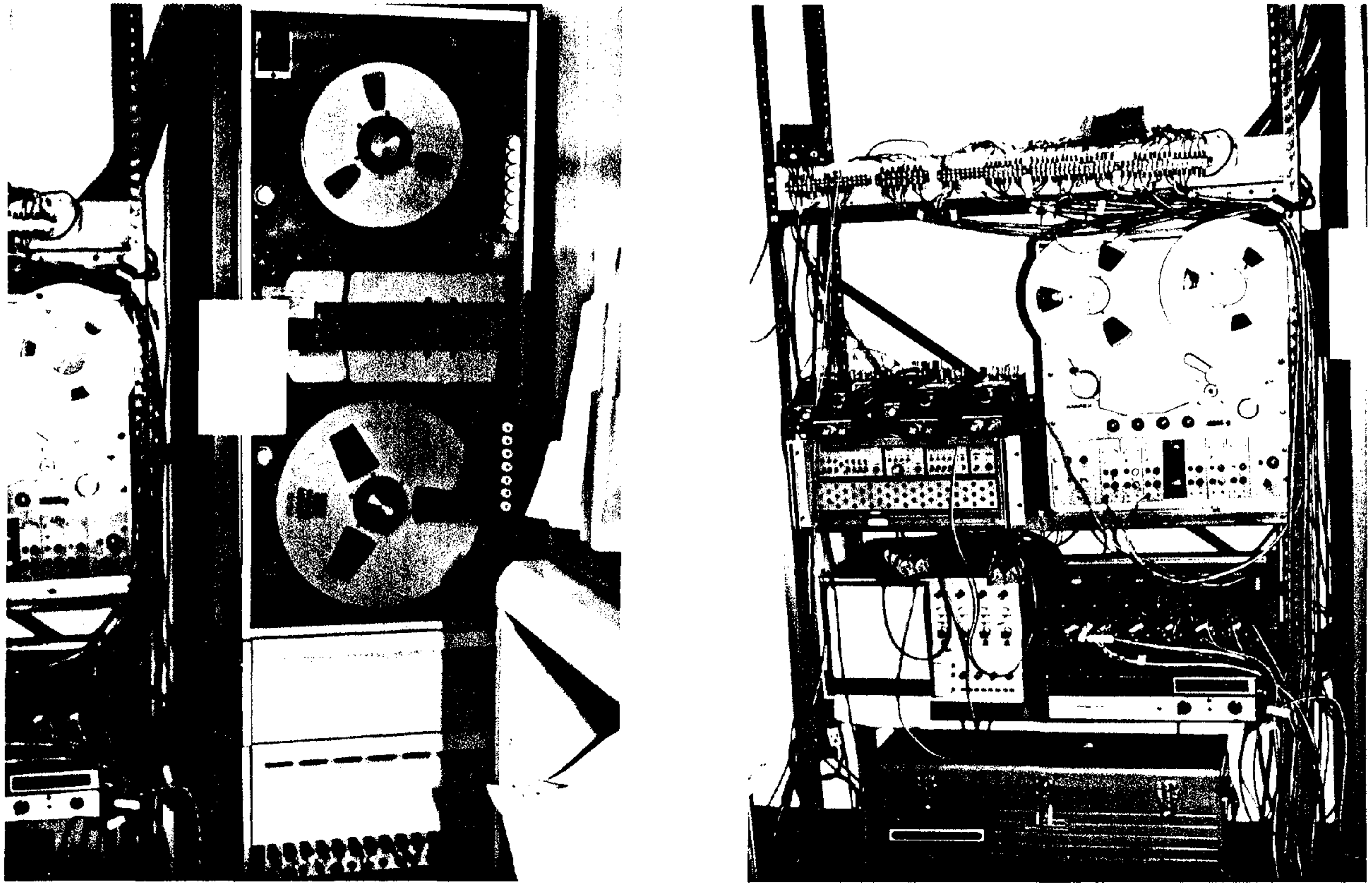


FIGURE 4-4. DATA ACQUISITION SYSTEM

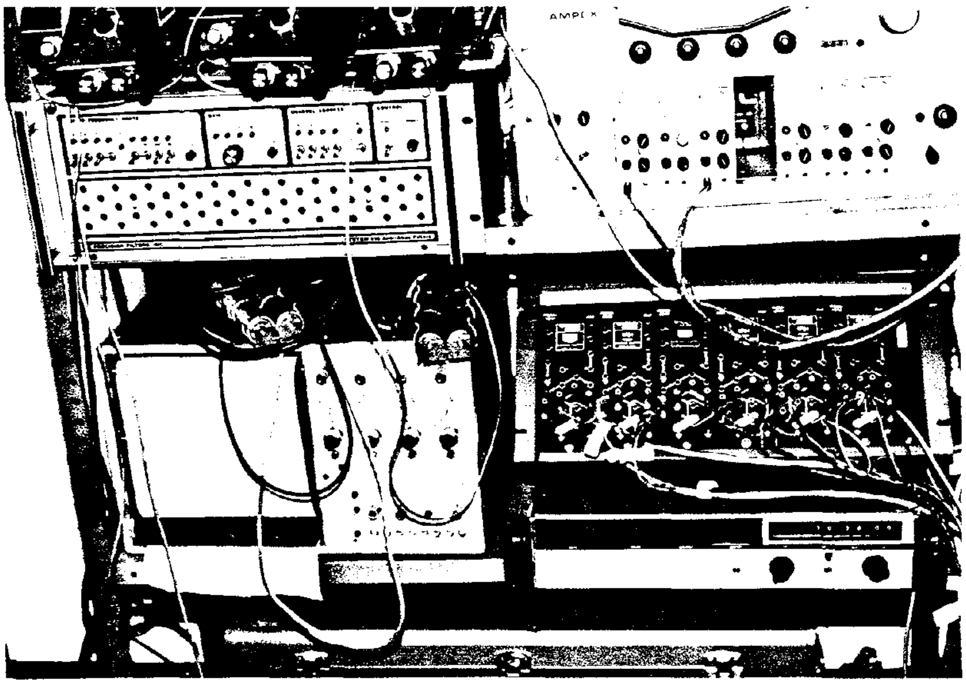


FIGURE 4-5. DATA ACQUISITION SYSTEM

SECTION 5

TEST RESULTS

5.1 INTRODUCTION

Fifteen instrumentation channels recorded the test results along the height of the test panel. There were nine velocity gages that were equally spaced along the height of the panel. One displacement gage was placed at midheight of the panel. Five strain gages were spot welded to one of the vertical rebars. They were located, one at center of panel and the other four were directly below and above the center at an 8-in. spacing. A typical wall instrumentation layout is shown in Figure 4-3. All four masonry wall panels were tested with the general loading sequence described in the following. The natural frequencies of the wall panels were determined analytically and are tabulated in Appendix B.

The testing of each wall was observed at various stages by representatives from several structural engineering firms including Agbabian Associates, Englekirk and Hart, and Kariotis Associates; University of Southern California; delegation from New Zealand; Masonry Institute of America; masonry contractors; City of Los Angeles; and by Raymond Bentson, consultant with the Los Angeles County Building and Safety Department (Fig. 5-1).

5.2 LOADING SEQUENCE

All the panels were first subjected to the required seismic motion described in Table 2-2. At the end of the 6th sequence (0.4 EPA) all the specimens remained elastic and there was no sign of visible distress. In order to be able to force these panels into inelastic range, additional and more intense seismic shaking was applied to these specimens. The additional sequence of seismic input included four seismic motions which are:

- Motion M7, El Centro 1940 NS scaled to 0.4g effective peak ground acceleration with a 30 sec shaking duration; rigid diaphragm input at top.
- Motion M8, which is the same as the motion described above, except that the duration of shaking was compressed to 15 sec. The effect of this signal compaction is to shift some of the energy of the signal to higher frequencies which are closer to the natural frequency of the wall panel. For comparison, the Fourier displacement amplitude spectrum for the 30-sec signal and the 15-sec signal are shown in Figures 5-2 and 5-3, respectively. The peak ground acceleration associated with the compacted motion is more than 1.0g, however it should be emphasized that this PGA is associated with a sharp spike and the total energy of the compacted displacement record is still the same as that of displacement record of Motion M7.
- Motion M9, which is the same motion as above except its duration is compressed into 12.5 sec.
- Motion M10, which is a version of Bonds Corner 1979 record, scaled in frequency domain to match ATC-3 S1 spectrum with effective peak ground acceleration scaled to 0.8g and a flexible diaphragm input at top.

Each panel was subjected to a combination of the basic seismic input sequences in addition to Motions M7 through M10.

5.3 RESULTS FOR PANEL #1

Panel #1 was cast on September 4, 1987 with 5-5/8" thick concrete masonry blocks. Uncracked natural frequency of the panel was calculated to be about 5.0 Hz. The panel had an H/t ratio of 43 and was fully grouted with 2-#5 rebars as vertical reinforcement. There was no lap splicing in the rebars and no strain gages were used in this panel. The wall was tested from March 29 until April 29, 1988.

The testing sequence and the resulting panel response are discussed below.

- The basic set of seismic input motions (Sequences 1 through 6) were applied first. The panel response was elastic, however cracks along the mortar joint developed at two courses above the midheight of the panel for Sequence 6.

- Sequences 7 through 9 consisted of Motion M7. The panel was responding elastically with the observed cracks along the mortar joints near the panel midheight opening and closing.

- Sequence 10 was applied as a repeat of Sequence 2 to study the effect of a small earthquake (possibly an aftershock) after the panel was subjected to some cracking.

- Motion M8 was repeatedly imparted onto the panel (Sequences 11 through 16). The panel was still behaving elastically. Figure 5-4 shows the absolute displacement at panel midheight for Sequence 14. The solid curve is the displacement time history integrated from the measured velocity, while the dashed curve is the direct measured displacement at midheight. There is good agreement between the two curves. By the end of Sequence 16, there were cracks in the bed joints of the four courses above the midheight of the panel.

- Sequences 17 and 18 consisted of Motion M9. At Sequence 17 some inelastic deformations were observed and at Sequence 18 the panel was well into inelastic range. Figure 5-5 shows the relative displacement time history at the panel midheight for Sequence 18. The maximum peak-to-peak relative displacement at midheight was about 10.8 in. Based on observation after the test, the maximum permanent deformation at panel midheight was about 4-1/2 in. Figure 5-6 shows the deformed panel at the end of Sequence 18.

Digitized velocity, displacement, and some selected relative displacement time histories for Panel #1 are given in Appendix A of Volume II.

5.4 RESULTS FOR PANEL #2

Panel #2 was cast on February 24, 1988 with 4-1/2 in. thick concrete masonry blocks. Uncracked natural frequency of the panel was calculated to be 4.0 Hz. The panel had H/t ratio of 53 and was fully grouted with 2-#4 vertical reinforcing bars without splicing. The strain gage layout is as described in Section 5.1. The wall was tested from May 22 until June 24, 1988.

The testing sequence and the resulting panel response are outlined below.

- Testing started by subjecting the panel to the recommended seismic motions which constitute Sequences 1 through 6 of the test input series. The panel response was elastic and no visible sign of distress was observed except for some minor mortar joint cracks that would close at the end of each test cycle.
- Sequence 7 was a repetition of Sequence 5.
- Sequences 8, 9, and 10 were a repetition of Sequence 2, 3, and 5, respectively. The objective of this repetition was to create a data base for comparing the panel response to a series of earthquakes before and after the panel was subjected to 0.4g EPA seismic environment. By the end of Sequence 10, the panel was populated with some cracks near its midheight, with some spread towards the upper portion of the wall. These cracks were located along the mortar joints spanning the entire width of the panel. However, the panel response was still elastic and the cracks were closed at the end of each test cycle.

- Motion M8 was applied for Sequence 11 and 12. During Sequence 11 a loud cracking sound was heard and after the test was ended a permanent deformation of 1-1/8" was induced at the panel midheight. After Sequence 12, the permanent deformation at panel center was increased to 1-3/4". At this point the cracks were well visible after the tests were over.
- Sequences 13 and 14 were a repeat of Sequence 5. The panel did not go further into the inelastic range and the permanent deformation at the center of the panel was unchanged.
- Motion M8 was the last sequence (Sequence 15) for this panel. At the end of the sequence the panel was well into inelastic regime. Figure 5-7 depicts the midheight absolute displacement history, while Figure 5-8 shows the relative displacement time history at the panel midheight. The peak-to-peak relative displacement at midheight was about 17.5 in. The permanent deformation at panel center at the end of the test was about 3-3/4". Figure 5-9 shows the panel crack pattern as well as the final panel deformed shape at the end of Sequence 15.

Digitized velocity, displacement, relative displacement, and rebar strain time histories for Panel #2 are given in Appendix B of Volume II.

5.5 RESULTS FOR PANEL #3

Panel #3 was cast on September 4, 1987 with 5-5/8" thick concrete masonry blocks. The uncracked natural frequency of the panel was calculated to be 5.6 Hz. The panel had a H/t ratio of 43 and was partially grouted with 2-#5 vertical reinforcing bars without lap splicing. The strain gage layout was different from what is given in Section 5.1. Here, both rebars were instrumented with one strain gage at panel midheight, and on one rebar two more gages were mounted one at 8" above and below the center of the panel for a total of four strain gages on the two bars. The panel was tested from July 7 until August 12, 1988.

The testing sequence and the induced panel response are summarized below.

- The panel was subjected to Motion Sequences 1 through 6 without showing any sign of visible distress. Minor cracking at mortar joints were observed while the test was in progress, however they did close at the end of the test cycle.
- Sequences 7 and 8 consisted of Motion M7. Panel response was elastic and basically the same as for Sequence 5.
- Sequences 9 through 15 consisted of Motion M8. In these sequences the panel was responding elastically. All the mortar joint cracks that were visible while the test was in progress would close at the end of each test. Figure 5-10 shows both the absolute and the relative midheight displacement time histories for Sequence 15. The panel midheight peak-to-peak relative displacement is 5.5". This displacement is 20% less than a comparable displacement for the fully grouted panel (specimen #1), indicating that the partially grouted panel may have a higher dynamic strength than the fully grouted panel.
- Sequence 16 was a repeat of Sequence 7.
- Sequences 17 through 25 consisted of Motion M8. The responses were similar to those obtained for Sequences 9 through 15. This similarity is depicted in Figure 5-11 where the solid curve shows the midheight relative displacement at Sequence 15 while the dashed curve shows the same quantity for Sequence 25. The repeated application of Motion M8 had not caused any apparent loss of integrity in the panel.

- The panel was subjected to Motion M10 as Sequence 26 and 27. There was considerable crack opening during the test, however these cracks were closed at the end of each test. The panel sustained no permanent deformation. It should be emphasized however, that although the cracks were closed at the end of each sequence, there was enough chipping at the mortar joints which caused the crack pattern to be recognizable.

- Sequence 28 consisted of Motion M10 with a 25% increase in amplitude. Again the panel response was elastic and had the same cracking pattern. Midheight relative displacement is shown in Figure 5-12.

- Sequence 29 and 30 consisted of Motion M9. The panel midheight was undergoing considerable relative displacement (8.1" peak-to-peak at midheight) with pronounced bias toward one direction as is depicted in Figure 5-13. The bias is consistent with the test observations. However, the panel response was still elastic.

At this point it was decided to end the testing of Panel #3 without being able to force the panel into the inelastic range.

Based on the above observations, it is reasonable to conclude that the ductility of the partially grouted wall is higher than its fully grouted counterpart.

Digitized velocity, displacement, relative displacement, and rebar strain time histories for Panel #3 are given in Appendix C of Volume II.

5.6 RESULTS FOR PANEL #4

Panel #4 was cast on May 4, 1988 with 5-5/8" thick concrete masonry blocks. The uncracked natural frequency of the panel was calculated to be 5.6 Hz. The panel's H/t ratio was 43. The panel was partially grouted with 2-#5 rebars as vertical reinforcement. The vertical reinforcing bars were spliced at 8 ft from the bottom and top of the panel. The requirement of lap splice development length given in Section 2409(e)3 of the 1988 edition of the Uniform Building Code was followed. The strain gage layout was as given in Section 5.1. The panel was tested from August 26 until August 30, 1988.

The testing sequence is identical with that used for Panel #3, the objective being the creation of a data base to study the effects of lap slicing on the panel response. A summary of the testing sequence and the induced panel response are given below.

- The panel responded to Motion Sequences 1 through 6 without any sign of distress and with only minor cracks opening during the test.
- Motion M7 was imparted onto the panel as Sequences 7 and 8 causing elastic response in the panel.
- Test Sequences 9 through 15 utilized Motion M8. In all these tests the panel was responding elastically. All the mortar joint cracks that were visible while the test was in progress did close at the end of the test. Figure 5-14 shows a comparison between the midheight velocity response for Sequence 15 of Panel #3 (no lap splicing; solid curve) and its equivalent for Panel #4 (with lap splicing; dashed curve). The responses of both panels are similar.
- Sequence 16 was a repeat of Sequence 7.
- Sequences 17 through 25 consisted of Motion M8. The corresponding responses were similar to those obtained for Sequences 9 through 15. This similarity is depicted in Figure 5-15. The solid curve shows the midheight velocity at Sequence 15 while the dashed curve shows the same quantity for Sequence 25. The repeated application of Motion M8 did not cause any loss of integrity of the panel.

- The panel was subjected to Motion M10 in Sequences 26 and 27. There was considerable crack opening during the test, however, at the end of the test all the cracks were closed and the panel did not sustain any permanent deformation.
- Sequence 28 consisted of Motion M10 with its amplitude increased by 25%. Again the panel response was elastic and had the same cracking pattern.
- Sequences 29 and 30 consisted of Motion M9. The panel midheight at Sequence 30 was undergoing a peak-to-peak relative elastic displacement of 7.9", as depicted in Figure 5-16.

By the end of Sequence 30 the panel was still elastic without any sign of permanent deformation. The cracking pattern was similar though not identical with that of Panel #3.

Based on preliminary observations made, the response of a panel with its vertical reinforcing bars spliced (Panel #3) and a panel which has no splicing in its rebars (Panel #4) was similar. In addition, the peak-to-peak relative displacement for Sequence 30 for Panel #3 is 8.1" while the same quantity for Panel #4 was 7.9".

Digitized velocity, displacement, relative displacement, and strain time histories for Panel #4 are given in Appendix D of Volume II.

5.7 SUMMARY

A total of four masonry slender wall panel specimens were tested in this effort. Each specimen was subjected to a series of seismic excitations. Table 5-1 through Table 5-4 represent a summary of the testing sequence and a summary of panel response for a given excitation, for specimens 1 through 4, respectively.

TABLE 5-1. TEST OBSERVATIONS FOR WALL PANEL #1

5-5/8" Block, 2-#5, Full Grout, No Lap Splice

Test Sequence	Earthquake Motion at Base	g's*	Duration (sec)	Panel Response
1	Hollister (M1)	0.1	30	↑
2	Saratoga (M2)	0.1	30	Elastic; hairline cracks
3	El Centro (M3)	0.2	30	
4	Castaic (M4)	0.2	30	↓
5	El Centro (M5)	0.4	30	↑
6	Castaic (M6)	0.4	30	
7	El Centro (M7)	0.4	30	Elastic; minor mortar joint cracks
8	M7	0.4	30	
9	M7	0.4	30	↓
10	M1	0.1	30	Elastic
11	El Centro (M8)	~1.0	15	↑
12	M8	~1.0	15	
13	M8	~1.0	15	Elastic; more cracks near midheight
14	M8	~1.0	15	
15	M8	~1.0	15	
16	M8	~1.0	15	↓
17	El Centro (M9)	~1.0	12.5	Inelastic
18	M9	~1.0	12.5	Inelastic; PD = 4.5" PRD = 5.6"

PD = Midheight permanent deformation
PRD = Midheight peak relative displacement

*Values indicate peak base accelerations; peak top accelerations are modified by top actuator

TABLE 5-2. TEST OBSERVATIONS FOR WALL PANEL #2

4-1/2" Block, 2-#4, Full Grout, No Lap Splice

Test Sequence	Earthquake Motion at Base	g's*	Duration (sec)	Panel Response
1	Hollister (M1)	0.1	30	
2	Saratoga (M2)	0.1	30	Elastic; minor mortar joint cracks
3	El Centro (M3)	0.2	30	
4	Castaic (M4)	0.2	30	
5	El Centro (M5)	0.4	30	
6	Castaic (M6)	0.4	30	
7	M5	0.4	30	Elastic; midheight vicinity populated with cracks
8	M2	0.1	30	
9	M3	0.2	30	
10	M5	0.4	30	
11	El Centro (M8)	~1.0	15	Inelastic; PD = 1-1/8"
12	M8	~1.0	15	Inelastic; PD = 1-3/8"
13	M5	0.4	30	Elastic; PD = 1-3/8"
14	M5	0.4	30	Elastic; PD = 1-3/8"
15	M8	~1.0	15	Inelastic; PD = 3-3/4" PRD = 10"

PD = Midheight permanent deformation
PRD = Midheight peak relative displacement

*Values indicate peak base accelerations; peak top accelerations are modified by top actuator

TABLE 5-3. TEST OBSERVATIONS FOR WALL PANEL #3

5-5/8" Block, 2-#5, Partial Grout, No Lap Splice

Test Sequence	Earthquake Motion at Base	g's*	Duration (sec)	Panel Response
1	Hollister (M1)	0.1	30	Elastic; hairline cracks
2	Saratoga (M2)	0.1	30	
3	El Centro (M3)	0.2	30	
4	Castaic (M4)	0.2	30	
5	El Centro (M5)	0.4	30	Elastic; mortar joint cracks
6	Castaic (M6)	0.4	30	
7	El Centro (M7)	0.4	30	
8	M7	0.4	30	
9	El Centro (M8)	~1.0	15	
10	M8	~1.0	15	Elastic; mortar crack opening & closing
11	M8	~1.0	15	
12	M8	~1.0	15	
13	M8	~1.0	15	
14	M8	~1.0	15	
15	M8	~1.0	15	PRD = 2.8"
16	M7	0.4	30	Elastic
17	M8	~1.0	15	
18	M8	~1.0	15	Elastic; mortar crack opening & closing
19	M8	~1.0	15	
20	M8	~1.0	15	
21	M8	~1.0	15	
22	M8	~1.0	15	
23	M8	~1.0	15	
24	M8	~1.0	15	
25	M8	~1.0	15	
26	Bonds Corner (M10)	~0.8	30	Elastic; crack pattern recognizable
27	M10	~0.8	30	
28	1.25 x M10	~1.0	30	
29	El Centro (M9)	~1.0	12.5	Elastic
30	M9	~1.0	12.5	Elastic; PRD = 5.8"

PRD = Midheight peak relative displacement

*Values indicate peak base accelerations; peak top accelerations are modified by top actuator

TABLE 5-4. TEST OBSERVATIONS FOR WALL PANEL #4

5-5/8" Block, 2-#5, Partial Grout, With Lap Splice

Test Sequence	Earthquake Motion at Base	g's*	Duration (sec)	Panel Response
1	Hollister (M1)	0.1	30	Elastic; hairline cracks
2	Saratoga (M2)	0.1	30	
3	El Centro (M3)	0.2	30	
4	Castaic (M4)	0.2	30	
5	El Centro (M5)	0.4	30	Elastic; mortar joint cracks
6	Castaic (M6)	0.4	30	
7	El Centro (M7)	0.4	30	
8	M7	0.4	30	
9	El Centro (M8)	~1.0	15	
10	M8	~1.0	15	Elastic; mortar crack opening & closing
11	M8	~1.0	15	
12	M8	~1.0	15	
13	M8	~1.0	15	
14	M8	~1.0	15	
15	M8	~1.0	15	
16	M7	0.4	30	Elastic
17	M8	~1.0	15	
18	M8	~1.0	15	Elastic; mortar crack opening & closing
19	M8	~1.0	15	
20	M8	~1.0	15	
21	M8	~1.0	15	
22	M8	~1.0	15	
23	M8	~1.0	15	
24	M8	~1.0	15	
25	M8	~1.0	15	
26	Bonds Corner (M10)	~0.8	30	Elastic; crack pattern recognizable
27	M10	~0.8	30	
28	1.25 x M10	~1.0	30	
29	El Centro (M9)	~1.0	12.5	Elastic
30	M9	~1.0	12.5	Elastic; PRD = 4.2"

PRD = Midheight peak relative displacement

*Values indicate peak base accelerations; peak top accelerations are modified by top actuator

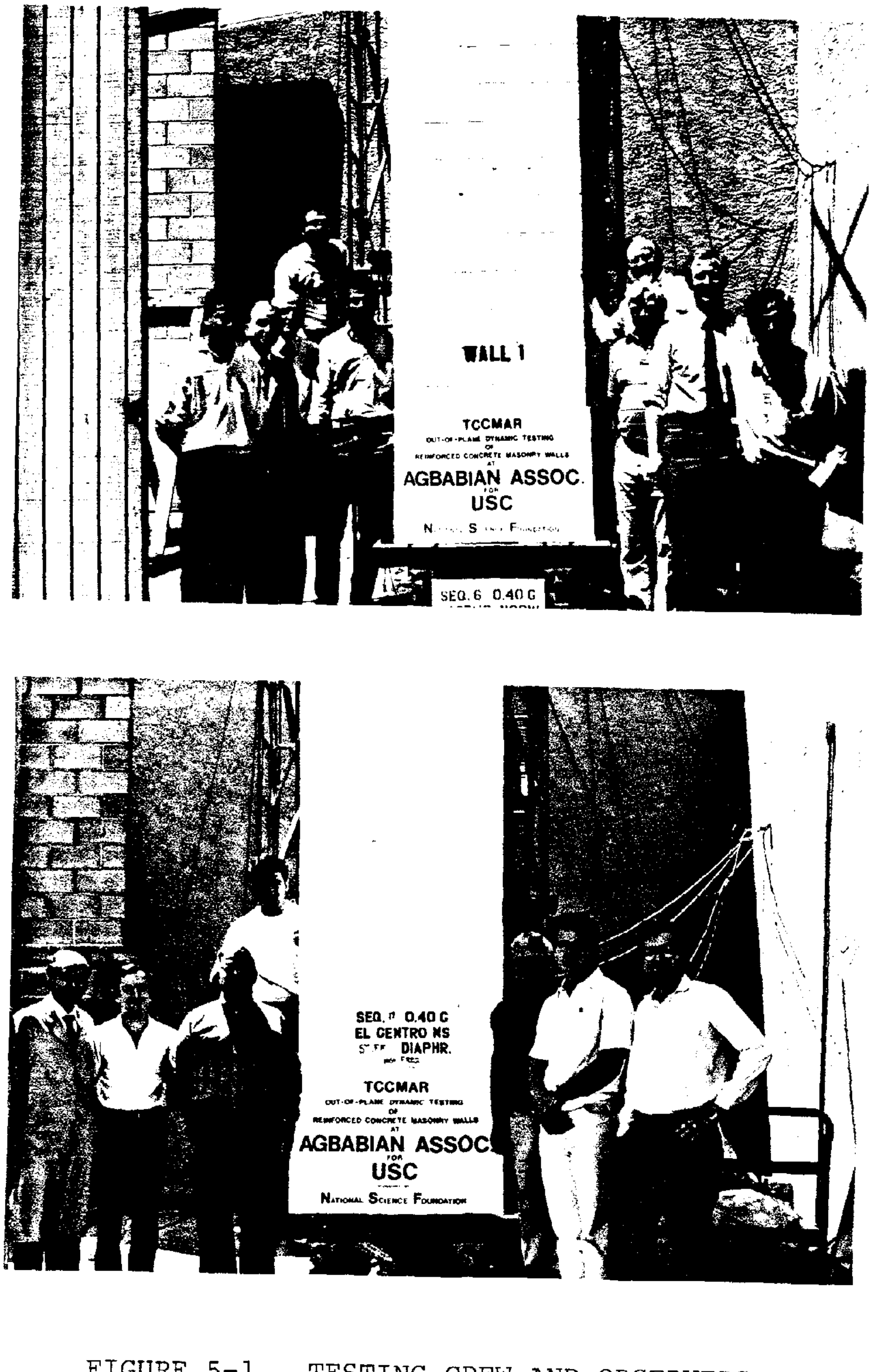


FIGURE 5-1. TESTING CREW AND OBSERVERS

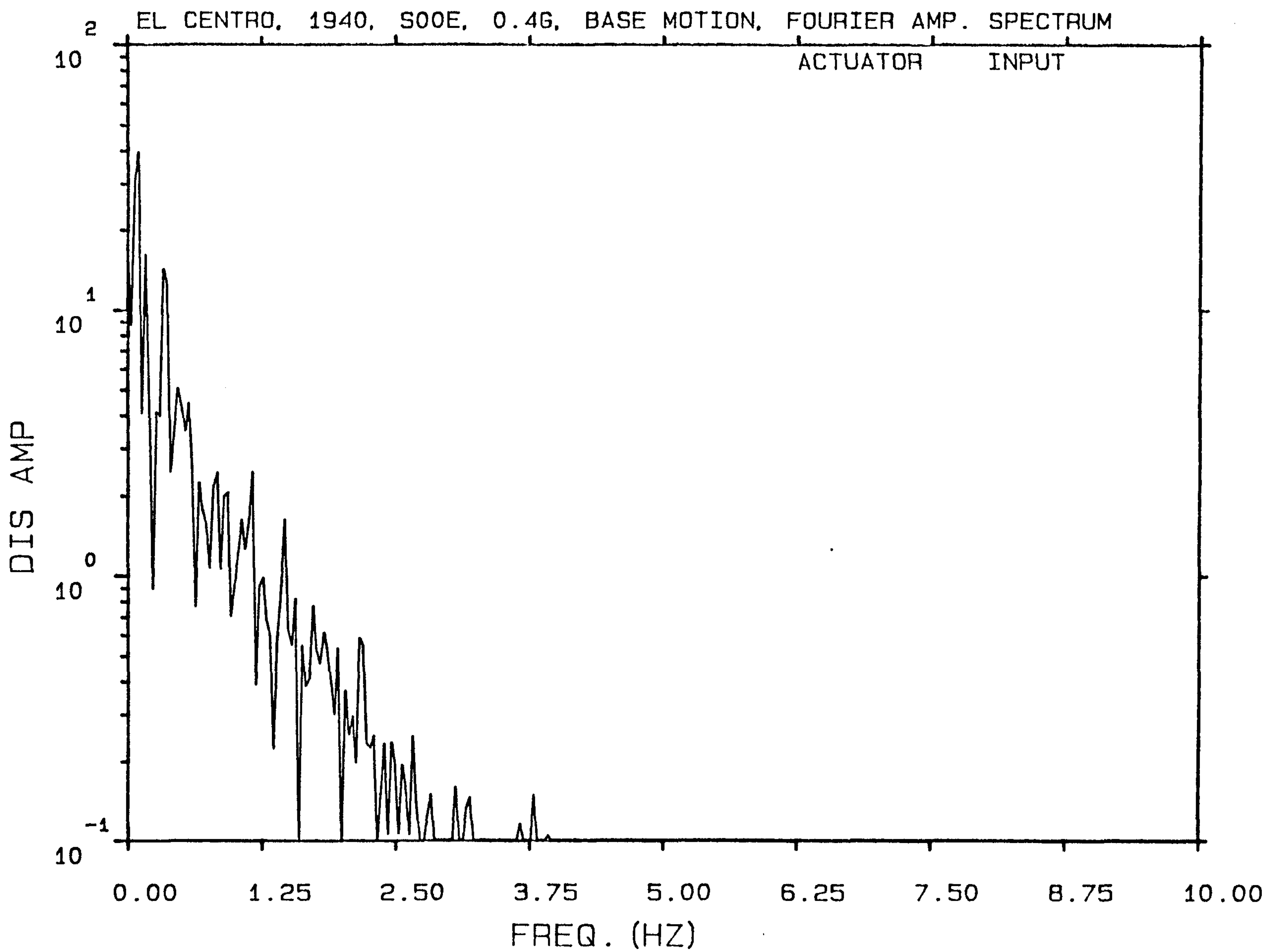


FIGURE 5-2. FOURIER SPECTRUM FOR EL CENTRO MOTION WITH 30-SECOND DURATION

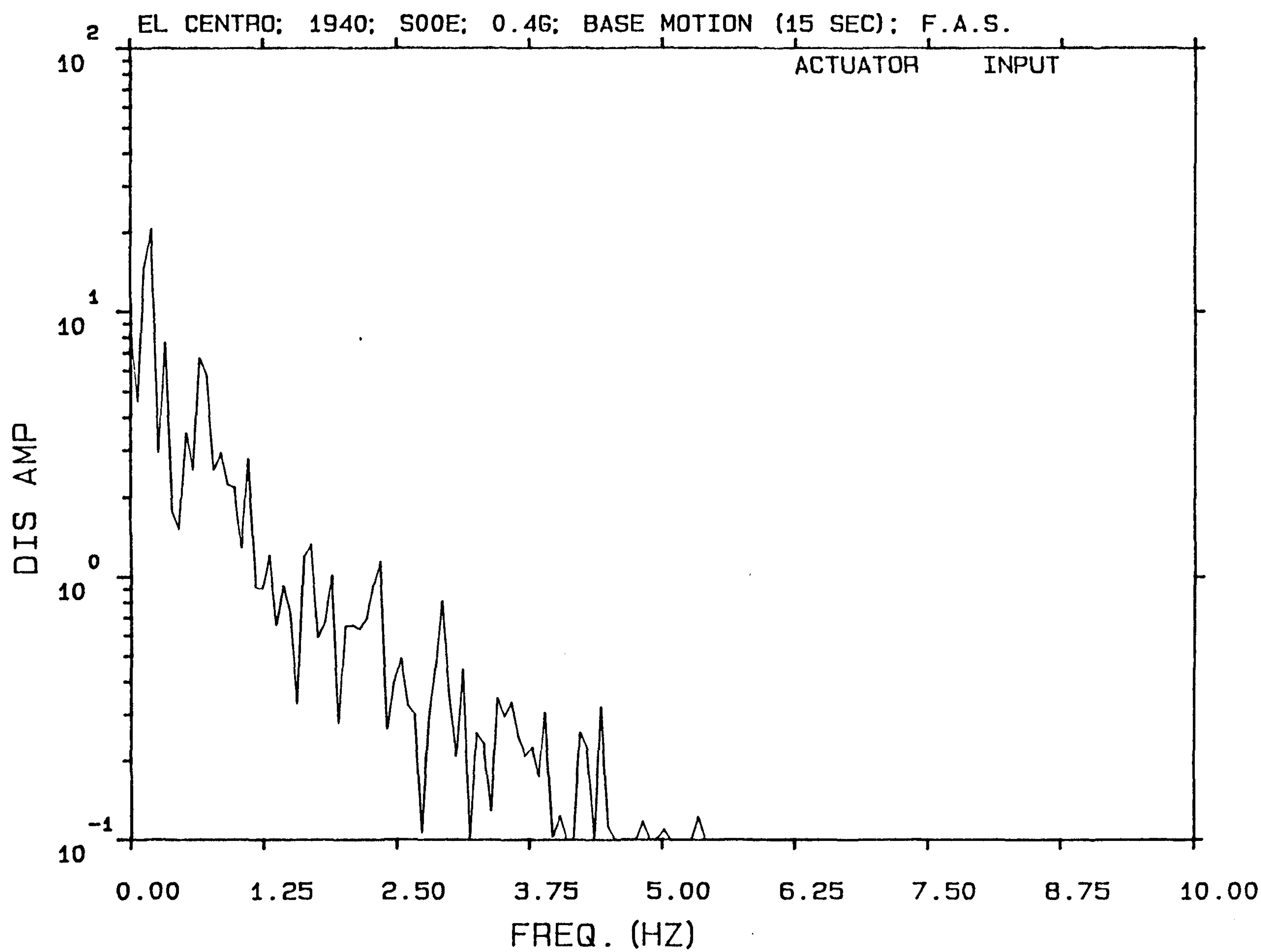


FIGURE 5-3. FOURIER SPECTRUM FOR EL CENTRO MOTION WITH DURATION COMPRESSED TO 15 SECONDS

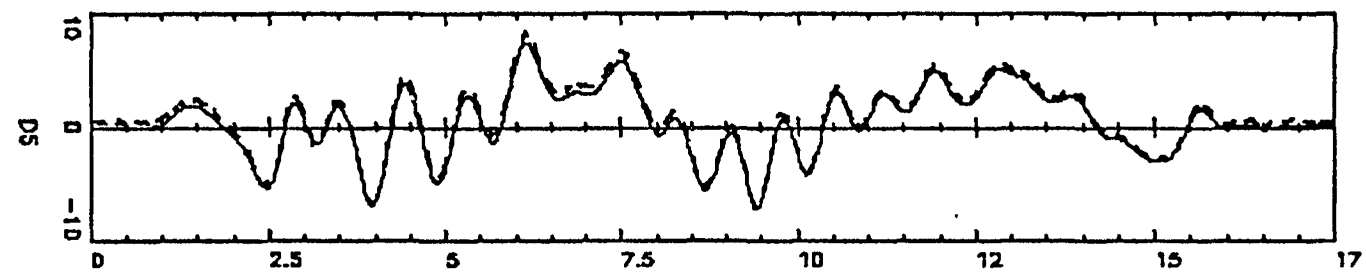


FIGURE 5-4. ABSOLUTE MIDHEIGHT DISPLACEMENT IN INCHES OF PANEL NO. 1 FOR SEQUENCE 14

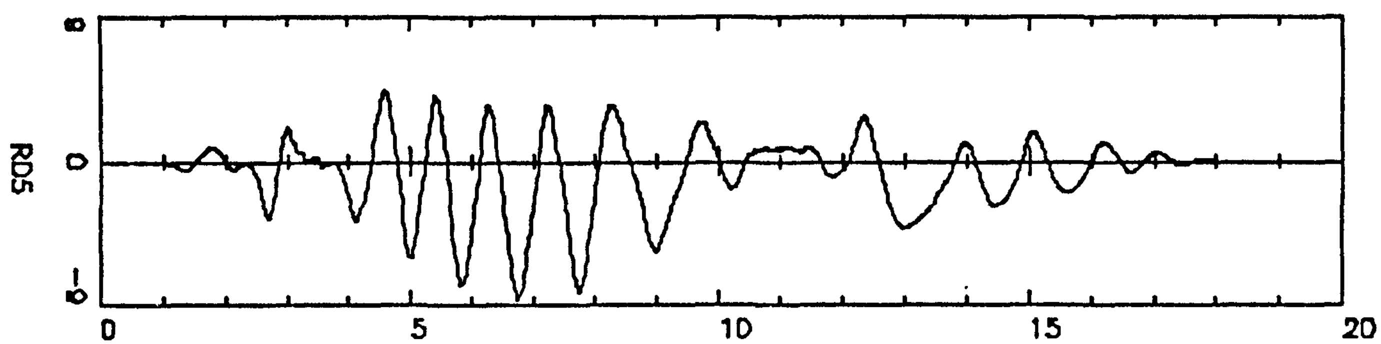


FIGURE 5-5. RELATIVE MIDHEIGHT DISPLACEMENT IN INCHES OF PANEL NO. 1 FOR SEQUENCE 18

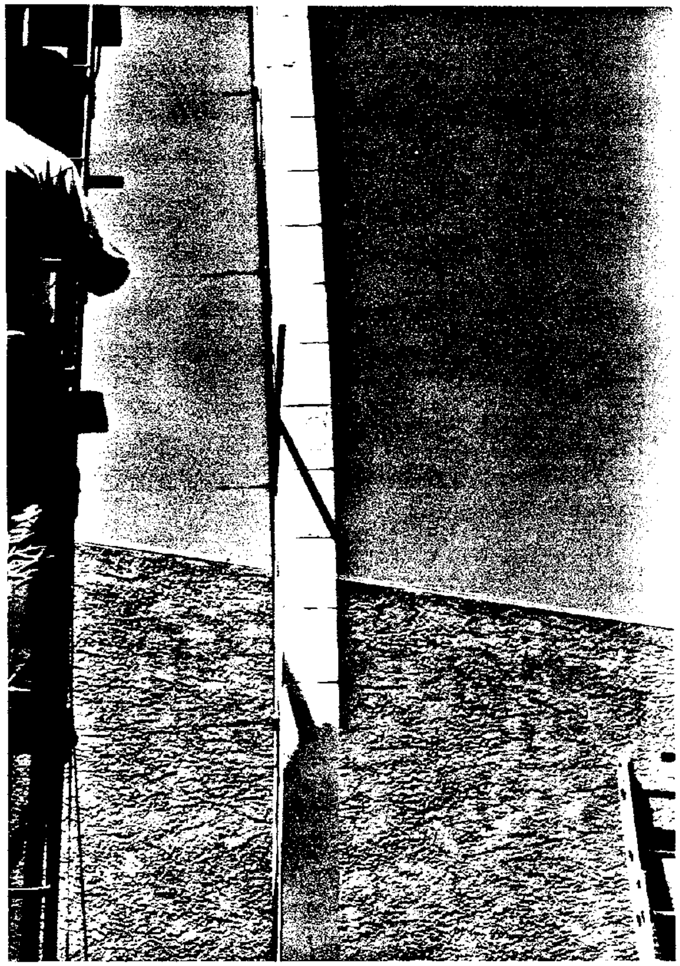


FIGURE 5-6. PERMANENT DEFORMATION IN WALL PANEL NO. 1 AFTER SEQUENCE 18

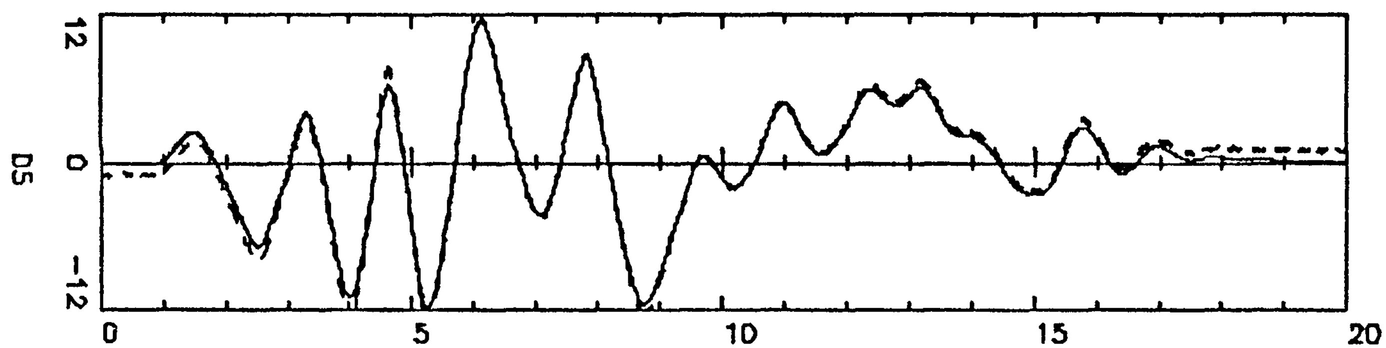


FIGURE 5-7. MIDHEIGHT ABSOLUTE DISPLACEMENT IN INCHES OF PANEL NO. 2 FOR SEQUENCE 15

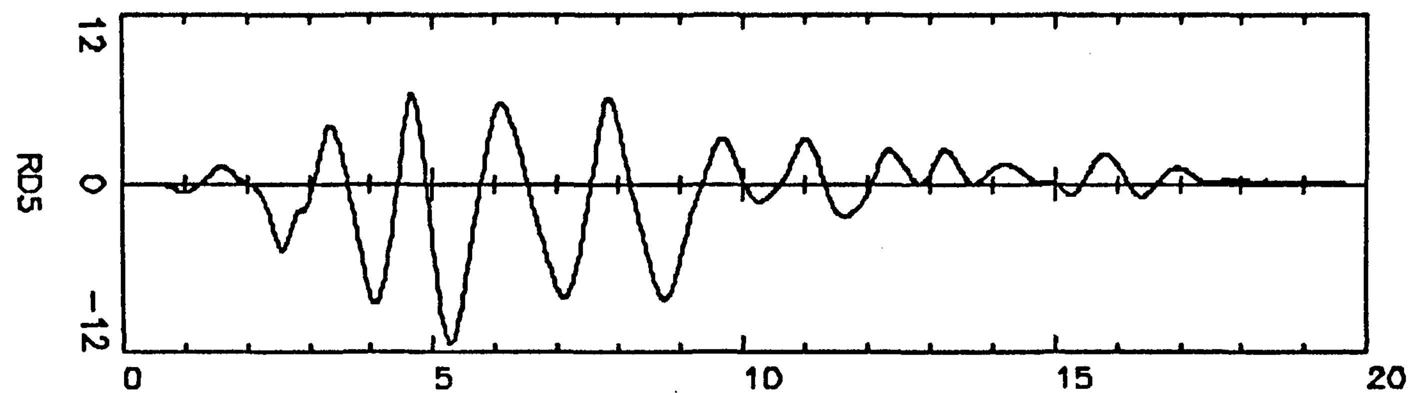
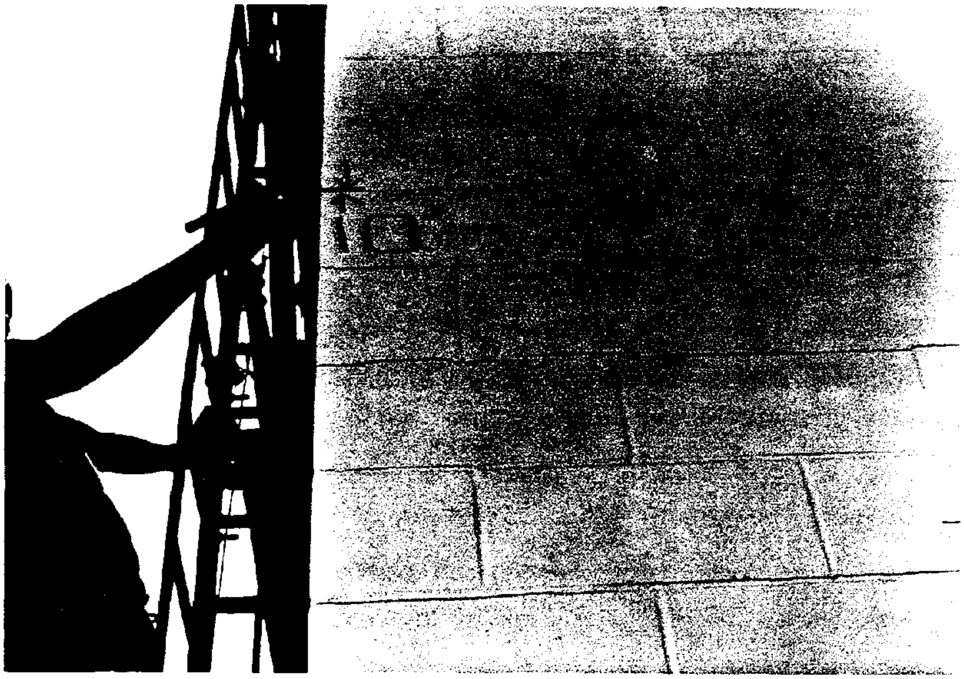


FIGURE 5-8. MIDHEIGHT RELATIVE DISPLACEMENT IN INCHES OF PANEL NO. 2 FOR SEQUENCE 15

(a)



(b)

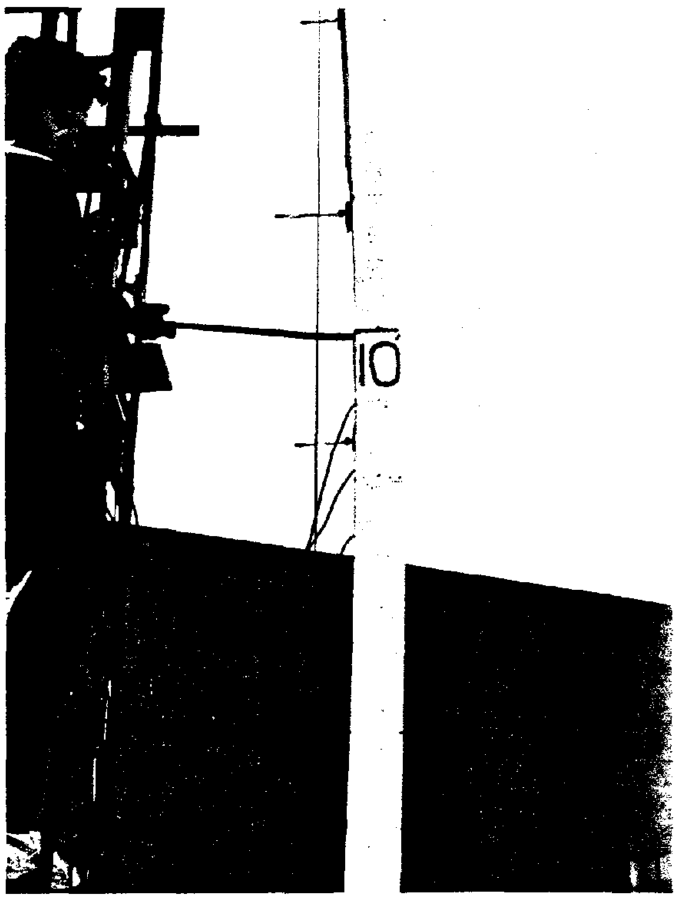


FIGURE 5-9. CRACK PATTERN (a) AND PANEL DEFORMATION (b)
OF PANEL NO. 2 AT THE END OF SEQUENCE 15

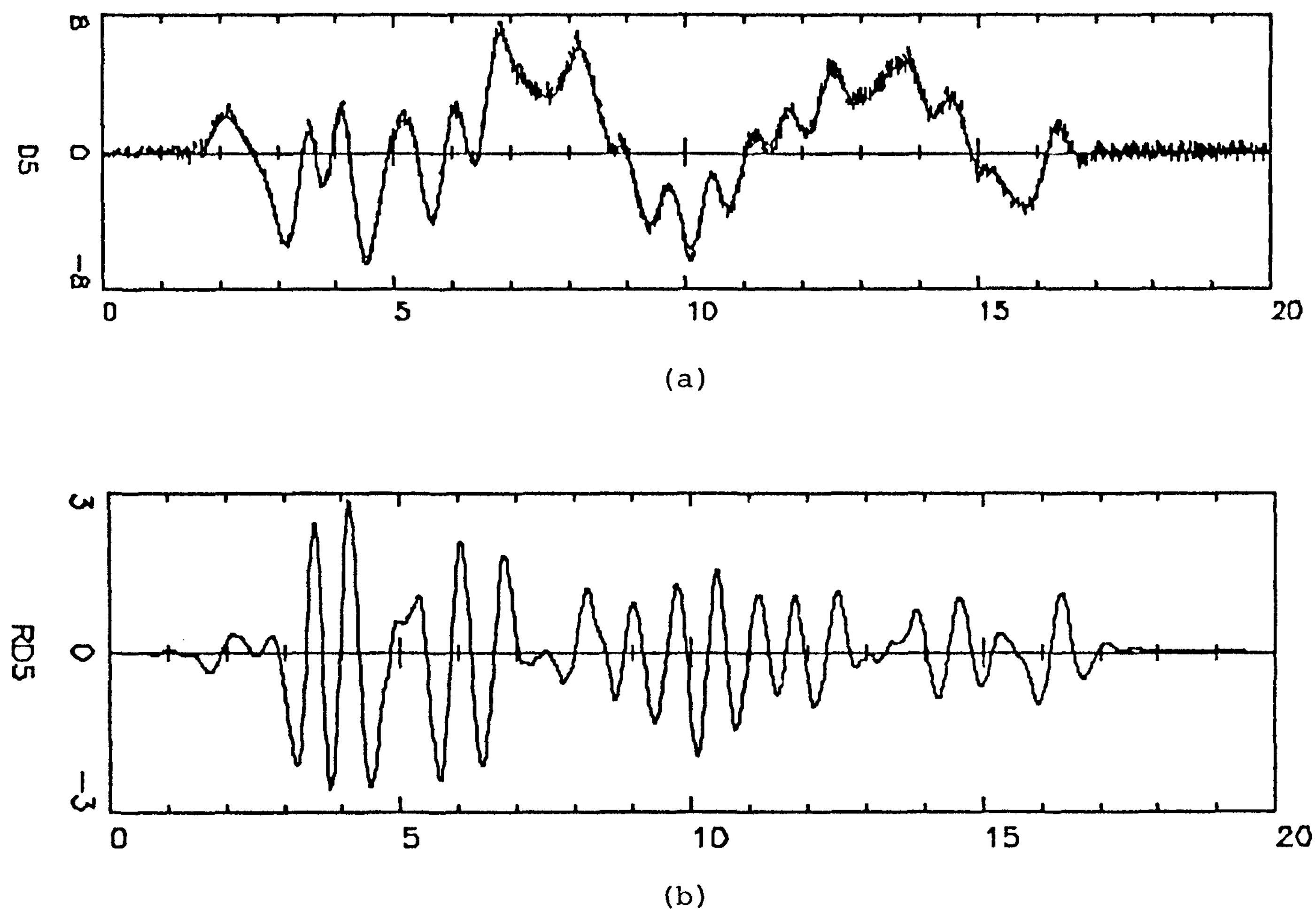


FIGURE 5-10. ABSOLUTE (a) AND RELATIVE (b) MIDHEIGHT PANEL DISPLACEMENT IN INCHES OF PANEL NO. 3 AT SEQUENCE 15

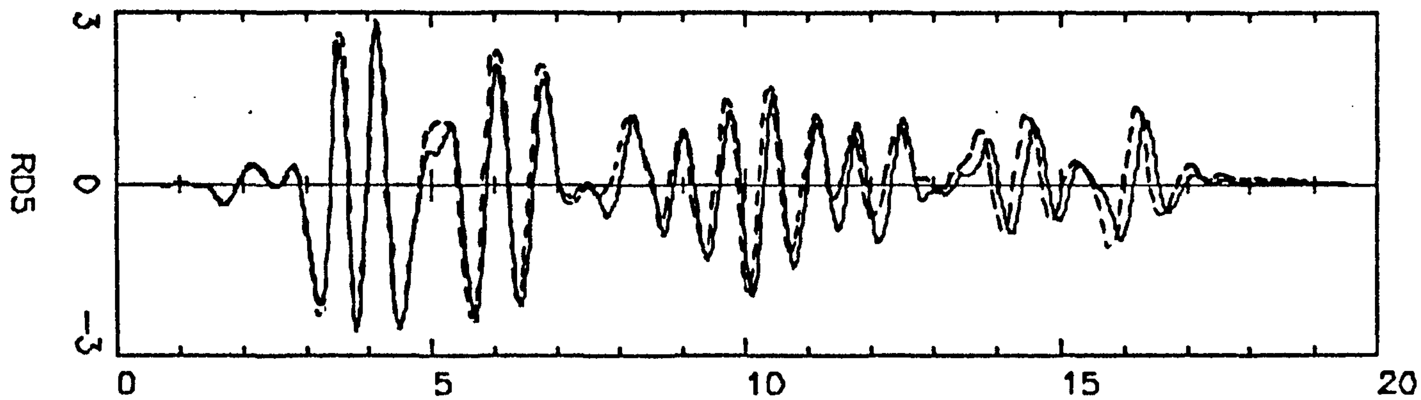


FIGURE 5-11. MIDHEIGHT RELATIVE DISPLACEMENT IN INCHES FOR PANEL NO. 3 FOR SEQUENCE 15 (SOLID) AND SEQUENCE 25 (DASHED)

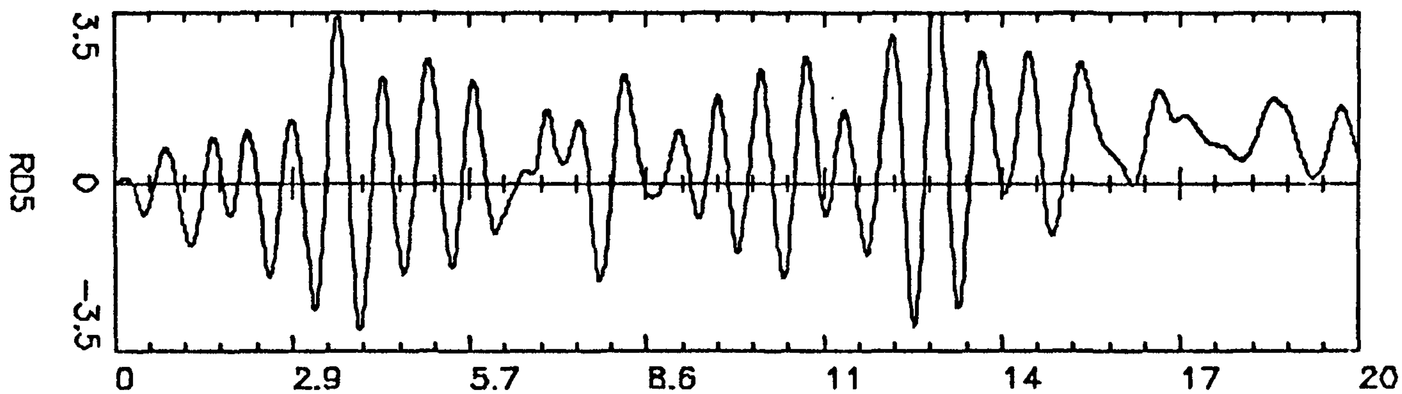


FIGURE 5-12. MIDHEIGHT RELATIVE DISPLACEMENT IN INCHES OF PANEL NO. 3 FOR SEQUENCE 28

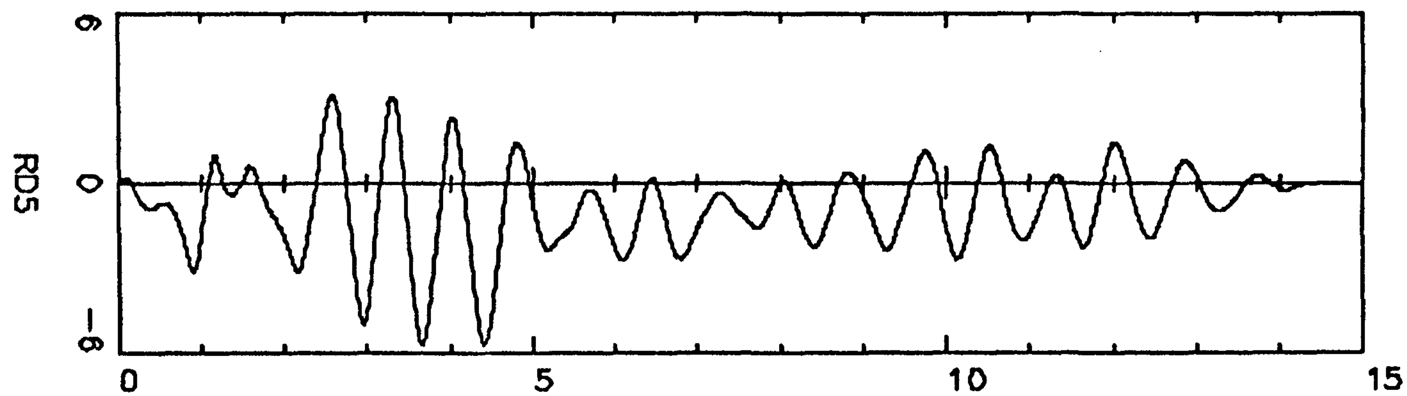


FIGURE 5-13. MIDHEIGHT RELATIVE DISPLACEMENT IN INCHES OF PANEL NO. 3 FOR SEQUENCE 30

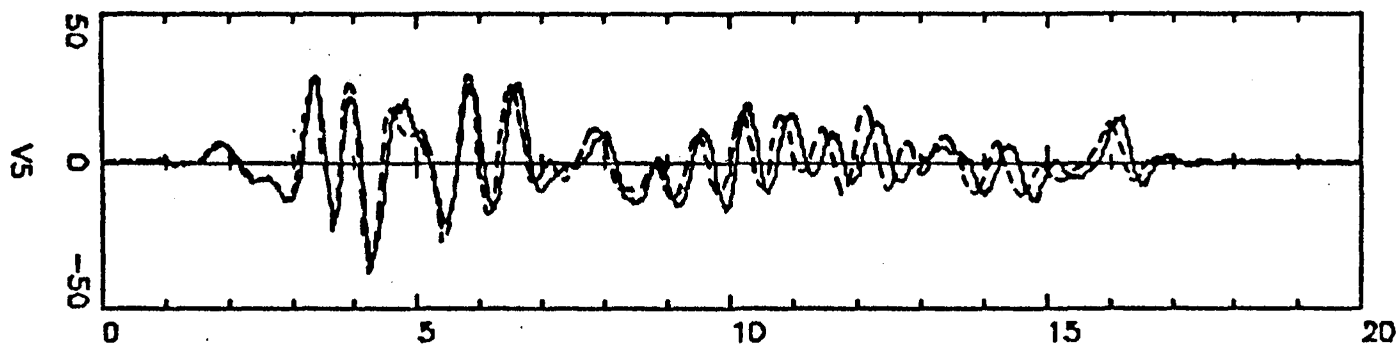


FIGURE 5-14. MIDHEIGHT VELOCITY RESPONSE IN INCHES/SECONDS AT SEQUENCE 15 OF PANEL NO. 3 (SOLID CURVE) AND PANEL NO. 4 (DASHED CURVE)

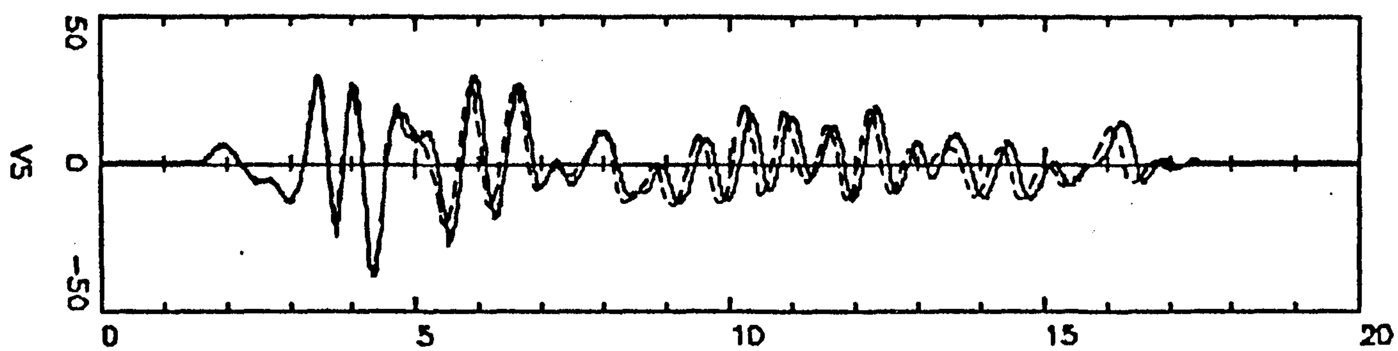


FIGURE 5-15. MIDHEIGHT VELOCITY RESPONSE IN INCHES/SECONDS OF PANEL NO. 4 FOR SEQUENCE 15 (SOLID CURVE) AND SEQUENCE 25 (DASHED CURVE)

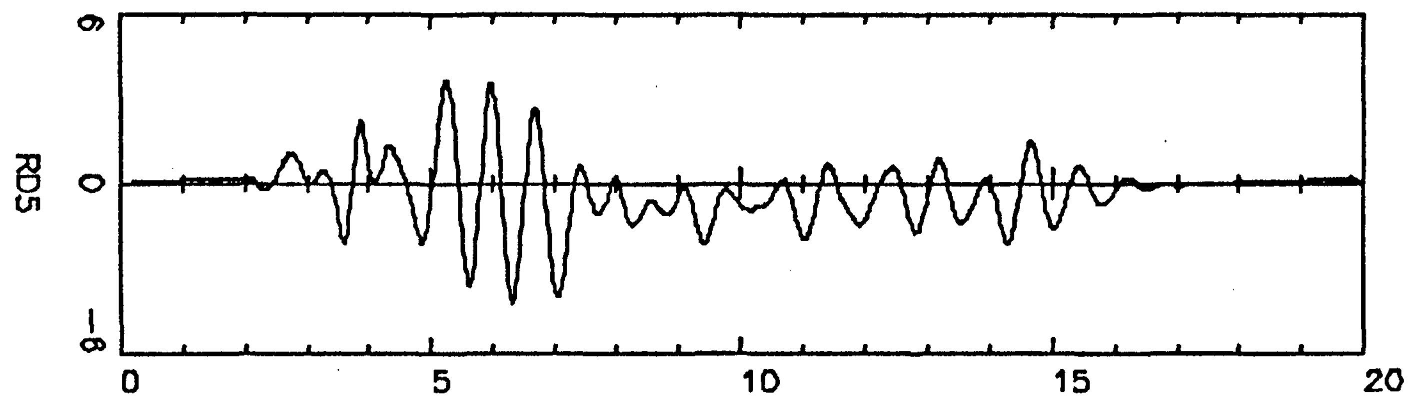


FIGURE 5-16. MIDHEIGHT RELATIVE DISPLACEMENT IN INCHES OF PANEL NO. 4 AT SEQUENCE 30

SECTION 6

PRELIMINARY EVALUATION OF TEST PROGRAM

The purpose of this section is to report the results of preliminary evaluation of the test program. More detailed analyses of the test data will be conducted in TCCMAR Task 2.4(b) using the developed analytical model for out-of-plane response.

The analyses presented in this section focus on three items: (1) actuator response to the input signal, (2) an evaluation of motion M8, and (3) wall panel response simulation to actuators motions. These items are discussed in the following subsections.

6.1 ACTUATOR RESPONSE

The first step in conducting the out-of-plane dynamic tests of the wall panels was to check the top and bottom actuators. This check was done in two steps. First the actuators were disconnected from the strong back. They were then checked on the side using heavy steel weights and different cyclic motions. The second step was to mount the actuators in their proper position in the test setup and connect them to panel 1.

In order to study the response of the two actuators (base and top) during dynamic testing a series of trial tests were conducted on panel 1 using the Hollister earthquake (low level motion was selected to avoid any damage to the panel itself). The signal was sent from the control computer to the actuators and was recorded on paper using an analog plotter. The actuator LVDT feedback signal was recorded and was plotted using the same analog plotter. Figure 6-1a shows the input recorded directly from the control computer and Figure 6-1b shows the LVDT feedback. Both figures are for the base motion. The figures are identical except for the uniform calibration factor of 1.67 (1.67 volts equals 1" displacement). Figure 6-2a is the control computer signal for the top actuator and Figure 6-2b is the LVDT feedback of the top actuator. The two signals are the same except for the uniform calibration factor of 1.61 (1.61 volts equals 1" displacement). As expected the calibration factors for the base and the top are basically the same (within the accuracy of the measurements made).

Based on these two figures, it was concluded that at the beginning of the test each actuator was faithfully reproducing the input signal and that the actuator transfer function in the frequency range of interest was essentially unity. As mentioned above these pilot runs were made using a low level excitation (EPA of 0.1g) to avoid wall damage. During the testing of the wall panels, after a test was run the recorded analog signals were played back and the general trend of each signal was visually inspected from the recordings made on paper using the analog plotter. During this cursory check only the general shape of the signal was inspected and due to calibration problems developed in the analog plotter the amplitude of the signals could not be determined with confidence.

After the data was digitized at the completion of the testing effort it became evident that, in some tests, some of the high frequency content of the input signal was missing from the signal recorded by the stringpot at station 9 (top actuator). There was also modification, although to a much lesser extent, in the signal recorded by the stringpot at station 1 (base actuator). This signal modification was present in tests that were associated with high level input accelerations (EPA of 0.4g). The question then becomes "Did the top actuator modify the input signal when driven into high g-level and to what extent?" It is important to clarify this point and to determine the energy and the characteristics of the actual excitations experienced by the wall panels. In the following discussion, the motions input to the actuators from the control computer will be referred to as the analytical input and the actual motions experienced at the base and the top of the wall panel (actuator output) will be referred to as the experimental input. The analysis reported in this and the subsequent sections are performed using MACRAN time series analysis program (USS, 1987).

In the following analysis, the energy content of the signals will constitute the main measure for performing comparisons between the analytical input and the experimental input. Comparisons will also be made on the Fourier spectra of these signals.

The root-mean-square (RMS) of the power of a signal is a good measure of the energy content of the signal. The RMS of the energy of a signal, X(t) is computed using the following three steps:

1. The power spectral density (PSD) of the signal is computed. The PSD gives a measure of energy distribution of a signal and is defined for a single record of duration T as follows:

$$S_{XX}(f) = \frac{2}{T} X(f) \cdot X^*(f)$$

Where,

$S_{XX}(f)$ = Power density function of X(t)
f = Frequency, Hz
X(f) = Fourier transform of X(t)
$X^*(f)$ = Complex conjugate of X(f)

2. The PSD computed above is then integrated over the frequency, f. Thus, the resulting quantity has the units of power and is a function of frequency.

$$\text{Power }(f) = \int_0^f S_{XX}(P)\,dP$$

In the above equation P is a dummy variable of integration.

3. Finally, RMS is computed by taking the square root of the power derived in the previous step. That is,

$$RMS(f) = \sqrt{\text{Power }(f)}$$

The RMS derived in the above equation is a function of frequency. In addition, its value at a given frequency determines how much of the power of the signal is concentrated within zero and the given frequency. Thus, comparing the RMS of two signals to each other, one can determine whether the two signals have the same energy content over the frequency range of interest.

The set of signals selected for the analysis is test #5 (Motion M5) of wall panel 3, i.e., sequence 305 (Table 5-3). Figure 6-3 shows the RMS plots for the velocity time histories at the base actuator. The curve labeled actuator input is the analytical input and the curve labeled actuator output is the experimental input. The figure clearly indicates that the base actuator has slightly modified the analytical input signal. The modification is only about 8.5% at 6 Hz which is the upper bound of the frequency range of interest, i.e., $0 \leq f \leq 6$ Hz. It can be concluded that the base actuator was reasonably tracking the analytical input. The Fourier amplitude spectra for the base analytical input and the base experimental input are shown in Figure 6-4 and Figure 6-5, respectively. Figure 6-6 shows the RMS plots for the velocity time histories at the top actuator. The energy associated with the analytical input (actuator input) is 65% larger than the experimental input (actuator output) at the frequency of 6 Hz. The Fourier spectra for the analytical input and the experimental input for the top actuator are shown in Figure 6-7 and Figure 6-8, respectively.

The velocity RMS is probably the best measure of energy content of a signal, and since the bulk of the recorded data is in the form of velocity time histories it was natural to use velocity data for energy comparisons. The displacement time history is obtained by integrating (smoothing) the velocity time history. Since the actuators were displacement driven devices it is also informative to compare the displacement RMS for the analytical and the experimental data. The RMS plots for the base displacement time history are shown in Figure 6-9. The RMS plot for the top displacement time history is shown in Figure 6-10. The difference in energy for the base motions is about 10% at 6 Hz and the energy difference for the top motions at the same frequency is about 15%. The acceleration time history for the base motion obtained by differentiating the experimental velocity time history at station 1 is shown in Figure 6-11a, while the corresponding acceleration time history at the top is depicted in Figure 6-11b. The peak acceleration at the base is 0.38g and the peak acceleration at the top is 0.29g. The corresponding peak accelerations at the base and at the top for the analytical input are 0.43g and 0.41g, respectively. Based on the results presented, it is obvious that the top actuator has modified the input signals for motions with high g-levels like motion M5.

The analysis of the experimental data as reported in this section indicates that the actuators, and the top actuator in particular have modified the input signal and the actual energy delivered to the test specimens were less than the energy of the input signals. However, the existing experimental data base can still be used for future analytical and/or numerical model calibrations since the actual input motion imparted on the specimens are known. Therefore, in any future analytical and/or numerical analysis of the wall panels the actual motions at the base (signal of stringpot at station 1) and at the top (signal of stringpot at station 9) should be used as input to a given model if it is desired to simulate the actual wall response during a given test. If the analytical input is used as the excitation of the model the simulated panel response may not correspond to the experimental results.

6.2 AN EVALUATION OF MOTION M8

Motion M8 is derived from Motion M7 which is the 1940 El Centro NS motion scaled to an EPA of 0.4g with an acceleration duration of 30 sec and is associated with a rigid diaphragm response at the top. The duration of motion M7 is compressed from 30 sec to 15 sec to yield the input signal referred to as

Motion M8. Test #9 of wall panel 3, i.e., sequence 309 is an example of this compacted motion. The acceleration record at the base of the wall panel obtained from the experimental input velocity record is shown in Figure 6-12a. The corresponding acceleration record at the top of the wall is shown in Figure 6-12b. The peak acceleration at the panel base is 1.26g and the peak acceleration at the panel top is 1.21g. It is of interest to establish the level of seismic intensity presented by this compacted seismic record.

It was determined that a simple procedure for obtaining the seismic intensity level of Motion M8 was to compare the RMS of the power of both the base and the top signals to the RMS of the power of the base signal of Motion M5. The base acceleration signal of Motion M5 has a peak ground acceleration of 0.43g which corresponds to an EPA of 0.4g. Such a comparison is shown in Figure 6-13. This figure illustrates (1) the acceleration RMS for the base input (analytical input) of Motion M5 (+); (2) the acceleration RMS for the base input (experimental input) of Motion M8 (#); and (3) the acceleration RMS for the top input (experimental input) of Motion M8 ($). The figure indicates that at a frequency of 6 Hz, RMS of the power of the base excitation signal of Motion M8 is 29% larger than that of Motion M5, and the RMS of the power of top excitation signal of Motion M8 is 38% larger than that of Motion M5. Based on this energy comparison one may conclude that if the EPA of Motion M5 is 0.4g, then the EPA associated with Motion M8 should be in the order of 0.5g (1.3 times 0.4g). However, it must be emphasized that this EPA value is only an estimate and more rigorous analysis of response spectra smoothing is required to establish accurately the EPA associated with Motion M8.

6.3 WALL PANEL RESPONSE EVALUATION

It is of interest to compare the response of the wall panels when subjected to the analytical input (actuator input) to the response when the experimental input (actuator output) is imparted on the panels. To perform this comparison an analytical model of the wall panel was created and the response of the model to different excitations were determined and comparisons were made.

The computer code used for wall panel modeling is the LPM/I computer program (EKEH, 1987) which allows for performing linear and nonlinear time history analyses. In the analysis presented in this section a linear elastic model consisting of six beam elements was created to represent a given masonry wall panel specimen. The properties of the beam elements are prescribed by the length of the beam, elastic modulus, density, shear area, and principal moment of inertia associated with out-of-plane bending. The compressive strength of masonry wall panel 3 was set at 3525 psi and the panel weight was set at 47 psf. The viscous damping associated with the beam element was set at 2% of the critical damping. The base of the model was excited by the base input velocity and the top of the model was excited by the top input velocity.

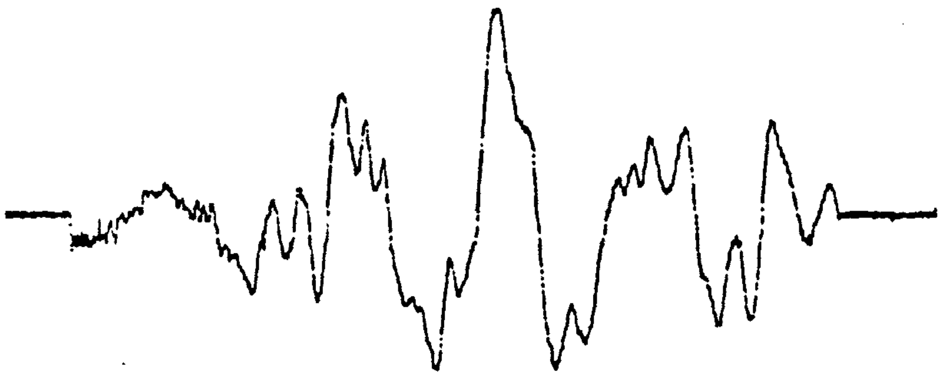
Due to the limited scope of this study only one panel (panel 3) and one testing sequence (sequence 305) were analyzed. The first analysis was conducted for panel #3 subjected to the analytical input (actuator input) associated with sequence 305. Figure 6-14 shows the midheight dynamic seismic moment. The midheight relative displacement is shown in Figure 6-15. The panel midheight relative displacement, Δ_{RM}, is defined as

$$\Delta_{RM} = 1/2\ (\Delta_T - \Delta_B) - \Delta_M$$

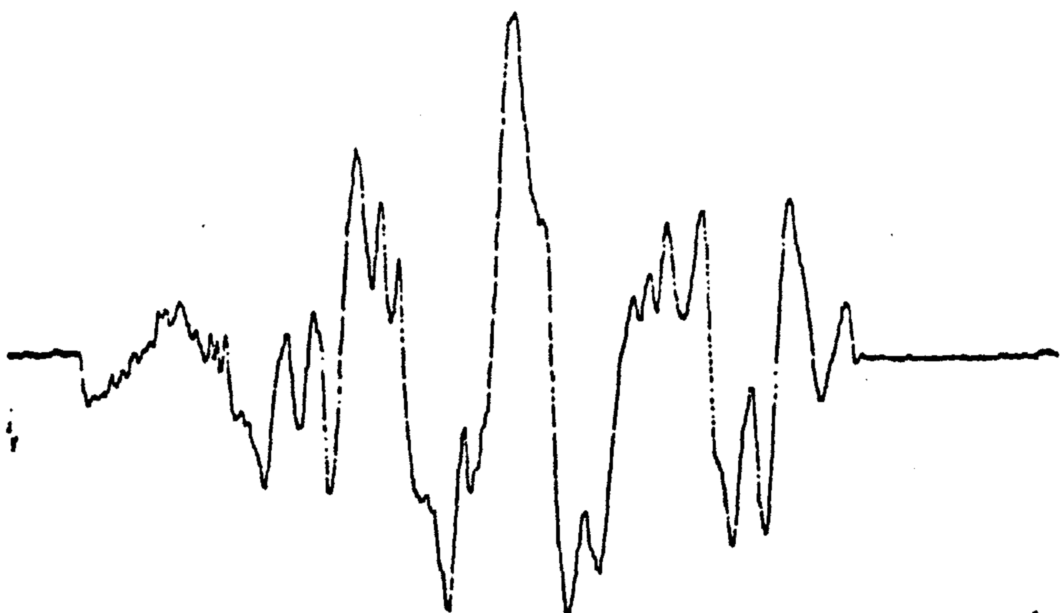
where

Δ_T = Absolute displacement at panel top
Δ_B = Absolute displacement at panel base
Δ_M = Absolute displacement at panel midheight

The second analysis was conducted for the same panel subjected to the experimental input (actuator output) associated with sequence 305 which is the actual motions experienced by the wall panel specimen. The midheight dynamic seismic moment for the second analysis is shown in Figure 6-16. The corresponding midheight relative displacement is shown in Figure 6-17. Comparing the results of the first simulation to those of the second simulation indicates that although the energy for the top motion for the experimental input (second analysis) was less than the energy for the top motion for the analytical input (first analysis), the induced midheight panel response is quite similar for both analyses. Further studies are required in order to fully understand the effects of this top actuator modification of the input signal on the response of the tested specimens. This brief analysis simply indicates that although the motion at the top was modified by the actuator, the induced response in the panel was not necessarily less intense than the induced response in the panel if no input modification was present.

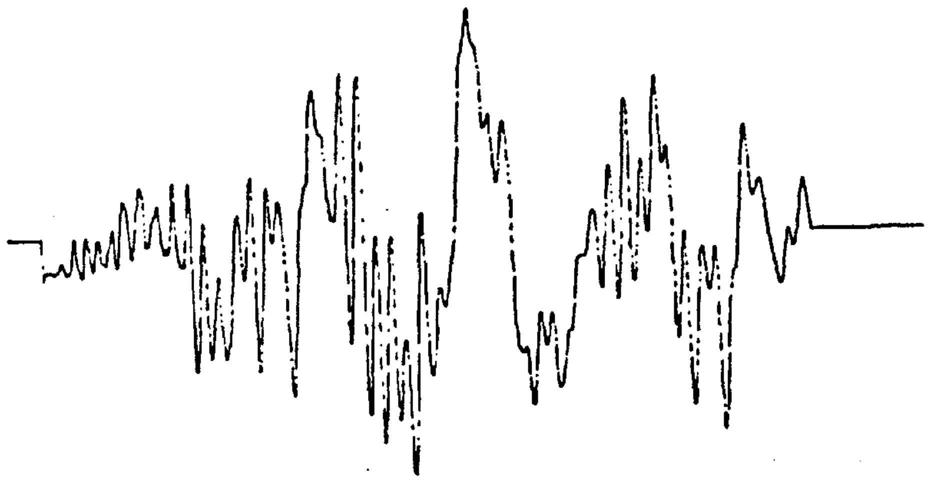


(a)

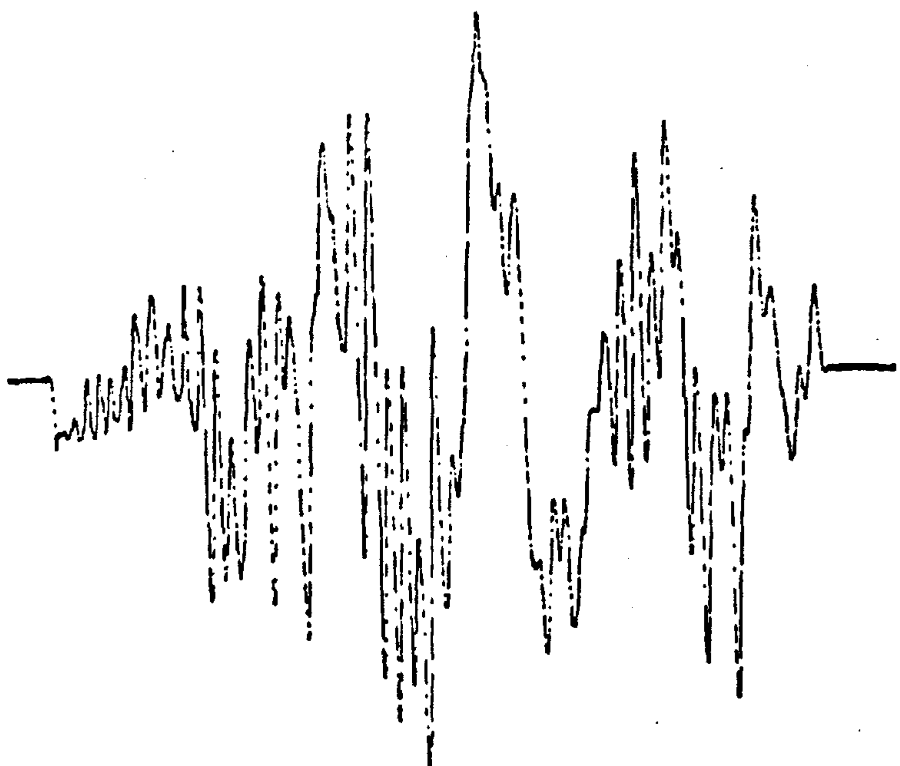


(b)

FIGURE 6-1. ANALOG SIGNAL FOR COMPUTER INPUT MOTION (a) AND THE ACTUATOR LVDT (b) FOR THE BASE ACTUATOR



(a)



CALIBRATION FACTOR = 1.61

(b)

FIGURE 6-2. ANALOG SIGNAL FOR COMPUTER INPUT MOTION (a) AND THE ACTUATOR LVDT (b) FOR THE TOP ACTUATOR

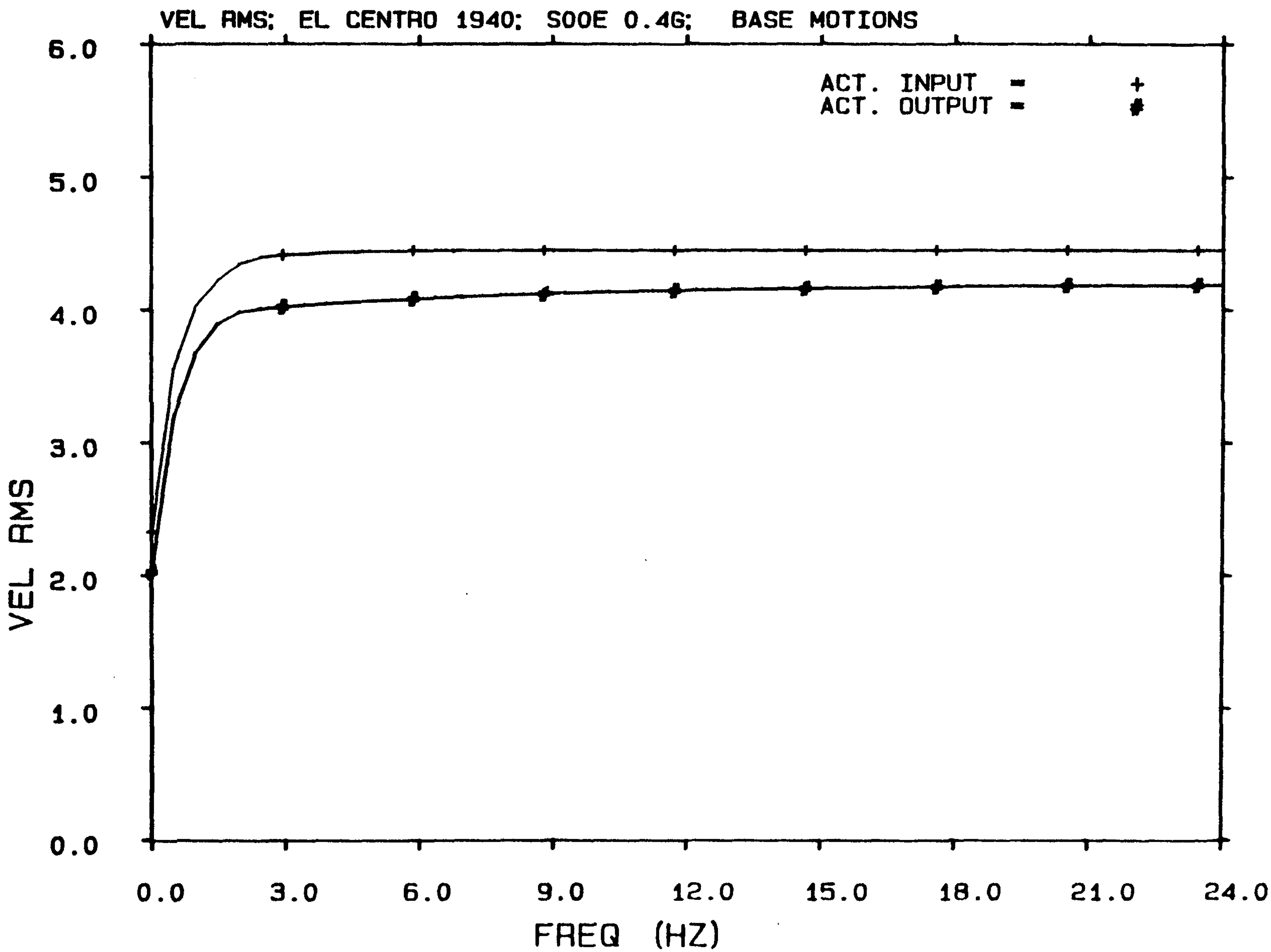


FIGURE 6-3. VELOCITY RMS COMPARISON FOR BASE ACTUATOR INPUT (+) AND OUTPUT (#) FOR SEQUENCE 305

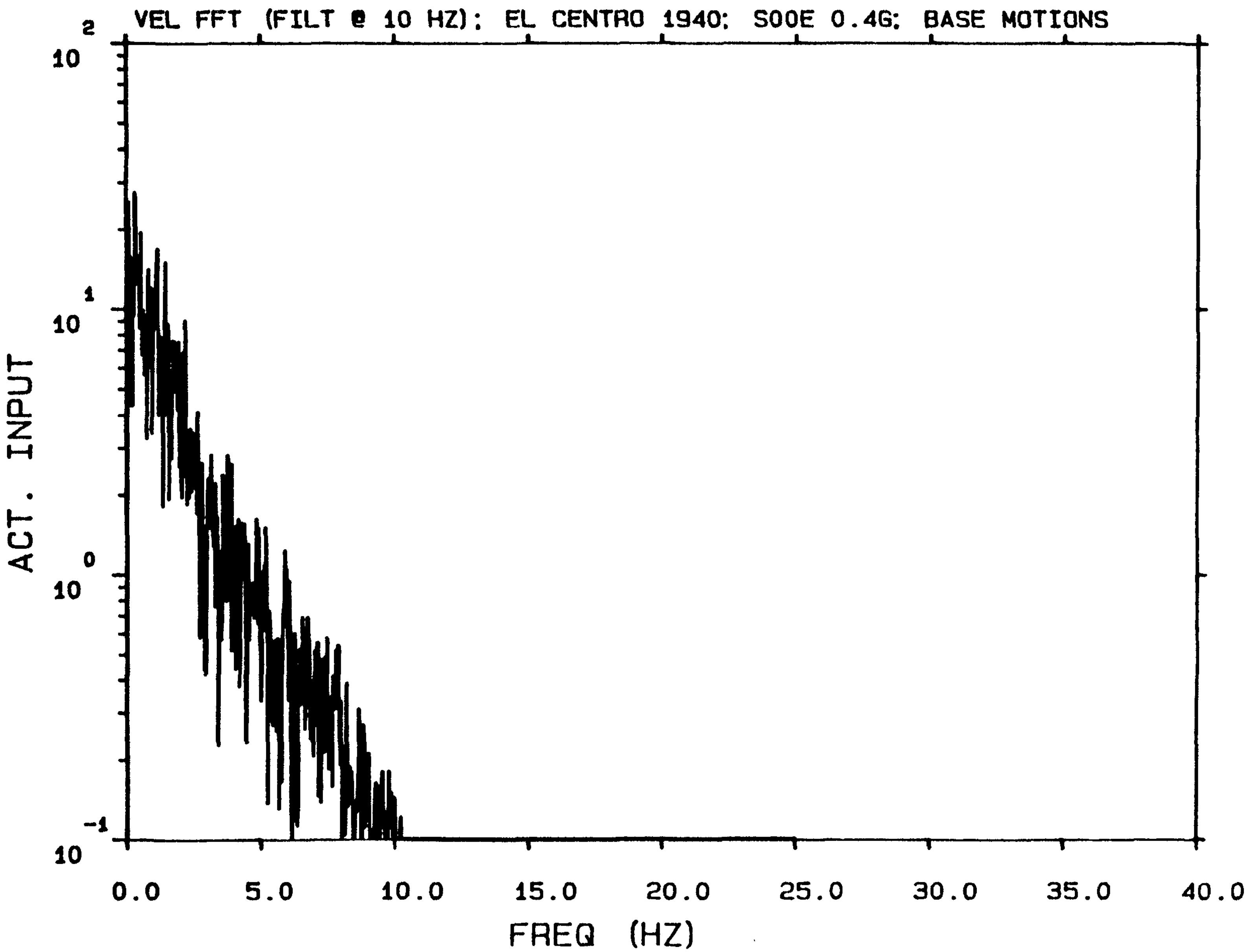


FIGURE 6-4. VELOCITY FOURIER AMPLITUDE SPECTRUM FOR BASE ACTUATOR INPUT FOR SEQUENCE 305

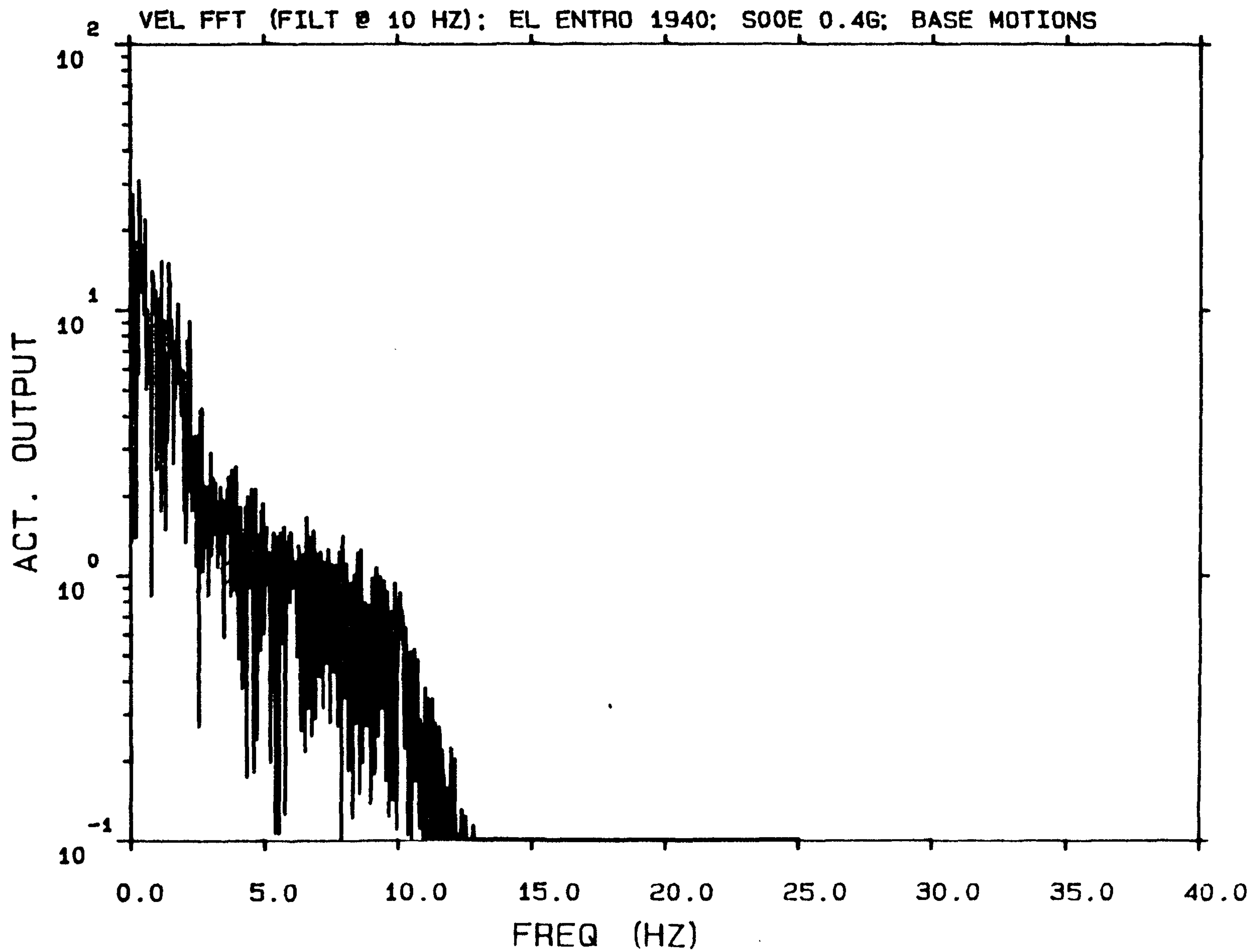


FIGURE 6-5. VELOCITY FOURIER AMPLITUDE SPECTRUM FOR BASE ACTUATOR OUTPUT FOR SEQUENCE 305

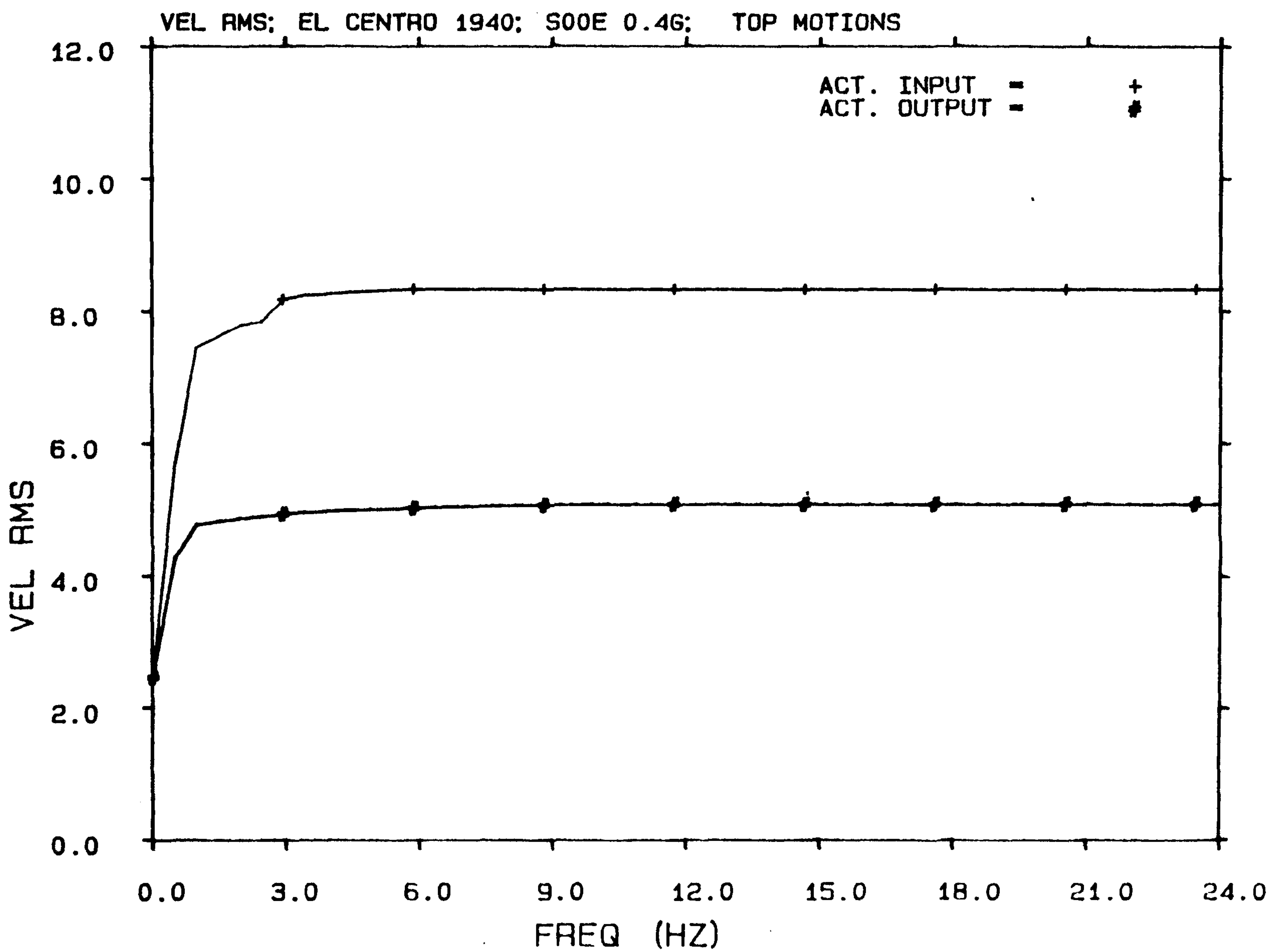


FIGURE 6-6. VELOCITY RMS COMPARISON FOR TOP ACTUATOR INPUT (+) AND OUTPUT (#) FOR SEQUENCE 305

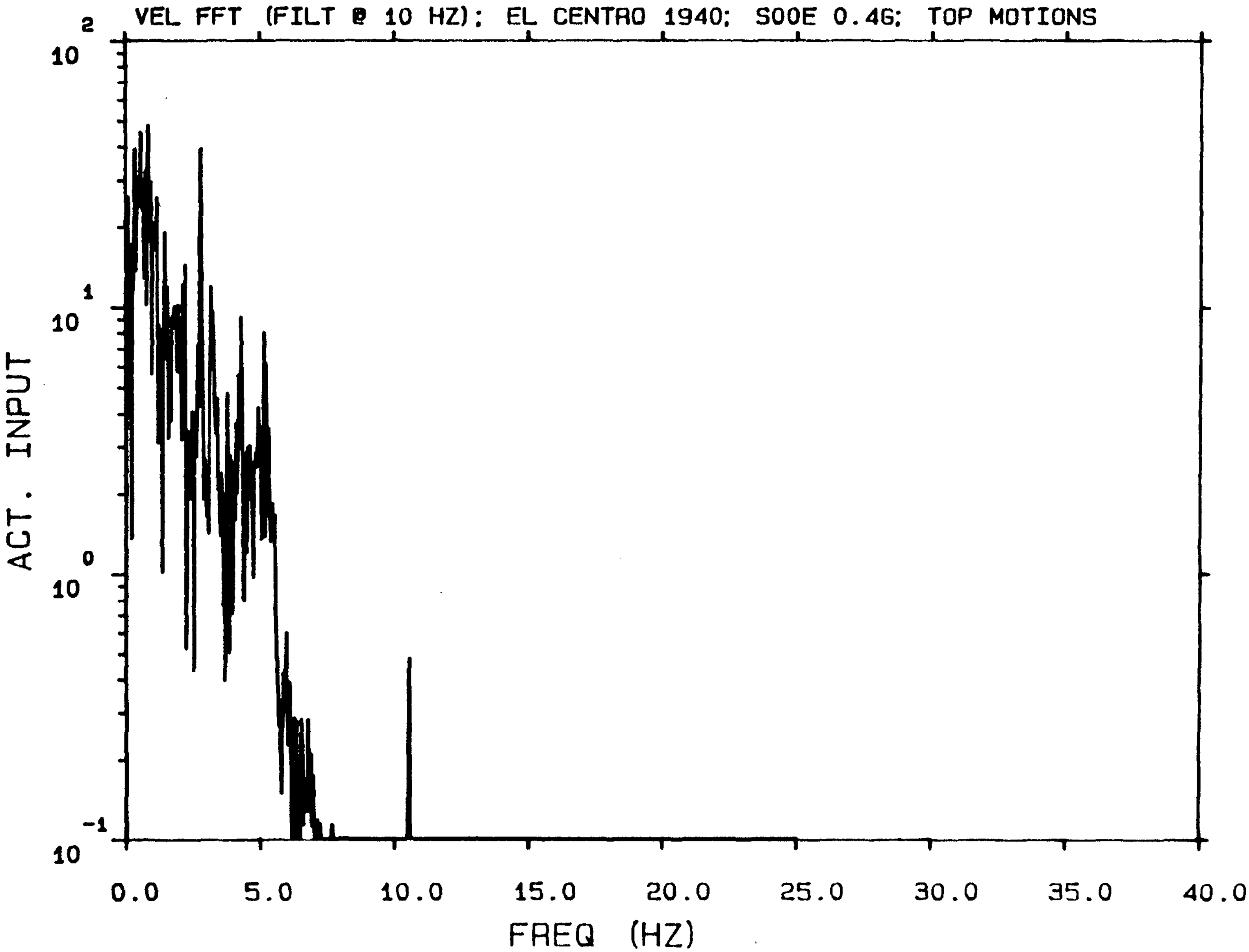


FIGURE 6-7. VELOCITY FOURIER AMPLITUDE SPECTRUM FOR TOP ACTUATOR INPUT FOR SEQUENCE 305

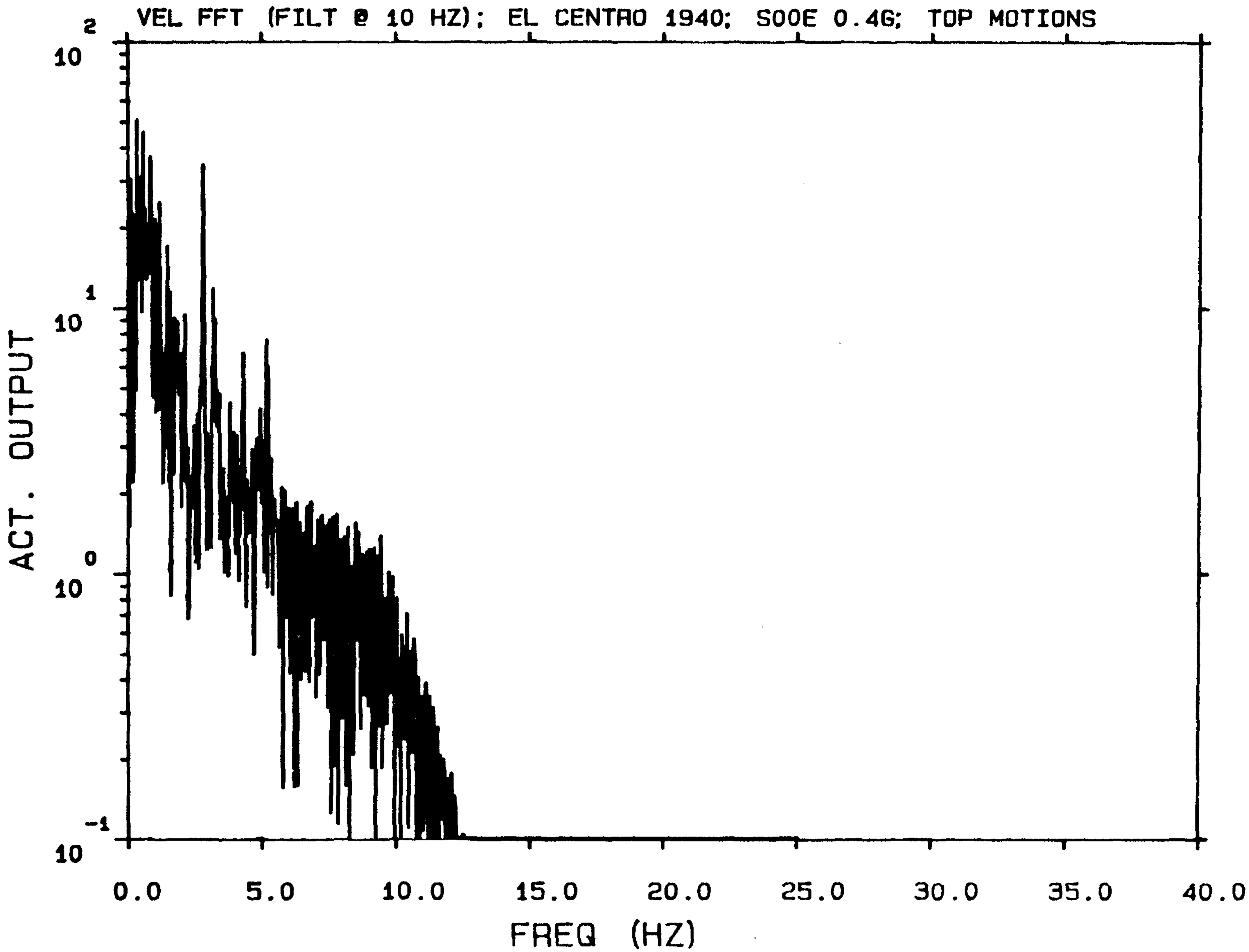


FIGURE 6-8. VELOCITY FOURIER AMPLITUDE SPECTRUM FOR TOP ACTUATOR OUTPUT FOR SEQUENCE 305

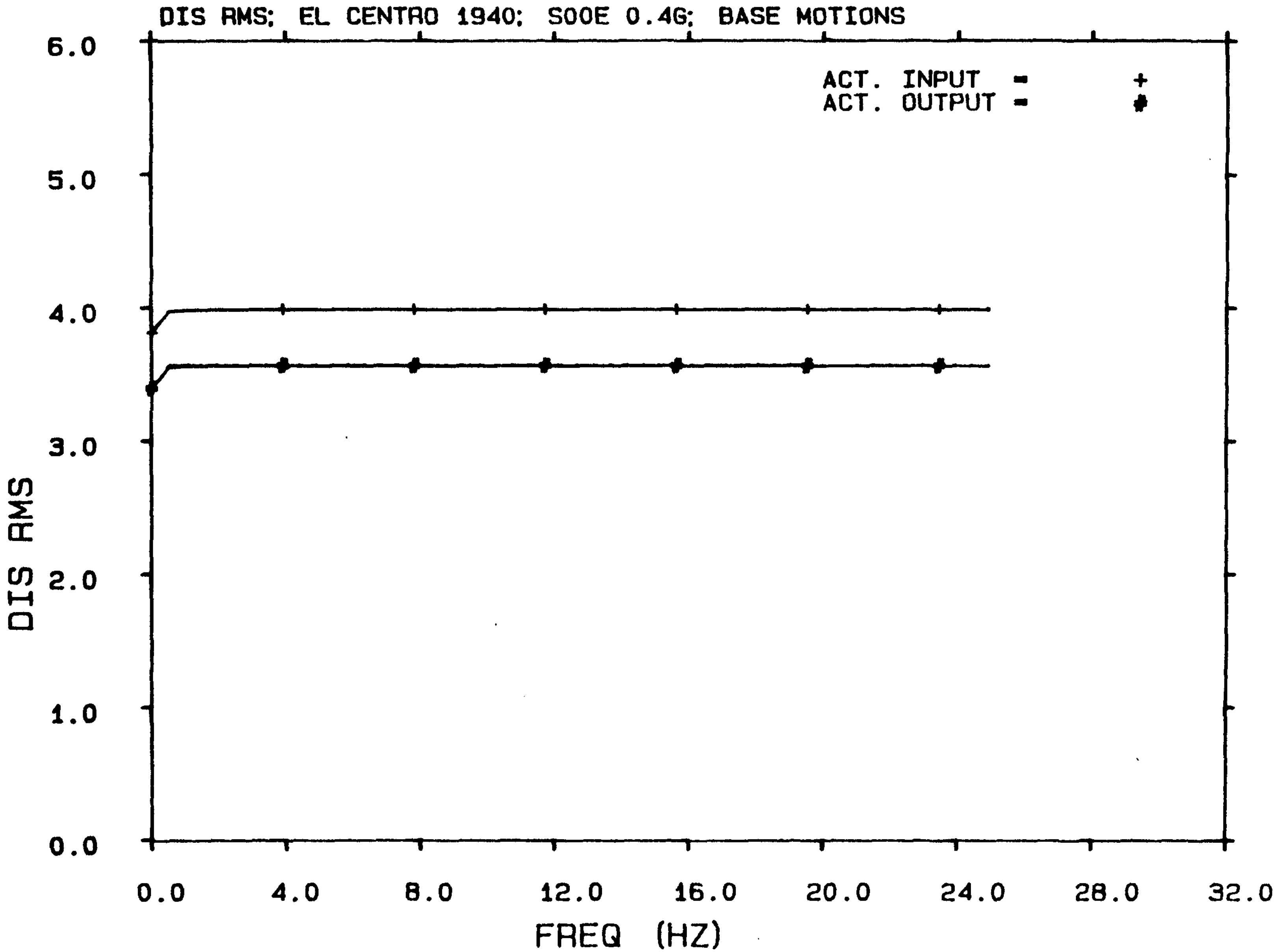


FIGURE 6-9. DISPLACEMENT RMS COMPARISON FOR BASE ACTUATOR INPUT (+) AND OUTPUT (#) FOR SEQUENCE 305

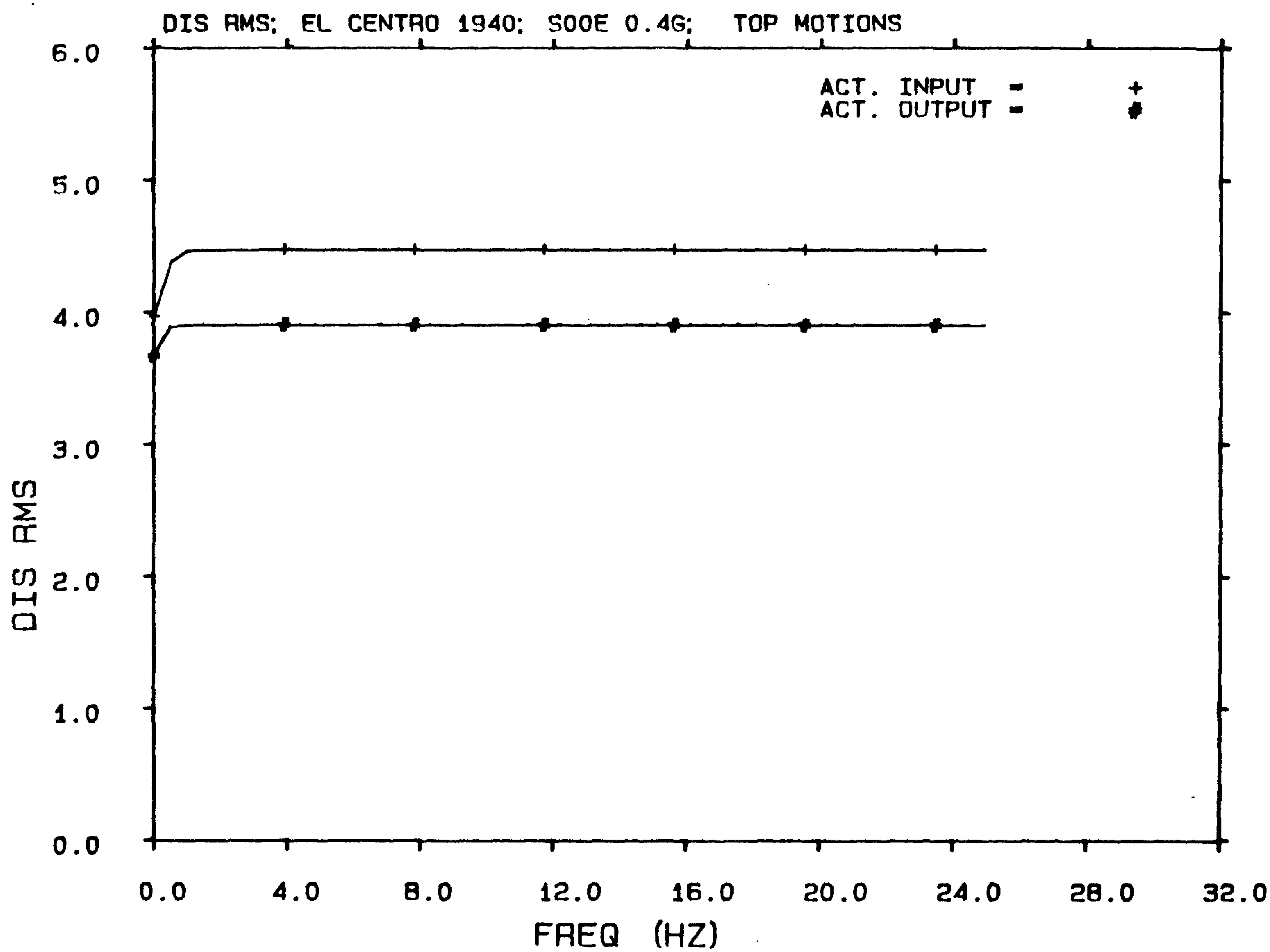
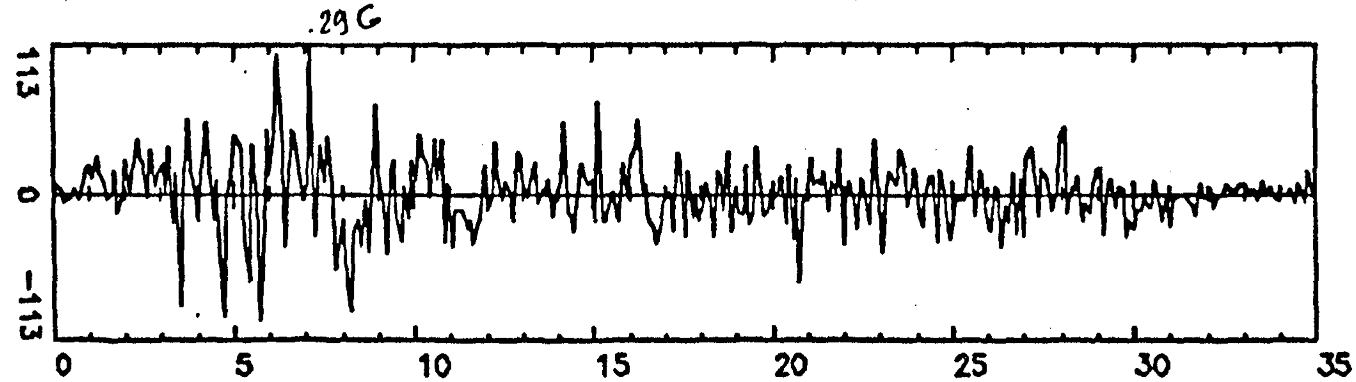
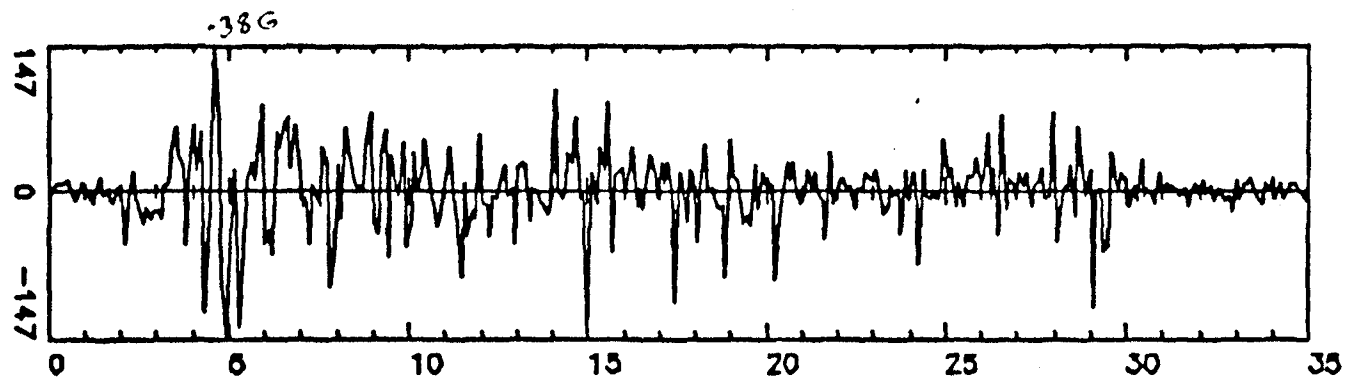


FIGURE 6-10. DISPLACEMENT RMS COMPARISON FOR TOP ACTUATOR INPUT (+) AND OUTPUT (#) FOR SEQUENCE 305

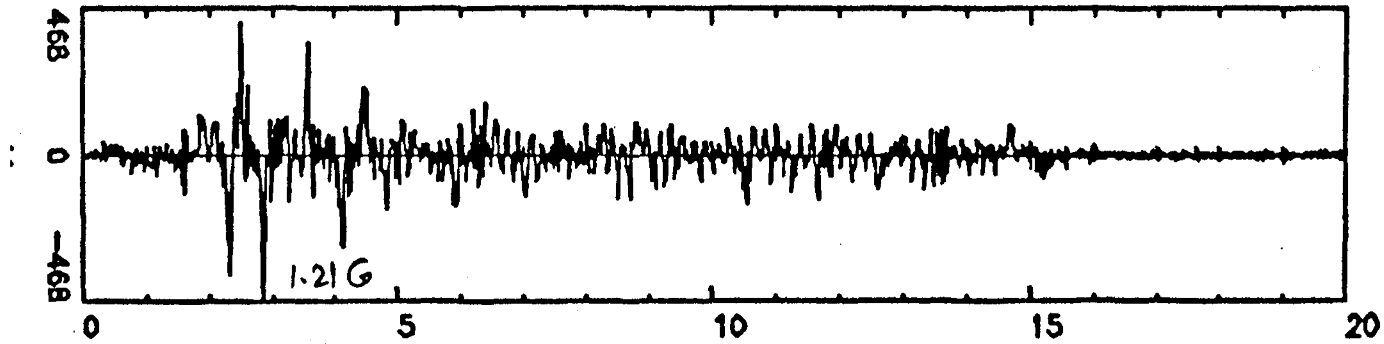


(b) Top

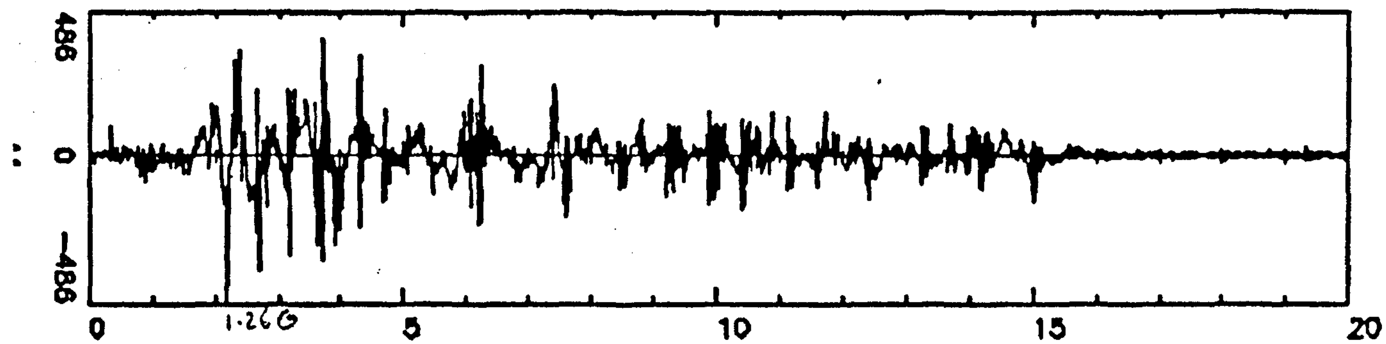


(a) Base

FIGURE 6-11. ACTUATOR OUTPUT BASE ACCELERATION (a) AND TOP ACCELERATION (b) TIME HISTORIES FOR SEQUENCE 305



(b) Top



(a) Base

FIGURE 6-12. ACTUATOR OUTPUT BASE ACCELERATION (a) AND TOP ACCELERATION (b) TIME HISTORIES FOR SEQUENCE 309

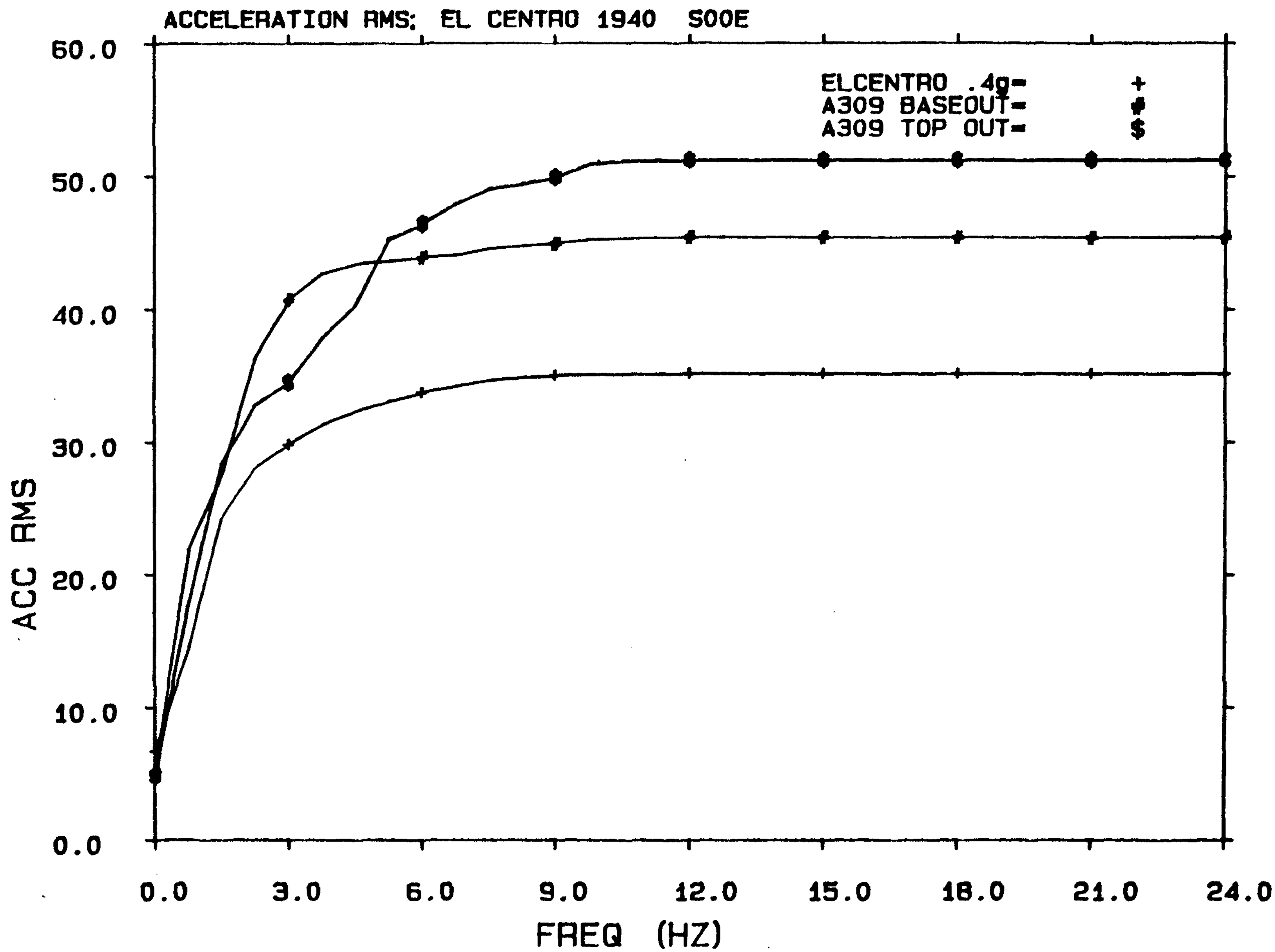


FIGURE 6-13. ACCELERATION RMS COMPARISON BETWEEN THE EL CENTRO GROUND ACCELERATION WITH EPA OF 0.4G AND THE ACCELERATION RECORDS FOR MOTION M8 OF TEST SEQUENCE 309

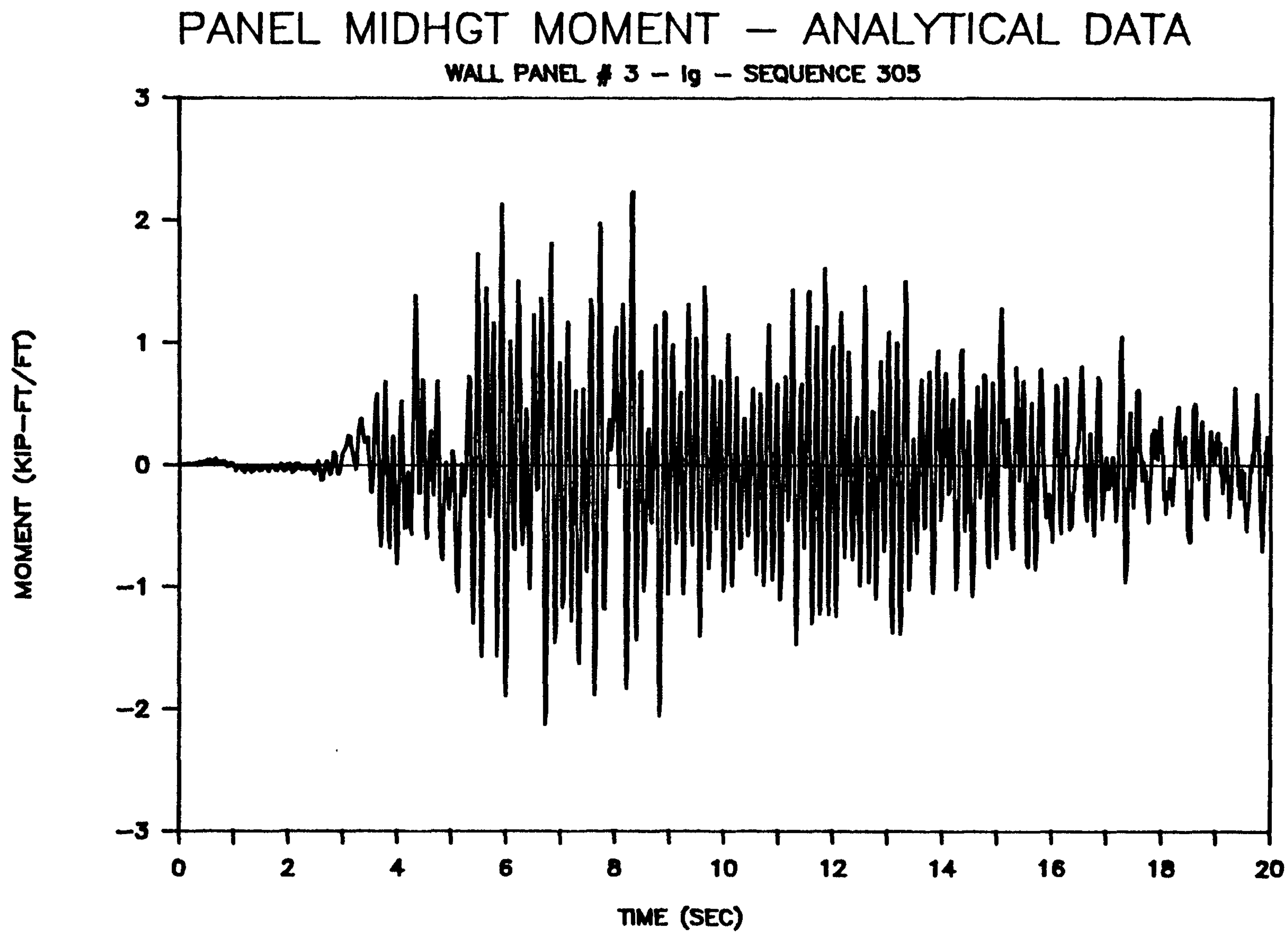


FIGURE 6-14. MIDHEIGHT PANEL MOMENT SIMULATION FOR SEQUENCE 305 FOR ANALYTICAL EXCITATIONS

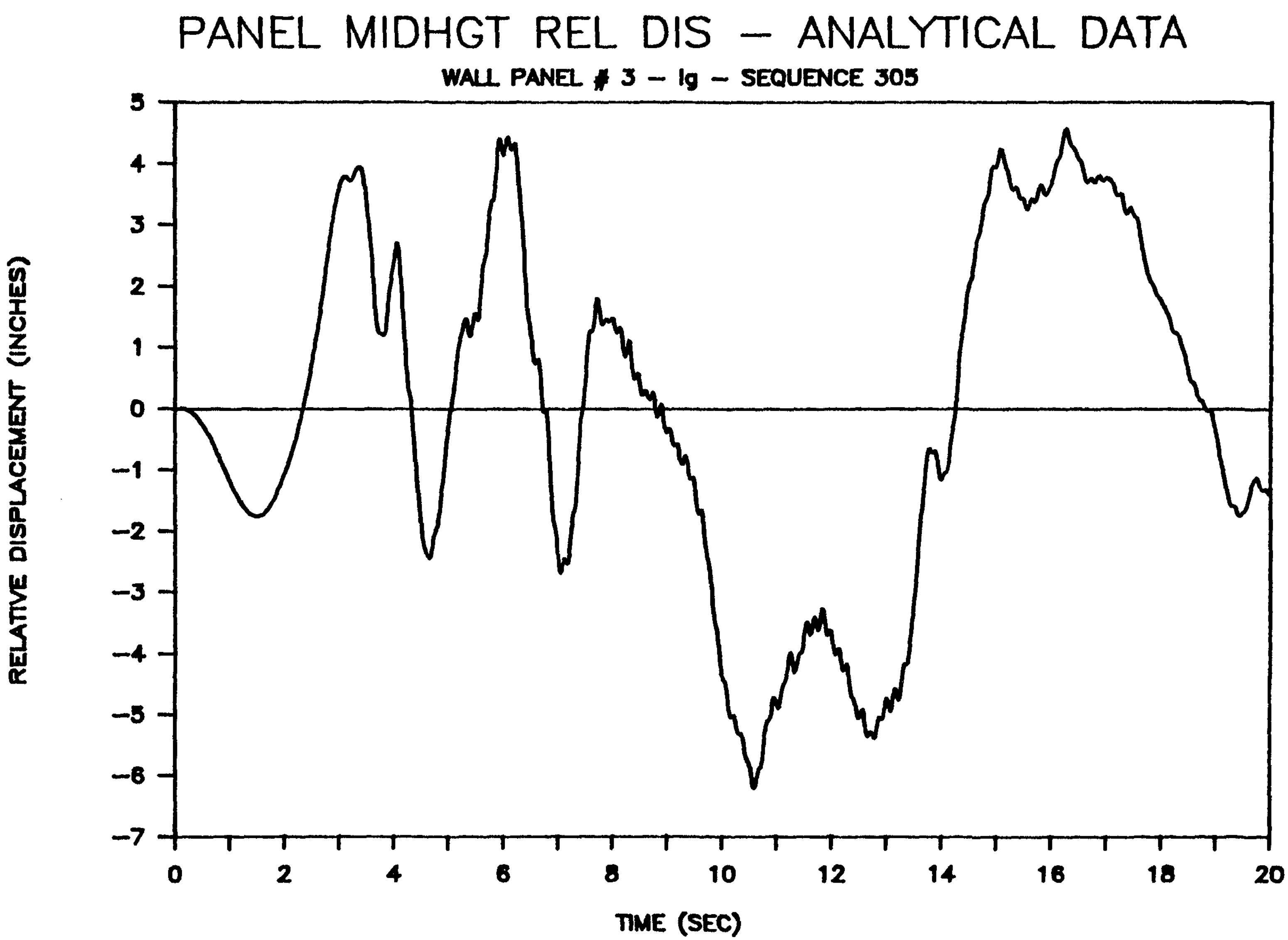


FIGURE 6-15. MIDHEIGHT PANEL RELATIVE DISPLACEMENT SIMULATION FOR SEQUENCE 305 FOR ANALYTICAL EXCITATIONS

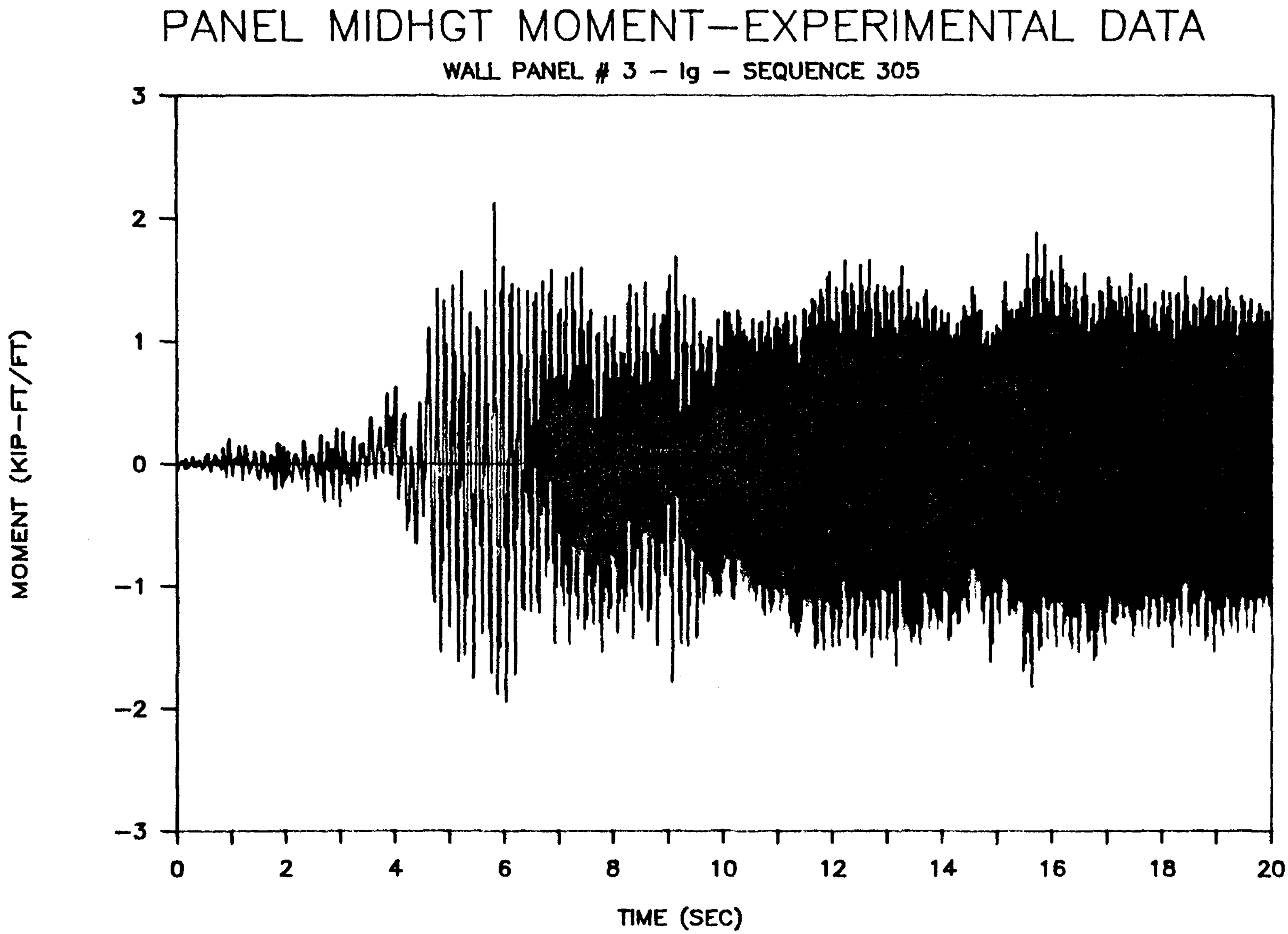


FIGURE 6-16. MIDHEIGHT PANEL MOMENT SIMULATION FOR SEQUENCE 305 FOR EXPERIMENTAL EXCITATIONS (ACTUAL MOTIONS)

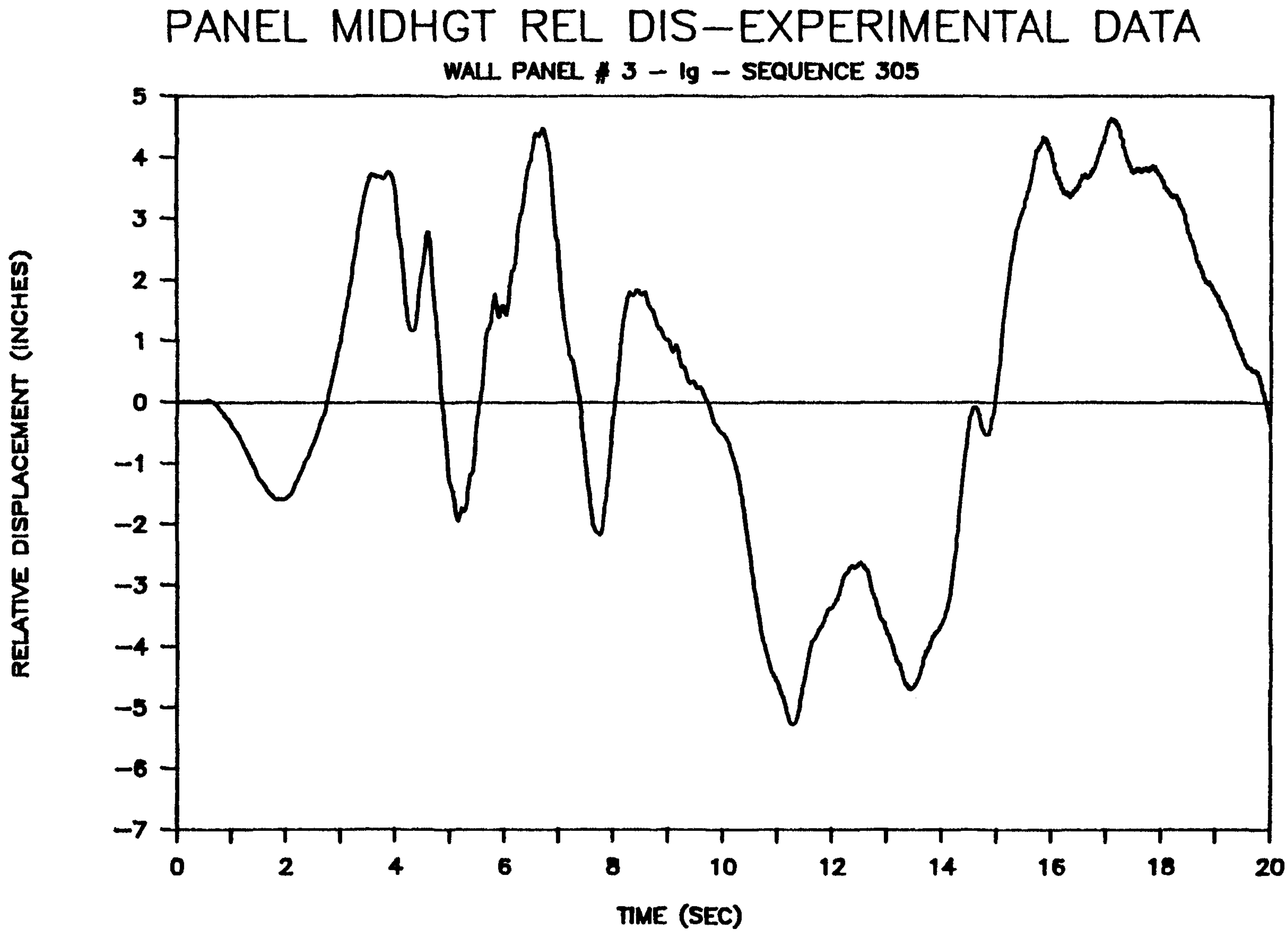


FIGURE 6-17. MIDHEIGHT PANEL RELATIVE DISPLACEMENT SIMULATION FOR SEQUENCE 305 FOR EXPERIMENTAL EXCITATIONS (ACTUAL MOTIONS)

SECTION 7

DISCUSSION

Four masonry block slender wall panels were tested in this experimental program. A summary of the testing sequence and the specimen response is given in Section 5.7. The limited posttest analysis of the digitized data indicated that the top actuator modified the input signal in some tests. It is therefore necessary to use the recorded signals at the base and at the top as the input excitations for any model in the future investigations in order to ensure that the model response simulations will be compatible with the experimental measurements.

The discussions presented herein are based on observations and the study of selected digitized data of panel responses including velocity time history, midheight displacements, and relative displacement time histories.

The test results indicate the following:

1. All panels responded elastically to the first set of input motions, i.e., Sequences 1 through 6.

2. Minor cracks along the mortar joints developed during Sequences 5 and 6, but the cracks did close at the end of each event.

3. Motion M9 which is the El Centro 1940 earthquake motion scaled to 0.4 EPA associated with rigid diaphragm action and a duration of shaking compressed to 12.5 sec, from the original 30 sec, provided the most critical dynamic input for all specimens.

4. Panel #1 was subjected to 18 simulated earthquake shakings. Motion M9 pushed the panel into inelastic range. The midheight peak-to-peak relative displacement during Sequence 18 was 10-3/4". The permanent deformation induced in the panel midheight was about 4-1/2" at the end of Sequence 18. Panel response was elastic up to the end of Sequence 16.

5. Panel #2 was subjected to 15 seismic input motions. Motion M8 at Sequence 11 caused the panel to begin to go into inelastic range. The permanent deformation at panel midheight at the end of Sequence 15 was 3-3/4", with a midheight peak-to-peak relative displacement of 17-1/2" during the testing cycle.

6. The H/t ratio for Panel #2 was 53 which is 23% higher than the H/t of 43 for Panel #1. This caused Panel #2 to be more vulnerable to damage than Panel #1. The development of postelastic behavior in Panel #2 at an earlier stage than Panel #1 is a clear indication of this effect.

7. Panel #3 was subjected to 30 seismic input motions with Motion M9 constituting Sequences 29 and 30 of the input series. At the end of Sequence 30 the panel did not sustain any permanent deformation and was behaving elastically. The midheight peak-to-peak relative displacement during the shaking of Sequence 30 was 8.1".

8. A comparison between Panel #1 and Panel #3 behavior clearly indicates that the partially grouted panel (Panel #3) was more ductile than the fully grouted panel (Panel #1).

9. Panel #4 was subjected to 30 seismic input motions that are identical to those motions applied to Panel #3. The response of Panel #4 remained elastic throughout the sequence of test. Its midheight peak-to-peak relative displacement for Sequence 30 was about 7.9" which is almost identical to response obtained for Panel #3.

10. A comparison between the response of Panel #3 (without reinforcing bars lap splices) and Panel #4 (with reinforcing bars lap splices) indicates that their response is identical. Therefore the effect of such splices on the response of these panels is insignificant.

All the observations made above are based on the comparison of response quantities in time domain. Further comparisons performed in frequency-domain will shed more light on the general behavior of these panels.

The above observations are also based on representative data processed from an extensive experimental data base provided by this program. Further analysis and evaluation of the remainder of the data in future programs will provide a wealth of information on the response of these walls.

SECTION 8

REFERENCES

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APPENDIX A

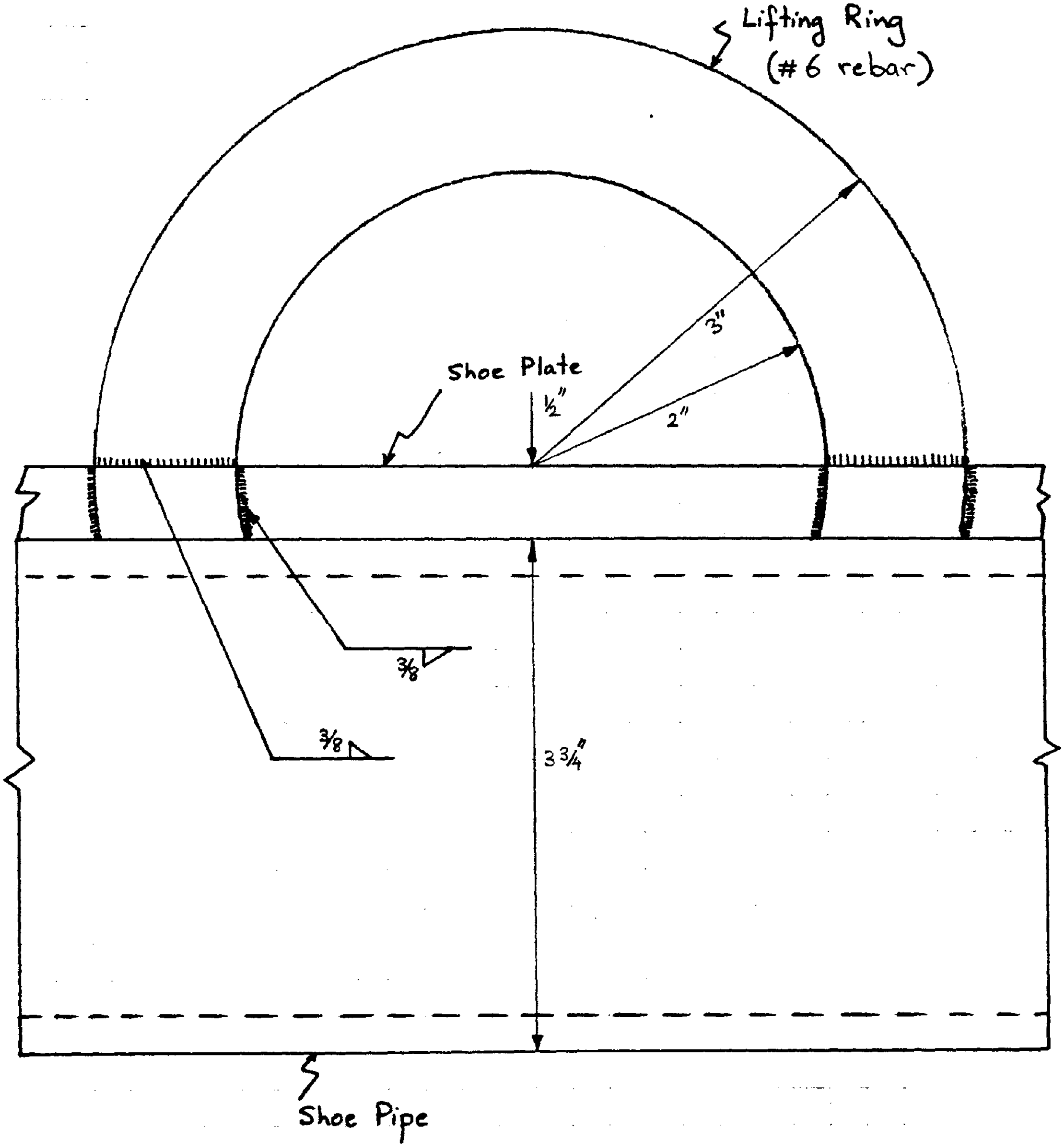
SHOE DETAILING AT THE BASE OF THE WALL PANEL

AGBABIAN ASSOCIATES	SUBJECT Shoe Detailing Specifications
	AUTHOR VA DATE 7/29/87 FILE NO.
	CHECKED BY DATE PAGE 1 OF 9

MATERIAL LIST				
	QTY.	MATERIAL	TYPE	NOTES
A	1	½" PL 7" x 40"	A36	SEE PAGE 4
B	2	¼" PL 1" x 40"	A36	SEE PAGE 4
C	1	3¾" φ TUBE WALL THICK 5/16 IN.	1018	SEE PAGE 4
D	5	#4 REBAR 24" LONG	Grade 60	SEE PAGE 7

Total Weight (without welds) = 90 lbs

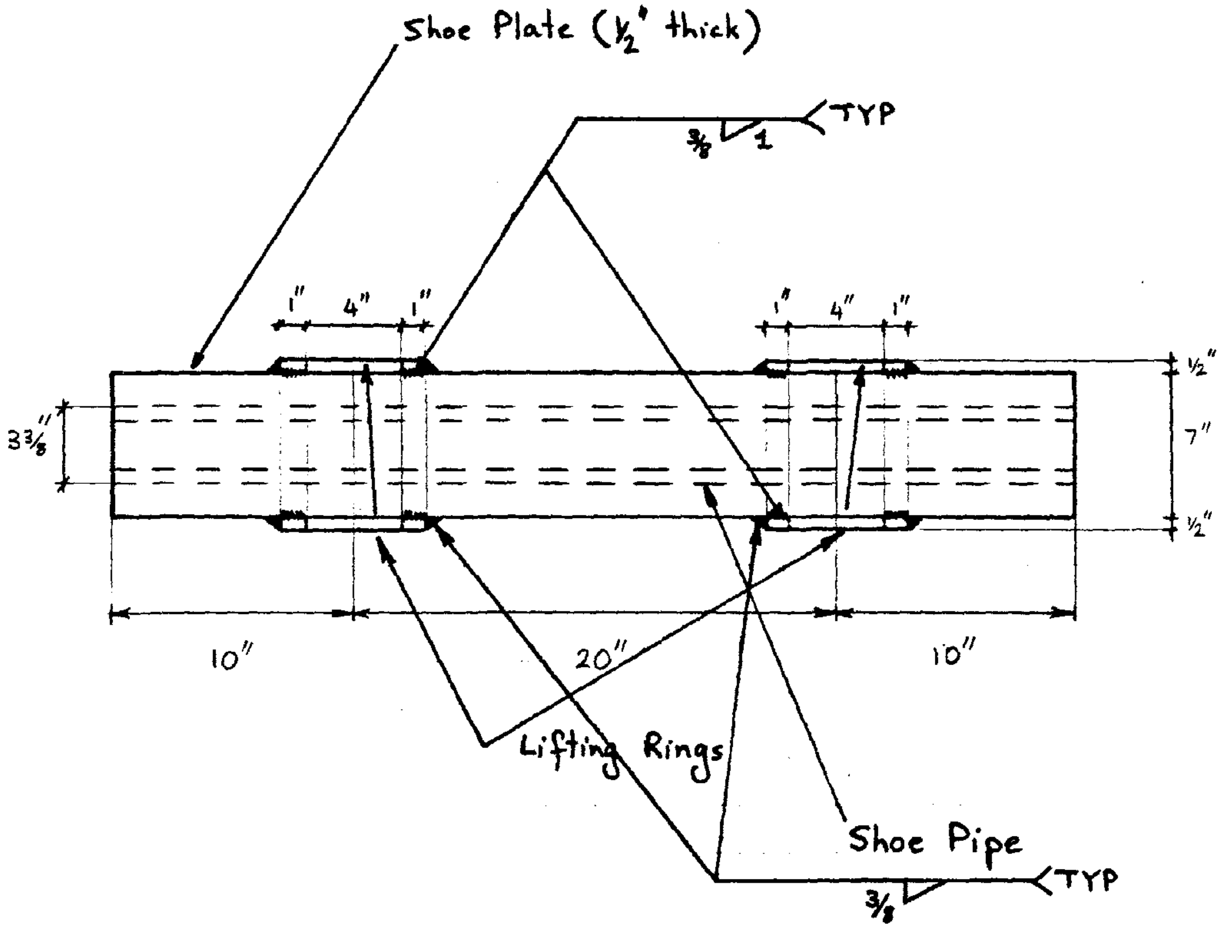
AGBABIAN ASSOCIATES	SUBJECT Shoe Detailing Lifting Ring Detail - Elevation View
	AUTHOR VA DATE 7/22/87 FILE NO.
	CHECKED BY DATE PAGE 2 OF 9



Scale : 1.0" = 1.0"

A3

AGBABIAN ASSOCIATES	SUBJECT Shoe Detailing Lifting Rings & Shoe Plate - Plan View
	AUTHOR VA DATE 7/22/87 FILE NO.
	CHECKED BY DATE PAGE 3 OF 9

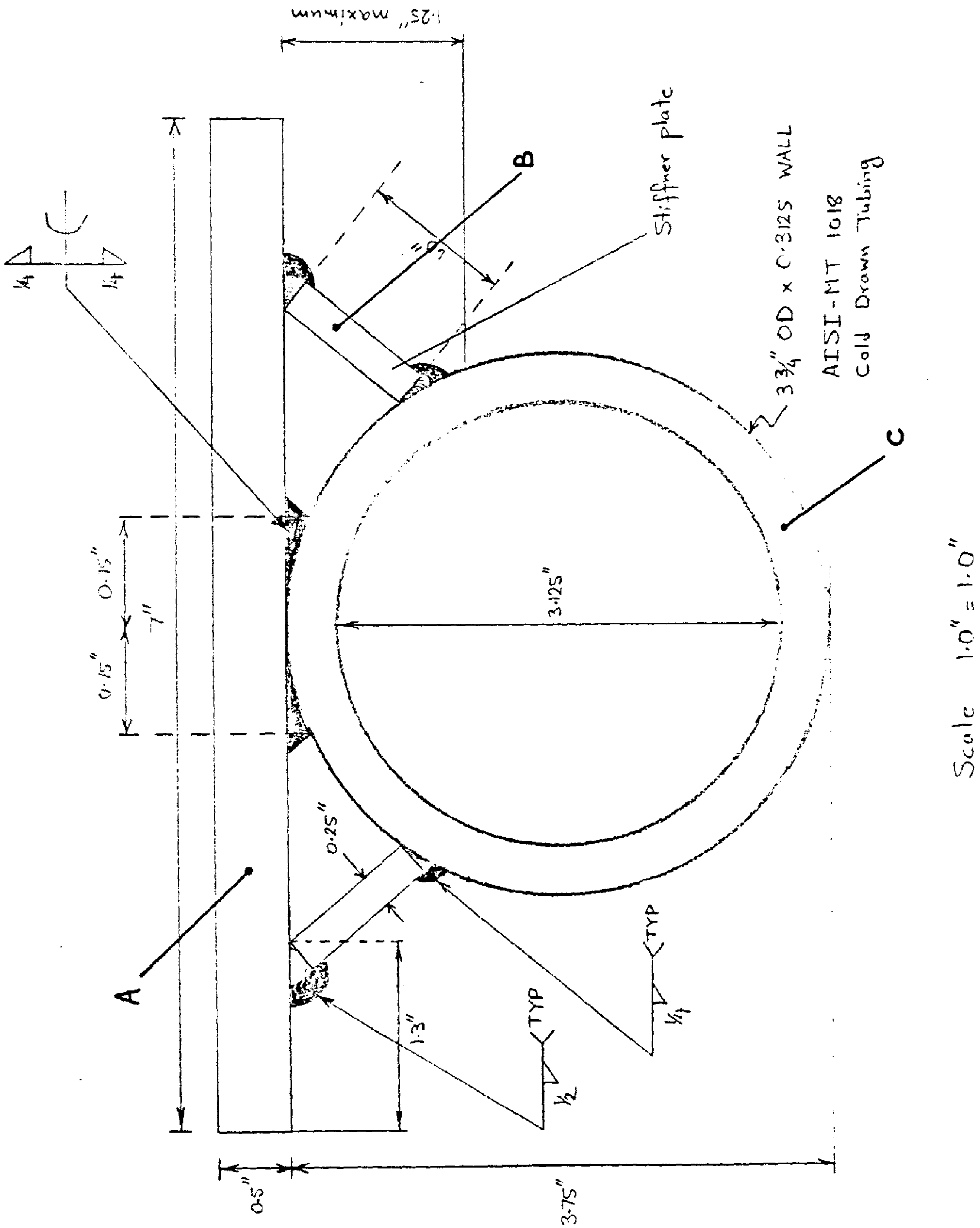


scale: 1/8" = 1.0"

A4

AGBABIAN ASSOCIATES

SUBJECT Shoe Detailing			
Shoe Pipe & Plate - Elevation View			
AUTHOR VA	DATE 7/22/87	FILE NO.	
CHECKED BY	DATE	PAGE 4	OF 9

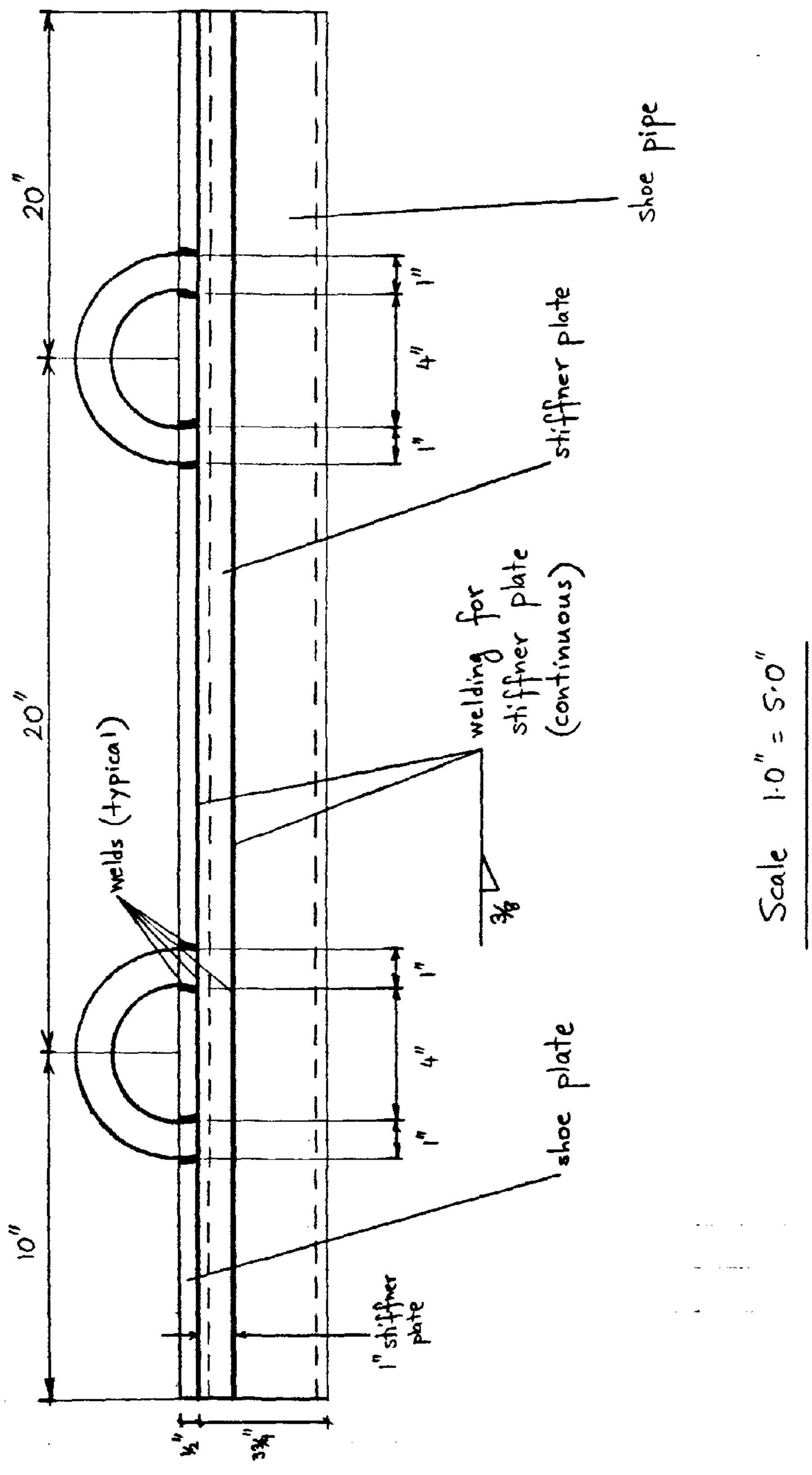


AGBABIAN ASSOCIATES

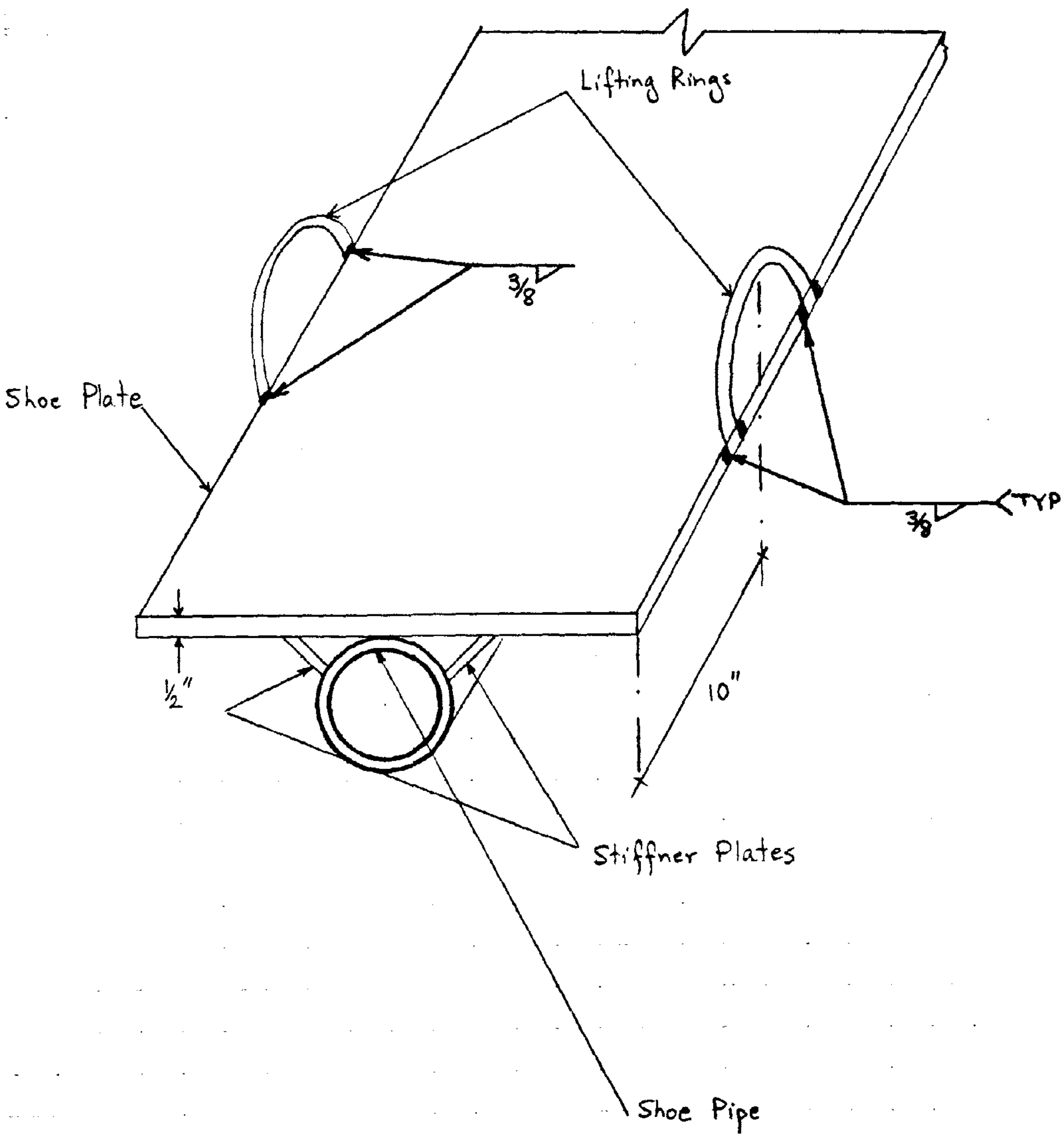
SUBJECT: Shoe Detailing
Lifting Ring & Shoe Assembley - Side View

AUTHOR: VA DATE: 7/22/87 FILE NO.

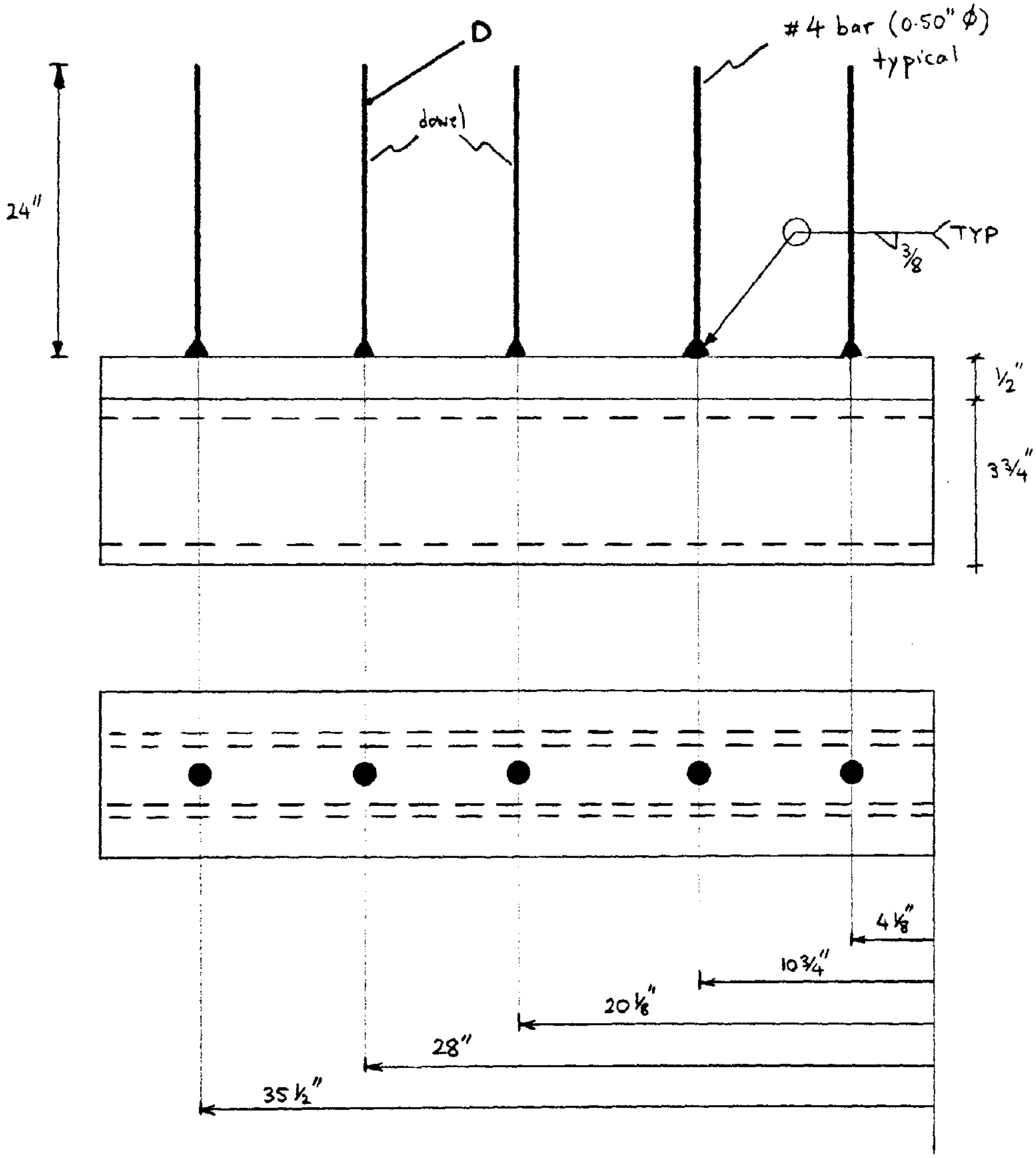
CHECKED BY DATE PAGE 5 OF 9



AGBABIAN ASSOCIATES	
	SUBJECT Shoe Detailing Perspective
	AUTHOR VA DATE 7/22/87 FILE NO.
	CHECKED BY DATE PAGE 6 OF 9



not in scale

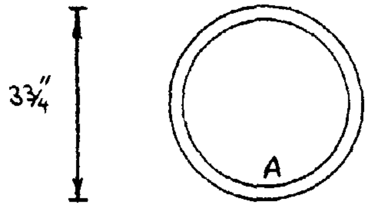

D
#4 bar (0.50" ϕ)
typical
dowel
24"
TYP
3/8
1/2"
3 3/4"
4 1/8"
10 3/4"
20 1/8"
28"
35 1/2"

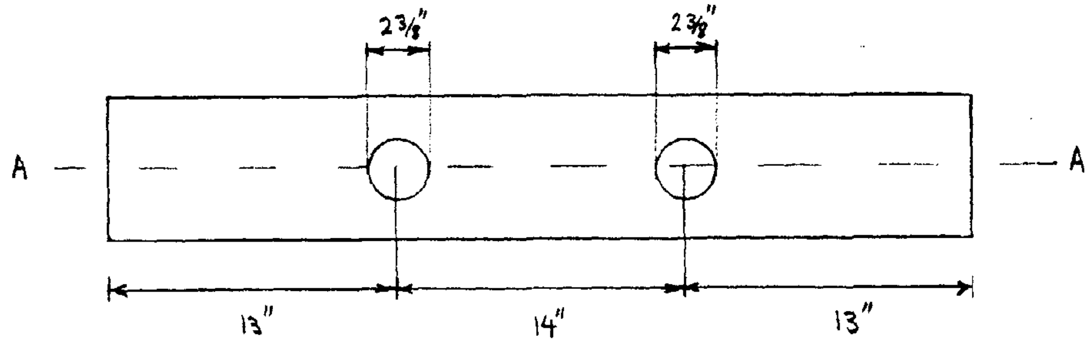
AGBABIAN ASSOCIATES

SUBJECT Shoe Detailing
Pipe Hole Location

AUTHOR YA DATE 7/23/87 FILE NO.

CHECKED BY DATE PAGE 8 OF 9



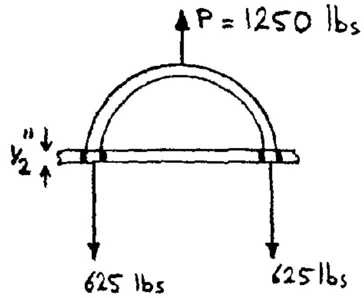


Hole Locations Along the
Shoe Bottom (in Pipe)
For Alignement Pins

Weld strength

Wall weight $\simeq$ 5000 lbs

load on each ring P= 5000/4



1) 1/2" weld

effective area = $(2)\left[(\frac{1}{2} \times .707)(\frac{1}{2})\right] = 0.3535 \text{ in}^2$

$\sigma = \frac{625}{0.3535} = 1768$ psi

allowable shear stress is 13.6 ksi O.K.

2) 3/16" weld

effective area = $(2)\left[(\frac{3}{16} \times .707)(\frac{1}{2})\right] = .1326 \text{ in}^2$

$\sigma = \frac{625}{.1326} = 4715$ psi O.K.

3) 3/8" weld

effective area = $(2)\left[(\frac{3}{8} \times .707)(\frac{1}{2})\right] = 0.2651 \text{ in}^2$

$\sigma = \frac{625}{.2651} = 2357$ psi O.K.

select 3/8" weld

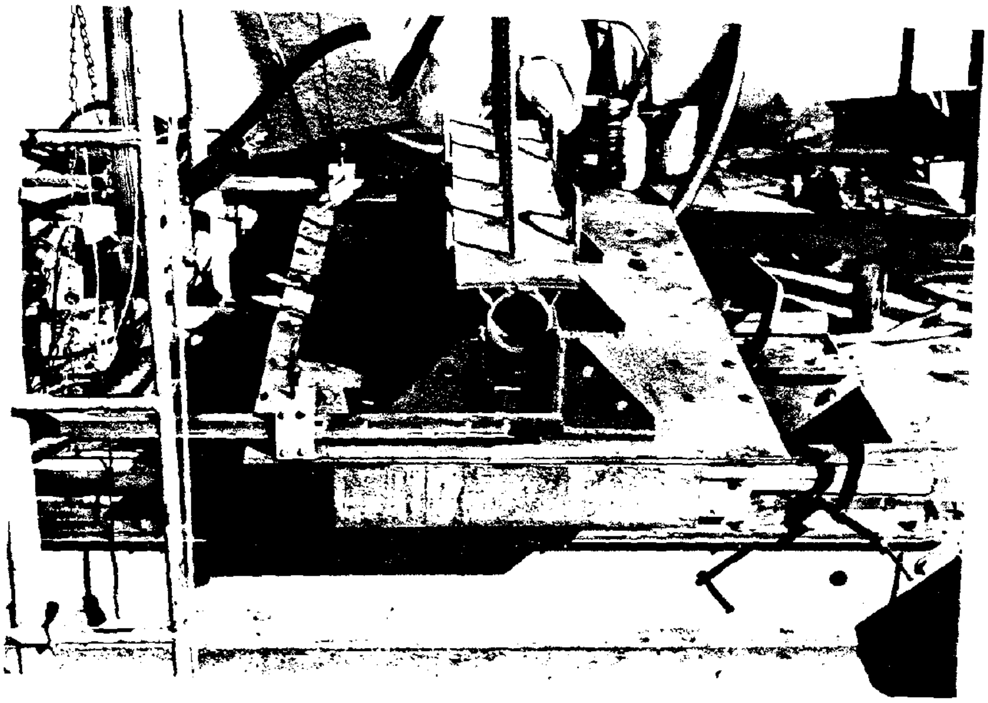
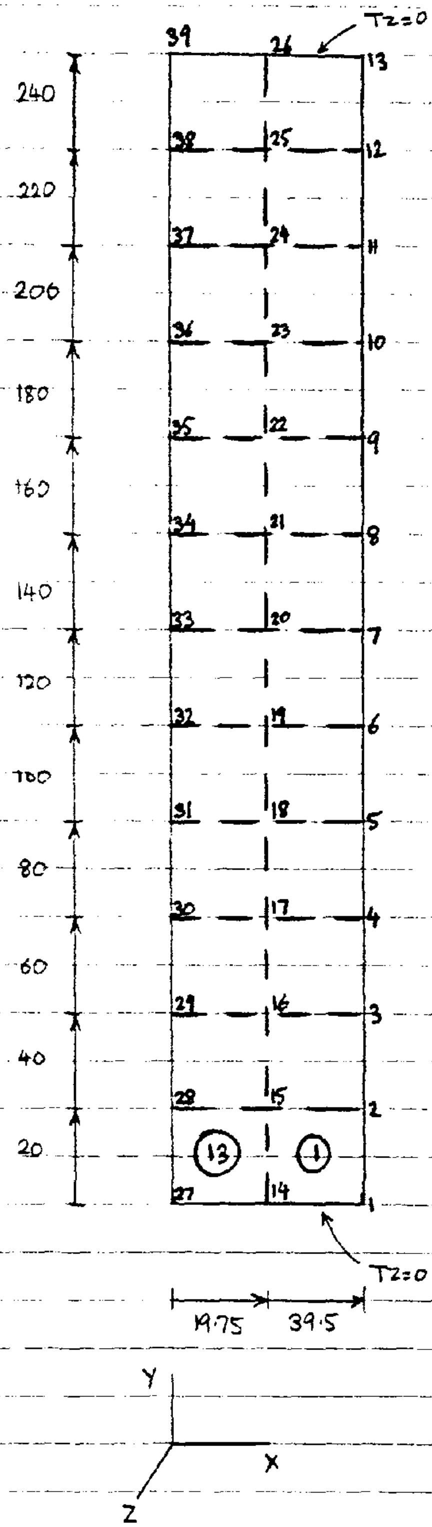


FIGURE A-1. PANEL SHOE POSITIONED ON
THE SHAKING TABLE

APPENDIX B

EIGENVALUE CALCULATIONS FOR WALL SPECIMENS

Eigenvalues of the Concrete Masonry Wall Specimens



Max. Compressive Strength of CMU = 2500 psi

Min. Compressive Strength of wall = 1625 psi

$E = 1000 f'_m$

$\nu = 0.2$

$G = 677.1$ ksi

partially grouted:

$\gamma = 105 \ lb/ft^3$, $\rho = 1.5742 \times 10^{-7} \frac{k\text{-}sec^2}{in^4}$

fully grouted:

$\gamma = 128 \ lb/ft^3$, $\rho = 1.9190 \times 10^{-7} \frac{k\text{-}sec^2}{in^4}$

TX = 0 TY = 0 TZ ✓

RX ✓ RY ✓ RZ = 0

Partially Grouted ($\gamma = 105$ lb/ft^3)

$f_m = 2500$ psi $\quad t = 5.625''$

$\rho = 1.5742 \times 10^{-7}$ k-sec^2/in^4

	$f'_m = 1625$ PSI	$f'_m = 2000$ PSI	$f'_m = 2500$ PSI
f_1 (Hz)	4.498	4.990	5.579
f_2 (Hz)	17.98	19.94	22.30
f_3 (Hz)	39.08	43.35	48.47

Fully Grouted ($\gamma = 128$ lb/ft^3)

$f_m = 2500$ psi $\quad t = 5.625''$

$\rho = 1.9190 \times 10^{-7}$ k-sec^2/in^4

	$f'_m = 1625$ PSI	$f'_m = 2000$ PSI	$f'_m = 2500$ PSI
f_1 (Hz)	4.074	4.520	5.053
f_2 (Hz)	16.28	18.06	20.20
f_3 (Hz)	35.39	39.26	43.90

Partially Grouted ($\gamma = 105$ lb/ft^3)

$f_m = 2500$ psi $\quad t = 4.5''$

$\rho = 1.5742 \times 10^{-7}$ k-sec^2/in^4

	$f'_m = 1625$ PSI	$f'_m = 2000$ PSI	$f'_m = 2500$ PSI
f_1 (Hz)	3.599	3.993	4.464
f_2 (Hz)	14.40	15.97	17.86
f_3 (Hz)	31.26	34.68	38.77

Fully Grouted ($\gamma = 128$ lb/ft^3)

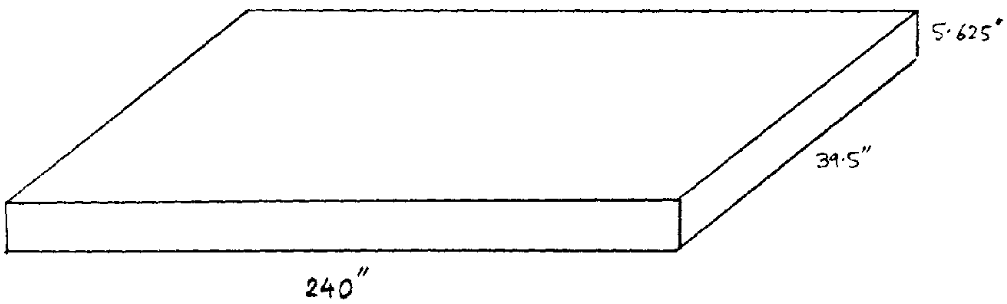
$f_m = 2500$ psi $\quad t = 4.5''$

$\rho = 1.9190 \times 10^{-7}$ k-sec^2/in^4

	$f'_m = 1625$ PSI	$f'_m = 2000$ PSI	$f'_m = 2500$ PSI
f_1 (Hz)	3.260	3.617	4.044
f_2 (Hz)	13.04	14.46	16.17
f_3 (Hz)	28.31	31.41	35.12

AGBABIAN ASSOCIATES

SUBJECT: CMU Wall Natural Frequency
5-5/8" wall, partially grouted
AUTHOR: VA DATE: 4/27/88 FILE NO.
CHECKED BY: DATE: PAGE: OF:



$$I = \frac{bd^3}{12} = \frac{(39.5)(5.625)^3}{12} = 585.85 \ \text{in}^4$$

$$v(x) = a_o \sin\frac{\pi x}{l} \qquad \frac{dv}{dx} = a_o \frac{\pi}{l}\cos\frac{\pi x}{l} \qquad \frac{d^2v}{dx^2} = -a_o\left(\frac{\pi}{l}\right)^2 \sin\frac{\pi x}{l}$$

$$V = \frac{1}{2}\int_0^l EI\left(\frac{d^2v}{dx^2}\right)^2 dx = \frac{1}{2}\int_0^l EI\, a_o^2\left(\frac{\pi}{l}\right)^4 \sin^2\frac{\pi x}{l}\, dx$$

$$V = \frac{1}{2} EI a_o^2 \left(\frac{\pi}{l}\right)^4 \left[\frac{l}{2}\right] = \frac{\pi^4}{4} a_o^2 \frac{EI}{l^3}$$

$$m(x) = \rho(x) A = 222.2\rho$$

$$T = \frac{1}{2}\int_0^l m(x)\dot{v}^2 dx = \frac{1}{2}\omega^2 \int_0^l \rho A a_o^2 \sin^2\frac{\pi x}{l}\, dx$$

$$T = \frac{1}{2}\omega^2 \rho A a_o^2 \left[\frac{l}{2}\right] = \frac{\omega^2}{4}\rho A l a_o^2$$

$$T = V \qquad \frac{\omega^2}{4}\rho A l a_o^2 = \frac{\pi^4}{4} a_o^2 \frac{EI}{l^3}$$

$$\omega^2 = \frac{\pi^4 EI}{\rho A l^4} = \frac{\pi^4 (2000)(585.85)}{1.5742\times10^{-7} \times 222.2 \times (240)^4}$$

$$f = 4.991 \ \text{Hz}$$