INTERACTION BETWEEN BRICK MASONRY BUILDINGS AND THEIR ROOF DIAPHRAGMS DURING EARTHQUAKES

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ABSTRACT Wood and other types of diaphragms supported on brick masonry walls are found in numerous buildings in various seismic zones. Such diaphragms have suffered little or no damage during past earthquakes. However, the supporting brick masonry walls have suffered degrees of damage varying from minor tension cracks to separation from the diaphragms and complete collapse. A lumped parameter dynamic analysis of wood diaphragms was conducted and correlated with full-scale dynamic tests of diaphragms. The analytical model accounted for the hysteretic nonlinear response of the diaphragm. The results of the analytical/ experimental program indicated that the seismic response of a wood diaphragm is highly nonlinear and its response directly affects the masonry wall damage.

1. INTRODUCTION

Wood roof diaphragms supported on brick masonry walls are found in numerous buildings located in various seismic zones. Reports of damage from past earthquakes indicate that such diaphragms have suffered little or no damage during earthquake excitations. However, the supporting brick masonry walls have suffered degrees of damage varying from minor tension cracks to separation from the diaphragms and complete collapse (Figs. 1 and 2). The diaphragm's response and masonry wall damage are directly related because of the effect of diaphragm deformation on the out-of-plane motions induced in the masonry walls.

Studies by Blume et al. (1) on full-scale school buildings indicated that (a) wood roof diaphragms had periods ranging between 0.17 and 0.75 sec; and (b) to reduce the response of these buildings to earthquake motions, roof diaphragms should be stiffened. However, these results were obtained from low-amplitude vibration tests and therefore are limited to low magnitude earthquake shaking.

Static tests of full-scale lumber and plywood-sheathed diaphragms were conducted by several investigators (2, 3). The results of these tests indicate that the behavior of these diaphragms is highly nonlinear. Therefore, these diaphragms would exhibit longer periods when subjected to high intensity earthquake shaking.

This paper represents part of a study that combined analytical and experimental investigations to develop a methodology for the mitigation of seismic hazards in existing unreinforced masonry buildings (URM) located in various seismic zones of the United States (4).

A lumped parameter analytical model is used in this paper for the dynamic analysis of a typical one-story unreinforced masonry building with wood roof diaphragms (5,6). The analyses provide quantitative estimates of the out-of-plane excitations induced in the masonry walls. The analytical model includes a nonlinear hysteretic element that simulates the behavior observed in cyclic tests of wood diaphragms. The results of the nonlinear dynamic analyses are compared with corresponding results obtained with linear elastic and nonlinear elastic assumptions and results obtained from a companion test program. In the test program, diaphragms, typical of those found in existing structures, were

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FIGURE 1. ROOF TRUSS BEATING OF PARAPET; MARKET, MACKAY (IDAHO, 1983)

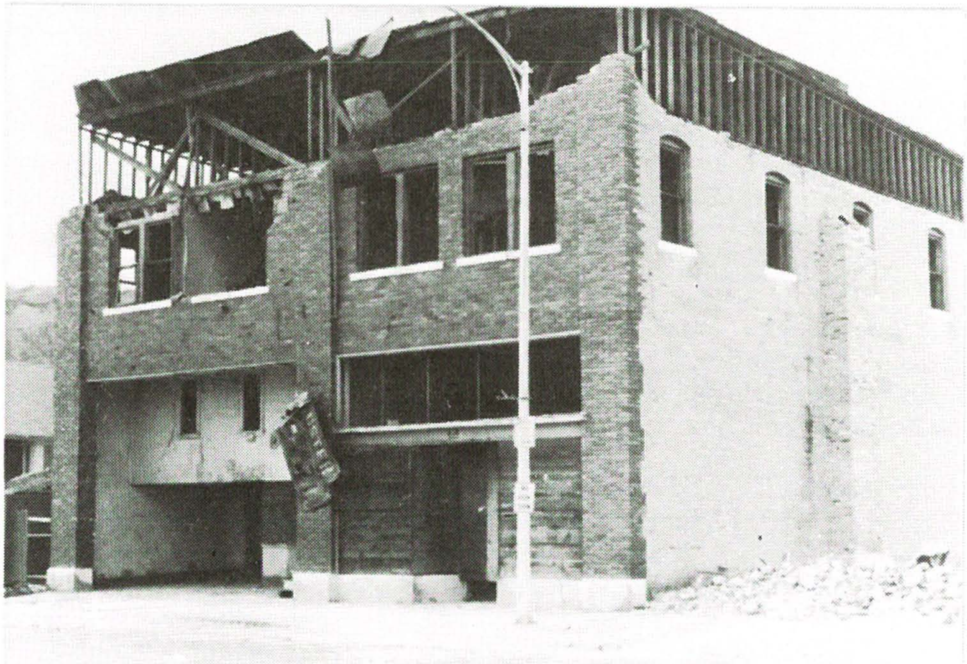


FIGURE 2. ROOF TRUSS MOVEMENT KNOCKED HIGH PARAPETS; CUSTER HOTEL, MACKAY (IDAHO, 1983)

tested. The tests were conducted on 14 configurations of 6.1 m by 18.3 m (20 ft by 60 ft) diaphragms (including repaired and retrofitted specimens) by static, in-plane displacement and by dynamic, in-plane shaking. These tests were conducted to (a) obtain stiffness characteristics, (b) assess the degree of nonlinearity, (c) study dynamic responses under earthquake loading, (d) assess the effect of various kinds of retrofit strengthening, (e) verify and calibrate the above analytical model that has been developed for the dynamic analysis of diaphragms, and (f) determine the required anchorage forces of brick masonry walls to diaphragms.

While these tests were performed under a program of seismic hazard mitigation for existing URM buildings, they were carried out over a broader range of systems than is normally found in URM buildings. Some of the data, then, is applicable to new construction that uses these systems.

2. ANALYTICAL MODEL

A typical one-story masonry building, shown in Figure 3, was selected for analysis. The building consists of a wood roof diaphragm supported on four edges by masonry walls. The earthquake ground motion was assumed to be directed normal to the side wall. The analysis reported here is primarily concerned with the response of the diaphragm and its effect on the out-of-plane motions induced in the masonry side walls. A previous study (5) indicated that the end walls transmit the ground motion from the foundation level (C and D, Fig. 4a) to the roof level (A and B, Fig. 4b) with little modification. The masonry side walls were assumed to crack when subjected to the out-of-plane excitation resulting from the earthquake motions, and the mass of the side walls is included in the model as shown in Figure 4b.

The diaphragm was idealized as a deep shear beam and was divided into several shear segments (eight segments are shown in Figure 4b) with a mass at each segment interface (node). Each nodal mass represents the lumped tributary mass of the appropriate diaphragm segment as well as the tributary mass of the side walls. The earthquake motions at each end were identical, so due to symmetry only a half model was analyzed. The resulting lumped parameter model is shown in Figure 5.

Internal springs 1 through 4, shown in Figure 5, represent the nonlinear shear stiffness of the diaphragm, and internal dampers 5 through 8 represent viscous damping of the roofing material. The earthquake input motion at the end wall is described by coordinate X_1, and the independent degrees of freedom (DOF) of the diaphragm/wall system are described by coordinates X_2 through X_5. Displacement gages 9 through 12 shown connecting the DOF X_2 through X_5 with DOF X_1 are used to monitor the relative out-of-plane motions induced between the top and bottom of the side wall. This relative motion can be used in a separate stability analysis of the side walls. The analyses were performed using the STARS/III computer program (7).

Based on the available test results, a nonlinear hysteretic material model for the wood diaphragms has been developed as shown in Figure 6. In this model, the total deformation mechanism of the diaphragm is smeared into a nonlinear, hysteretic shear element. The overall force-deflection envelope is shown in Figure 6a, and a typical cyclic load path for the spring is shown in Figure 6b.

3. ANALYSIS OF TYPICAL ROOF DIAPHRAGMS

Dynamic analyses using the model described in the previous section were conducted for two types of earthquake ground motions. The inputs correspond to the highest seismic areas of the United States where effective peak acceleration is 0.40 g (8). The S00E component of the 1940 El Centro record was scaled (1.25

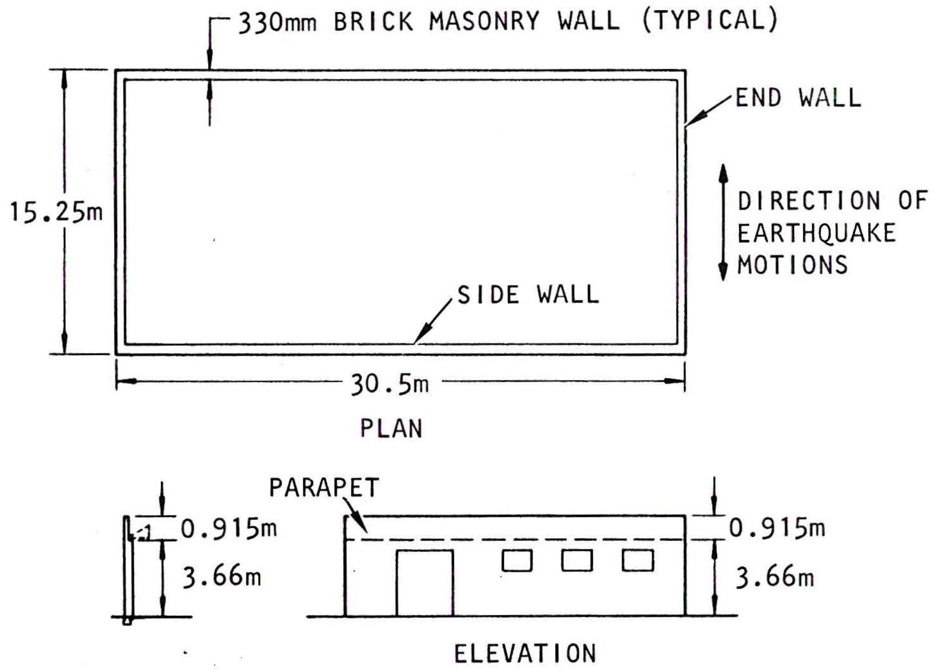


FIGURE 3. TYPICAL ONE-STORY UNREINFORCED MASONRY BUILDING WITH A WOOD ROOF DIAPHRAGM

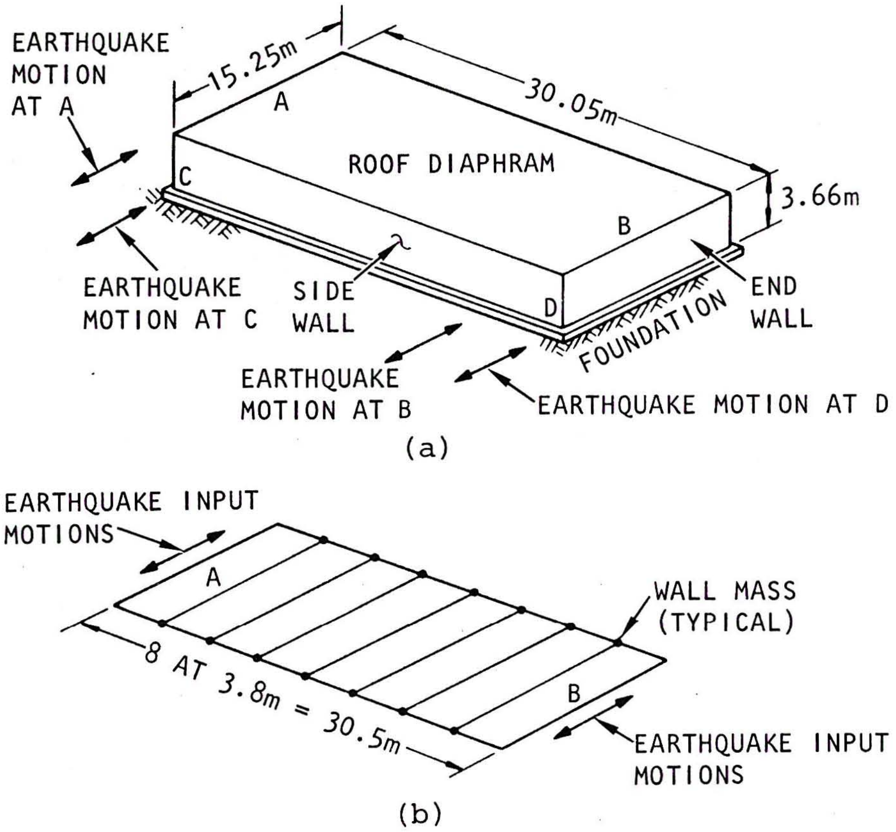


FIGURE 4. DIAPHRAGM/WALL CONFIGURATION AND ANALYSIS MODEL

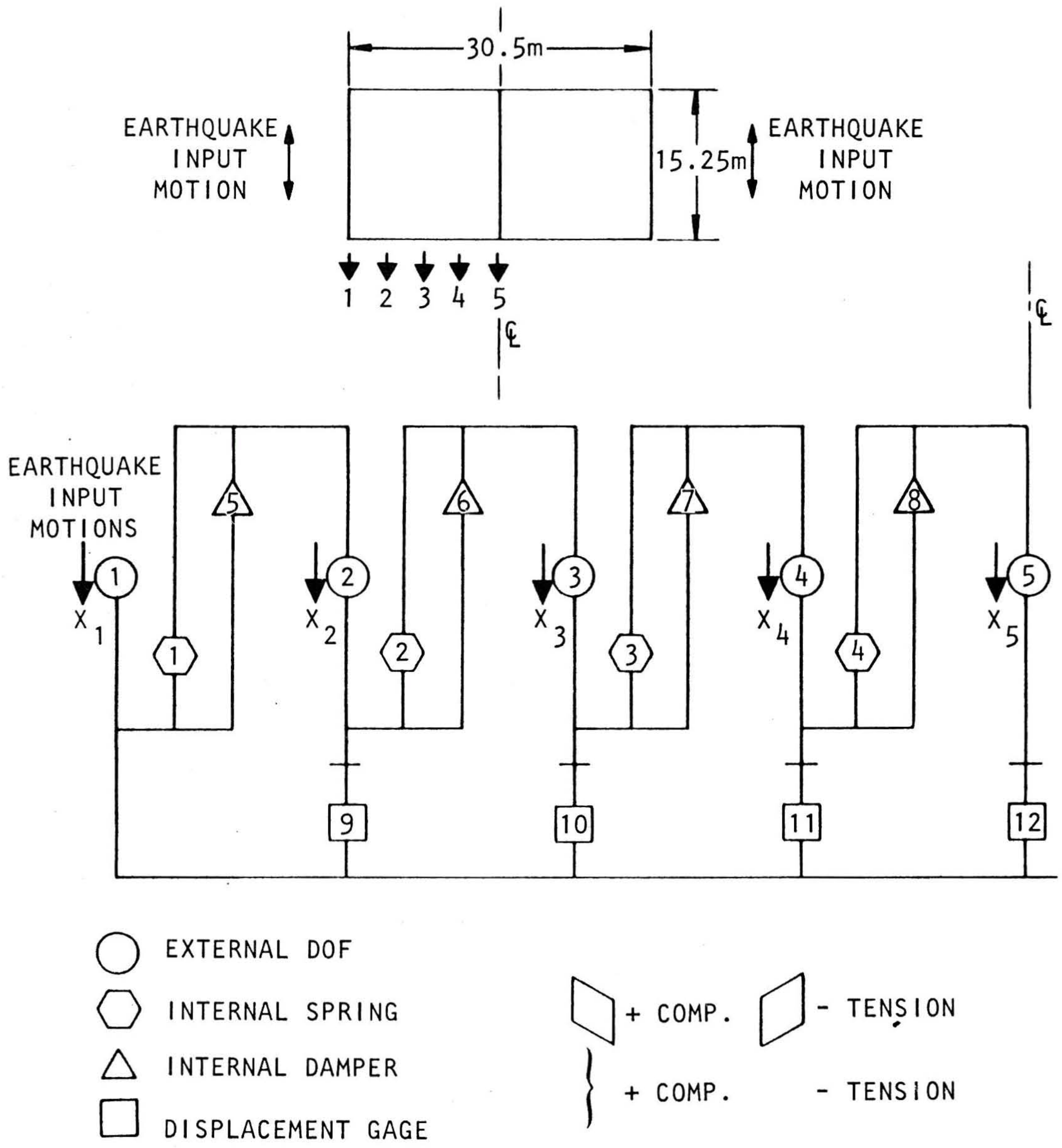
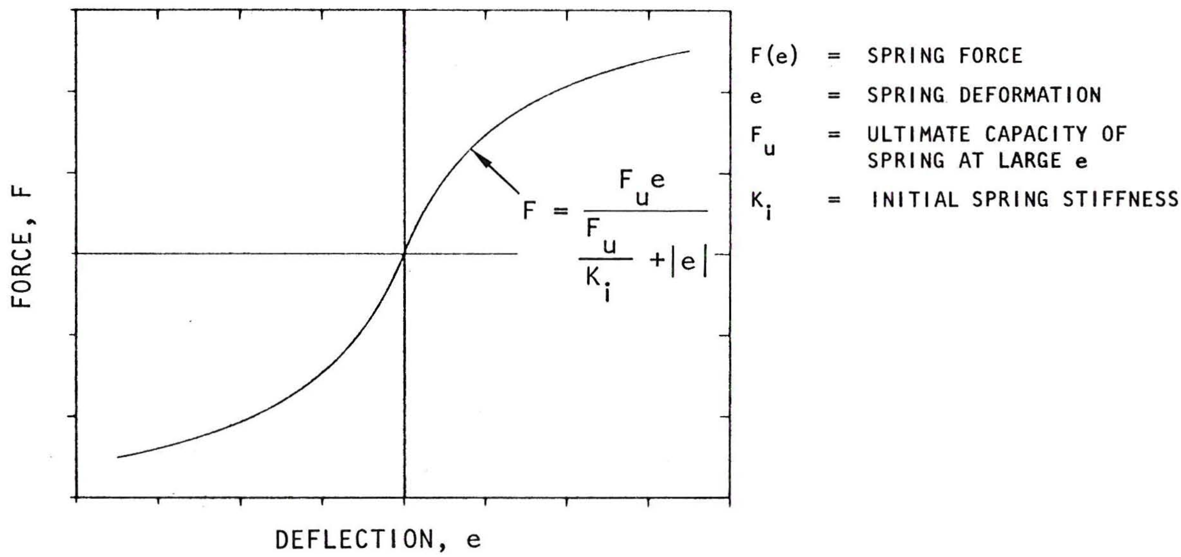
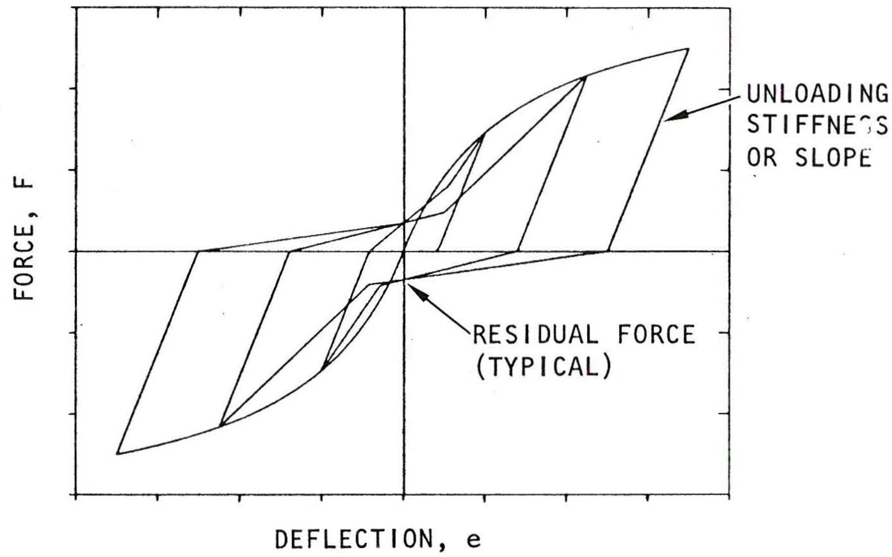


FIGURE 5. LUMPED PARAMETER MODEL (8 SEGMENTS – HALF MODEL)



(a) Force-deflection envelope of model



(b) Typical cyclic load-deflection diagram for model

FIGURE 6. LOAD DEFLECTION MODEL FOR WOOD DIAPHRAGMS

scale factor) to the 0.40-g level. The N69W component of the 1971 Castaic record was scaled (1.80 scale factor) to the 0.40-g level. The same trend was observed for analyses cases of both plywood and diagonal sheathed diaphragms. Therefore, for the diagonal sheathing, only the results of linear elastic and nonlinear hysteretic are given in Table 1.

The nonlinear elastic cases show a general reduction (relative to the linear elastic cases) in peak responses at the midlength of the diaphragm (DOF 5), an attenuation of the peak relative deformation at the midlength (Spring No. 12), and significant reductions in the maximum shear force at the end wall (Spring No. 1). The nonlinear hysteretic cases show further significant reductions in all response quantities.

The relative deformation between the top and bottom of the masonry wall at the diaphragm midlength (i.e., Spring No. 12) gives an indication of the out-of-plane motions induced in the side walls. Typical results for this relative deformation are given in Figure 7. It was concluded that elastic analyses overestimate the actual response and that consideration of the hysteretic characteristics of the wood diaphragms is necessary for more realistic response predictions.

4. DIAPHRAGM TESTS

The diaphragms tested consisted of the following configurations.

- 20-ga steel decking, unfilled, unchorded, button-punched seams .46 m (18") O.C.
- 20-ga steel decking, unfilled, chorded, button-punched seams 0.15 m (6") O.C.
- 12.7 mm (1/2") plywood, unblocked, unchorded, built-up roofing
- 12.7 mm (1/2") plywood, unblocked, chorded, built-up roofing, roofing retrofit nailing
- 12.7 mm (1/2") plywood, unblocked, chorded
- 25.4 mm x 152 mm (1" x 6") straight sheathing, unchorded, built-up roofing
- 25.4 mm x 152 mm (1" x 6") straight sheathing, unchorded, built-up roofing, roofing retrofit nailing
- 25.4 mm x 152 mm (1" x 6") straight sheathing, 7.9 mm (5/16") plywood overlay, chorded
- 25.4 mm x 152 mm (1" x 6") diagonal sheathing, unchorded, built-up roofing
- 25.4 mm x 152 mm (1" x 6") diagonal sheathing, unchorded, built-up roofing, roofing retrofit nailing
- 25.4 mm x 152 mm (1" x 6") diagonal sheathing, 25.4 mm x 152 mm (1" x 6") straight sheathing overlay, chorded
- 12.7 mm (1/2") plywood, blocked, chorded
- 19 mm (3/4") plywood, 19 mm (3/4") plywood overlay, blocked, chorded
- 20-ga steel decking, 63.5 mm (2-1/2") concrete fill, chorded, button-punched seams .46 m (18") O.C.

The test setups for the dynamic, in-plane shaking and the quasi-static, cyclic, in-plane displacement of the diaphragms are shown schematically in Figures 8 and 9, respectively (9). The diaphragm specimens were supported on low friction,

TABLE 1. SUMMARY OF DIAPHRAGM DYNAMIC ANALYSIS RESULTS

Diaphragm	Earthquake Input	Material Model[6]			Peak Acceleration at Midlength DOF 5, g	Peak Velocity at Midlength DOF 5, m (in./sec)	Peak Displacement at Midlength DOF 5, m (in.)	Peak Relative Deformation at Midlength Spring 12, m (in.)[5]	Peak Shear Force at End Wall Spring 1, $N \times 10^3$ (kips)[5]
		Type	K_1 $N \times 10^3$/mm (kips/in.)	F_u $N \times 10^3$ (kips)					
Plywood[1]	Scaled[2] El Centro	Linear Elastic	4.4 (25)	-	2.4	2.85 (112.0)	0.51 (20.0)	0.39 (15.5)	814.35 (183.0)
		Nonlinear Elastic		623 (140)	1.48	1.72 (67.7)	0.44 (17.4)	0.36 (14.2)	339.535 (76.3)
		Nonlinear Hysteretic		623 (140)	0.62	0.9 (35.3)	0.18 (7.2)	0.15 (6.0)	202.92 (45.6)
		Nonlinear Hysteretic		321 (70)	0.45	0.69 (27.3)	0.21 (8.4)	0.15 (5.9)	141.07 (31.7)
	Scaled[3] Castaic	Linear Elastic	4.4 (25)	-	2.2	2.49 (98.2)	0.44 (17.4)	0.35 (13.7)	733.7 (166.0)
		Nonlinear Elastic		623 (140)	2.0	2.00 (77.5)	0.33 (13.0)	0.28 (10.9)	321.74 (72.3)
		Nonlinear Hysteretic		623 (140)	0.53	0.90 (35.4)	0.20 (8.0)	0.15 (6.1)	232.74 (52.3)
		Nonlinear Hysteretic		321 (70)	0.41	0.78 (30.5)	0.21 (8.3)	0.14 (5.4)	162.43 (36.5)
Diagonal[4] Sheathing	Scaled[2] El Centro	Linear Elastic	1.45 (8.3)	-	1.8	2.63 (103)	0.65 (25.4)	0.60 (23.6)	394.28 (88.6)
		Nonlinear Hysteretic		534 (120)	0.50	1.11 (43.6)	0.30 (12.0)	0.34 (13.5)	154.42 (34.7)
	Scaled[3] Castaic	Linear Elastic	1.45 (8.3)	-	1.66	2.33 (91.8)	0.48 (19.0)	0.45 (17.9)	342.21 (76.9)
		Nonlinear Hysteretic		534 (120)	0.48	0.92 (36.4)	0.30 (11.7)	0.24 (9.4)	117.93 (26.5)

[1] 12.7 mm (1/2 in.) plywood, unblocked, roofed
[2] El Centro, 1940, S00E, scaled by 1.25
[3] Castaic, 1971, N69W, scaled by 1.80
[4] Diagonal sheathing, roofed
[5] See Figure 5
[6] See Figure 6

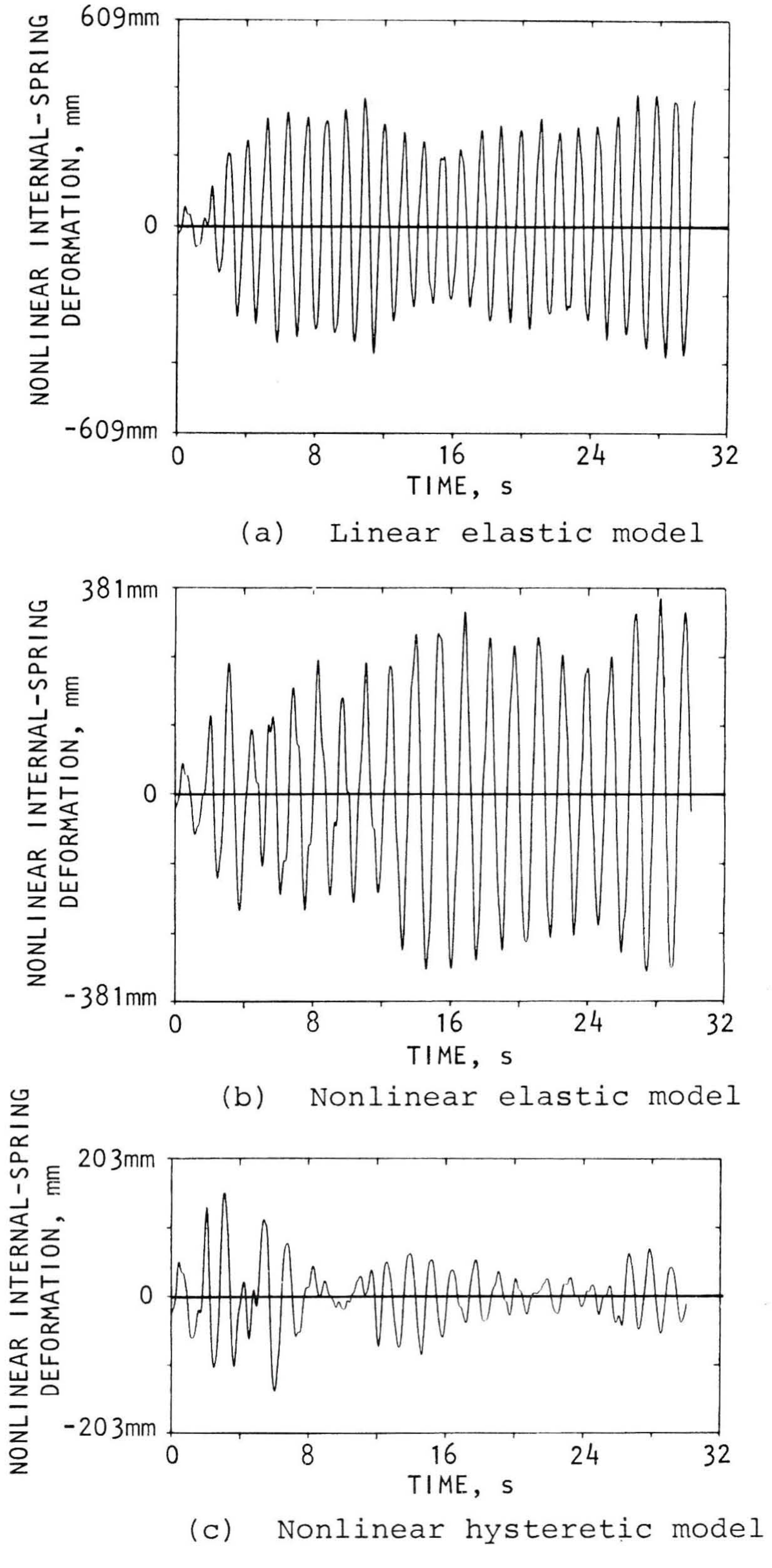


FIGURE 7. RELATIVE DEFORMATION BETWEEN TOP AND BOTTOM OF THE MASONRY WALL AT PLYWOOD DIAPHRAGM MIDLENGTH, EL CENTRO INPUT

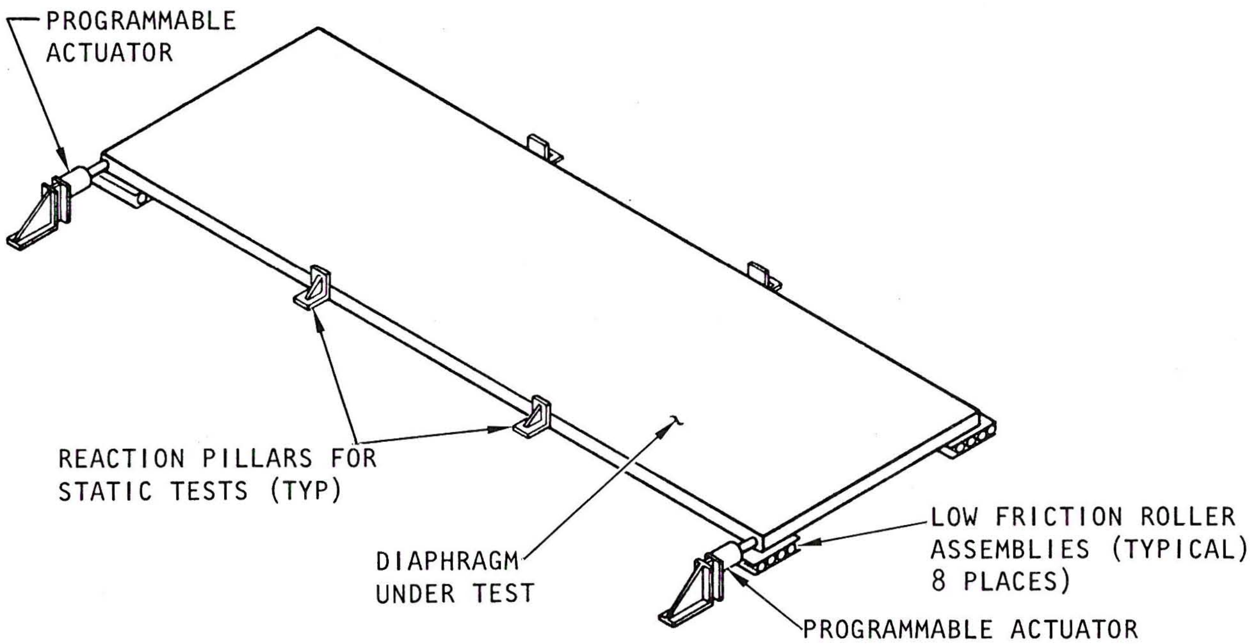


FIGURE 8. TEST SETUP FOR QUASI-STATIC TESTING OF DIAPHRAGMS

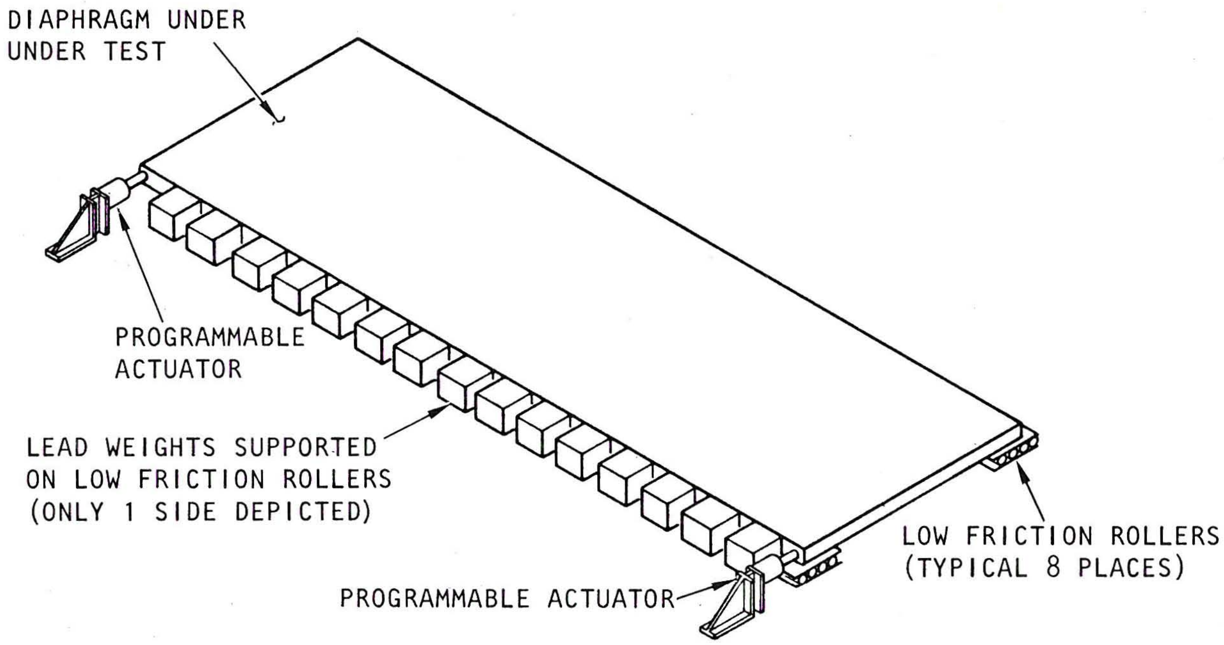


FIGURE 9. TEST SETUP FOR DYNAMIC TESTING OF DIAPHRAGMS

roller assemblies and were displaced, in-plane, at the two ends by servocontrolled hydraulic actuators. As shown in Figure 9, lead weights, separately supported on low friction roller assemblies, were attached to each of the 18.3 m (60 ft) sides of the diaphragm. The lead weights were intended to simulate the inertial effect of the masonry side walls during the dynamic testing. In the dynamic tests, the diaphragm with the attached weights was shaken in its plane by the hydraulic actuators which were programmed to produce the displacement time history of the selected, scaled earthquake ground motions given in Table 2. For the quasi-static tests, removable reaction pillars were moved into place at the diaphragm's third points and the diaphragm was cyclically displaced in its plane by the hydraulic actuators. The testing sequence of most of the diaphragms involved the intermingling of the quasi-static and dynamic tests in which the quasi-static, cyclic amplitudes of displacement and dynamic motions were sequentially increased to progressively produce more severe levels of relative deformation within the diaphragm. In order to reduce the time required to change between the quasi-static and dynamic testing, the lead weights were left in place for both types of tests. However, due to the very low testing speeds used for the quasi-static tests, the lead weights did not induce any appreciable inertial loads in the diaphragms. The diaphragm specimens were instrumented with load cells, accelerometers, and displacement sensors; and the data from each instrument was recorded on magnetic tape in digital form. Additional data was recorded in the form of still photographs, motion pictures, and observer notes or test logs. The test data have been integrated in "Methodology of Mitigation of Seismic Hazard in Existing Unreinforced Masonry Buildings" (4).

TABLE 2. EARTHQUAKE GROUND MOTION FOR THE DIAPHRAGM DYNAMIC TESTS

Earthquake Ground Motion No.	Geographical Region	Effective Peak Acceleration, g	Earthquake Record	Scale Factor
1	New England, Carolina, New Madrid-St. Louis	0.1	Taft, 1954, N21E	1.6
2	New Madrid-St. Louis	0.1	Hollywood Storage P.E. Lot, 1971, N90E	0.5
3	Puget Sound, New Madrid-Memphis	0.2	Olympia, 1949, S04E	1.1
4	Wasatch	0.2	Castaic, 1971, N69W	1.0
5	California Pacific Coast	0.4	Castaic, 1971, N69W	1.8
6	California Pacific Coast	0.4	El Centro, 1940, S00E	1.25

CONCLUSIONS

It is clear from the dynamic analysis and test results that the dynamic response of diaphragms is dominated by their nonlinear, hysteretic characteristics for EPAs greater than 0.1 g, and elastic analyses are not valid for these earthquake intensity levels. For the most part, the diaphragm specimens were relatively undamaged for all levels of earthquake ground motion tested, and when damage occurred, the diaphragms were still serviceable and the damage was repairable. The built-up roofing adds stiffness as long as it remains attached and detachment occurred at EPAs of approximately 0.2 g. In addition, the tests showed that the proposed analytical model is a good representation of the quasi-static

test results. The study indicates that stiffening the ends of the diaphragms through retrofitting results in higher amplification of seismic response, and that current design anchorage forces of diaphragm to walls should be increased to account for roof diaphragm amplification of seismic forces.

6. ACKNOWLEDGEMENT

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