IV–18. Response of a Single-Story Brick Masonry House to Simulated Earthquakes

P. Gülkan
Associate Professor of Civil Engineering, Middle East Technical University, Ankara, Turkey; formerly Visiting Associate Research Engineer, Earthquake Engineering Research Center, University of California, Berkeley.

R.L. Mayes
Assistant Research Engineer, Earthquake Engineering Research Center, University of California, Berkeley, and Principal, Computech, Berkeley.

R.W. Clough
Professor of Civil Engineering and Assistant Director, Earthquake Engineering Research Center, University of California, Berkeley.

ABSTRACT

An overview of an experimental investigation on the reinforcement requirements for single-story masonry dwellings subjected to seismic loads is presented. The objective was to determine the appropriateness of U.S. Department of Housing and Urban Development (HUD) criteria regarding reinforcement requirements for adequate resistance of typical masonry housing construction for the type of seismic activity expected in Seismic Zone 2 areas of the 1973 Uniform Building Code. The response of a brick masonry house subjected to simulated earthquakes on the Berkeley shaking table is described. Behavior of partially reinforced and unreinforced wall panels before and after substantial crack occurrence is illustrated. Conclusions and tentative recommendations based on the entire program are also presented.

On présente ici l'exposé d'un programme expérimental sur les exigences de renforçage pour des maisons a un étage, construites de maçonnerie, qui sont soumises à chargement séismique. L'objet était l'évaluation des critères proposés par le Département de Logement et Dévelopement Urbain aux États-Unis (HUD) concernant les exigences de renforçage appartient à la construction en maçonnerie, qui sont nécessaire dans les régions designés comme Zone Séismique 2 du Uniform Building Code (Code de Bâtiment) 1973.

La résponse d'une maison en briques soumises aux séismes simulés sur la table vibrante de Berkeley est présenté. Le comportement des panneaux (sans et avec armature partielle) avant et après la formation des fissures est illustré. Enfin, les avis et conclusions tentatives obtenu du programme sont présentés.

Ein Überlick von einer angewandten Untersuchung über die Verstaerkungsordnug für einfachiges Zielgestein-Mauerwohnhaus; unterliegend zur Seismischenlast demnach überreicht. Das Ziel dafür war die Bestimmung der Verwendbarkeit des Kriteriums der U.S. Abteilung der Unterbringung und der Stadtentwicklung (HUD); hinsichtlich der Verstaerkungsordnung erforderlich und angemessen. Der Widerstand von Mustermauer Obdachkonstruktion für den Typ von seismischer Taetigkeit welche angehöriger Weise erwartet wurde, in seismischen Zonengebiet 2, für gleichmaessiges Gebaeudegesetz im Jahre 1973 anerkannt. Die Erwiderung von einem Ziegelsteinmauerhaus welches zu einem nachgemachten Erdbeben Unterworfen ist, war mit Hilfe der Schwingungstafel in Berkeley erklaert. Das Vorgehen mit teilweise verstaerkt und unverstaerktes Mauerfach; zuvor und nach starkem Rissvorfall ist illustriert. Die Zusammenfassung und vorübergehende Vorsclaege, die sich auf den ganzen Programme basieren, sind auch vorgestellt.

In queste trattate e pressentata un'espesiziene di una ricerca sperimentale sulle esigenze di rinferze per le case di muratura a sole piane, settemesse cariche sismiche. Le scepe e determinare la convenienza delle norme propeste dal Dipartimento di Alloggio e lo Svluppe Urbanistice degli Stati Uniti (HUD) riguardante alle esigenze di rinferze all'edilizia di muratura, chi sono necessarie alle regioni dell'attivita sismica, anticipate alle Regioni Sismiche 2 alla Coda della Costruzione Uniforma. E presentate la rispesta di una casa cestruita con matteni, seetomessa terrameti simulati, seconde le tavele vibrante di Berkley. E spiegate il comportamento delle pannelli di muratua, con e senza rinferze, prima e dopo la formazione delle fissure. Alla fine, sone presentate le risultate e le proposte tentative, basate sul intere programma.

INTRODUCTION

The promulgation of "Local Acceptable Standard No. 2" by the HUD Phoenix, Arizona regional office that required partial reinforcement in single-story masonry houses financed under HUD's mortgage insurance programs was questioned by the local housing industry on the grounds that it was not rational, and that it would lead to increased costs. The investigation reported in part in this paper was undertaken at the request of HUD and was designed to provide information on the response of simple masonry structures subjected to simulated earthquakes and to determine how much, or if any, reinforcement is required for adequate seismic protection. The unique feature of the program was the testing of full-scale structural components of typical masonry houses on the Berkeley shaking table.

Four masonry houses were tested.[1] Three of these were made from a standard 6×4×16 in. concrete block; one was constructed with 6×4×12in. hollow clay bricks. The results from this specimen are presented in this paper.

The determination of appropriate reinforcement requirements for all future one-story houses based on the results of a limited number of simulated earthquake tests on four masonry houses of essentially the same geometry is an imposing undertaking. The role of the earthquake simulator as a tool for general structural research is similar to the role of the digital computer in that both yield information regarding the behavior of physical models which must then be judged and evaluated before being transformed into code regulations with general applicability. In addition, the number of conceivable variations and combinations of geometry, architectural and structural details, foundation conditions and roof arrangements as well as earthquakes pertinent to Zone 2 conditions did not permit a parametric study to be performed within reasonable budgetary constraints. Consequently, the research results must be carefully evaluated before final recommendations are presented. This task will be performed by an Applied Technology Council Advisory Panel and Subcontractor.

THE TEST STRUCTURE

Feasibility of scaled models of a full-size house for the test specimen was initially considered but eventually discarded because of problems associated with extrapolating the data. Consequently, masonry wall panels 8 ft-8 in. high and 16 or 14 ft long were constructed with commercially available 6 in. wide hollow clay brick units. Built upon strip footings, these wall panels were assembled to form 16×16 ft test specimens. They were connected at the top by a timber truss roof structure. To account for the reduction of mass resulting from scaling the plan dimensions, concrete slabs were bolted to the roof. The weight of the slabs yielded a ratio of total roof load to total wall peripheral length similar to a full-size house with a distributed roof load of 20 psf.

In Fig. 1 an isometric exploded view is given for the test structure. Like other specimens in the program it was designed such that simultaneous transverse and in-plane response of both reinforced and unreinforced walls could be measured. Wall A, 16 ft in length and wall A1, 14 ft in length were unreinforced. Walls B and B1 were reinforced with a #4 vertical bar at each end. Material properties were as follows: masonry strength (net area) = 7,200 psi, Type S mortar strength = 2,340 psi, grout strength = 2,540 psi, yield strength of reinforcement = 54,000 psi.

The structure was tested by subjecting it to a series of horizontal base motions with increasing intensity. The three simulated earthquakes utilized in the program were the Taft, El Centro, and Pacoima Dam accelerograms for which no time scaling was adopted. Response was measured and recorded by means of a large number of devices which are indicated in numerals in Fig. 1. The house was initially tested with the orientation of the roof trusses in the roof assembly perpendicular to the table motion; halfway through the testing sequence the roof structure was rotated 90 degrees. At the same stage, cracks which had been formed in all the panels except B1 were repaired with the use of a surface bonding gypsum based plaster.

SELECTED RESULTS

The extrapolation of laboratory results to the field requires an assessment of the effects of factors which could not be included in the program. Among these are foundation flexibility, variability in workmanship, preexisting cracks in unreinforced walls, roof displacements resulting from larger diaphragms, and the effects of internal walls, reentrant corners and built-in structural lack of symetry. Furthermore, three dimensional ground motion of actual earthquakes could not be reproduced in the laboratory. Notwithstanding the omission of these variables in the laboratory tests, there are consistent common features of the measured response which permit generalizations,[2] and these are listed below.

Effect of Roof Truss Orientation

The orientation of the roof trusses produced significantly different response in the in-plane walls for two reasons. First, the stiffness of the roof structure was much greater when the trusses were parallel to the table motion than when they were oriented transversely. Consequently, in the former orientation roof inertia force could be transferred to the resisting walls with little reduction due to energy dissipation in the truss support assembly. Secondly, with the parallel truss arrangement in-plane walls were non-bearing and the restraint provided by the vertical load against overturning was absent. Greater damage to the walls was consistently associated with this unfavorable roof structure orientation.[2]

Pre- and Post-Cracking Behavior

In general, cracking in the partially reinforced panels was minor and had relatively insignificant effect on overall response. Even with the nominal amount of vertical reinforcement, walls B and B1 exhibited desirable behavior at all stages of testing, i.e., displacement amplitudes were such that overall instability was not imminent, and cracking was always controlled. Although it is difficult to pinpoint a precise base motion intensity at which unrein-

forced panels would be expected to develop significant cracking, the difference in response before and after its occurrence was striking. The effect is illustrated in Figs. 2 and 3 for two non-consecutive base motions in the transverse roof orientation. The simulated earthquake in Fig. 2(a) was El Centro and had a peak acceleration of 0.453 g. (It is referenced as E-0.453.) Figure 3(a) depicts the Pacoima signal for which the base motion reached a peak acceleration of 0.480 g (P-0.480). Various displacement measurements are displayed in the same order in both figures; the scales are sometimes different for clearer presentation. For the experiment shown in Fig. 2, there were no structural cracks in walls A and A1. In-plane displacements for walls A and B were measured at the top (Fig. 1); transverse displacements for walls A1 and B1 are described along a vertical axis down the middle and a horizontal axis at the top. The maximum displacement of wall A was recorded as 0.032 in.; for wall B it was 0.193 in. Coincident with the arrival of the peak base acceleration pulse wall A developed a diagonal crack in the wider pier adjacent to the window during Test P-0.480 shown in Fig. 3. The peak displacements for walls A and B were recorded as 0.357 in. and 0.477 in., respectively. A crack at midheight in wall A1 caused a maximum displacement of more than 3 in. as shown in Fig. 3(c). The reduction in overall rigidity is reflected also in the reduction of the natural frequency. Comparison of similar quantities in Figs. 2 and 3 reveals useful information on the visual effects of cracking.

Torsional Response

Even without significant cracking in wall A, a coupled translational and torsional response was obtained. In Fig. 4 the deflected shapes of panels A1 and B1 are shown at selected times during the experiments shown in Figs. 2 and 3. It is seen that during E-0.453 the edges of walls A1 and B1 adjacent to wall B were displaced more than the edges near wall A; a slight bulging at the middle of the transverse panels is also evident. After cracking in wall A1 during Test P-0.480 the overall deformation was augmented by the effects of local cracking.

Partial Reinforcement

A consistent trend in the effect provided by partial reinforcement in walls B and B1 is displayed in frames (b), (e), and (f) of Figs. 2 and 3 as well as Fig. 4. Reinforcement provided strength and reduced the displacement amplitudes of wall B1 compared with wall A1.

STRENGTH OF MASONRY WALLS

The large number of tests illustrated the effectiveness of partial reinforcement in preventing damage for all types of base motions included in the program. Unreinforced walls resisted motions with peak accelerations less than 0.3 g with no significant damage. Wall A1 cracked first during Test P-0.386, and wall A was damaged during P-0.480. Interpretation of the test data to determine when partial reinforcement is required depends on determining the strength of walls together with the effects of variables omitted from the program. This information can then be used with an adequate factor of safety for various seismic zones.

For transversely loaded masonry walls the moment vs axial load interaction curve for both reinforced and unreinforced sections can be easily established. In Fig. 5, the capacity of a 1 ft wide section for walls A1 and B1 is shown. Two curves are drawn for A1: with s denoting the ratio of tensile to compressive strength, s = 0.04 corresponds to the measured tensile strength of a masonry subassemblage, and s = 0 denotes the case when no tensile strength is admitted for the wall material. Because of the deliberate separation between transverse and in-plane walls, approximate transverse moments on walls A1 and B1 could be extracted from recorded accelerations on these walls. The circles in Fig. 5 denote maximum moments on wall A1 during the tests illustrated in Figs. 2 and 3; maximum moments on wall B1 were similar. Transverse moment resistance of unreinfored walls with negligible axial load is closely related to the bond strength between the mortar and the masonry units. If this is reduced to zero due to cracking, transverse walls will respond with large out-of-plane displacements as Figs. 3 and 4 indicate. Despite displacements in excess of 3 in., wall A1 did not collapse and this was attributed to secondary effects such as arching and the restraint provided by the roof structure.

Determination of the in-plane strength of masonry walls is more difficult. Failure modes of reinforced walls are classified as shear or flexural, and the transition from the former to the latter for appropriately reinforced walls is characterized with increasingly ductile behavior.[3] For Test P-0.480 derivation of the shear stress in the main pier from acceleration readings at the diaphragm level yields a nominal shear stress estimate of 30 psi. The distribution of cracks in the panels after this test is shown in Fig. 6. A more detailed analysis indicated that the damage to wall A could be explained in terms of uplifting and overturning rather than excess shear stresses.[1]

SUMMARY

1. A general statement regarding the intensity of base motion at which an unreinforced wall is likely to be damaged can not be made because of this depends on a large number of factors. However, for the specific brick test structure the unreinforced transverse wall was significantly cracked during Test P-0.386, and the in-plane wall cracked during Test P-0.480. The cracks in both walls occurred when the roof trusses were oriented transversely to the table motion.
2. The transverse strength of unreinforced walls is closely related to the bond strength between the masonry unit and the mortar. If bond strength is lost due to the occurrence of prior cracking, increasingly large out-of-plane displacements will occur when the walls are subjected to seismic forces. Although such large displacements were recorded for wall A1 after it was cracked, it did not collapse and this was attributed to secondary effects such as arching and roof structure restraint.
3. The in-plane resistance of wall A was governed by its resistance against overturning. The nominal shear

stress at significant cracking was about 30 psi; the development of the type of cracking illustrated in Fig. 6 is associated with tensile stresses arising from overturning.

4. Both the transverse and the in-plane strength of masonry walls reinforced with a #4 vertical bar grouted at each end was sufficient to resist the imposed loads of all the tests without significant cracking.
5. The necessity for requiring some reinforcement in single-story masonry houses in Seismic Zone 2 is dependent on the factor of safety appropriate for this type of construction. Based on force levels, a factor of safety less than about 1.3 should not require any reinforcement whereas a factor of safety greater than 4 will require at least partial reinforcement. An intermediate value will necessitate a much more detailed study of the test results which should include an assessment of the effects of parameters that were not included in the program.

ACKNOWLEDGMENTS

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3. Mayes, R.L., and R.W. Clough, "State-of-the-Art in Seismic Shear Strength of Masonry—An Evaluation and Review," Earthquake Engineering Research Center Report No. 75-21, University of California, Berkeley, 1975.

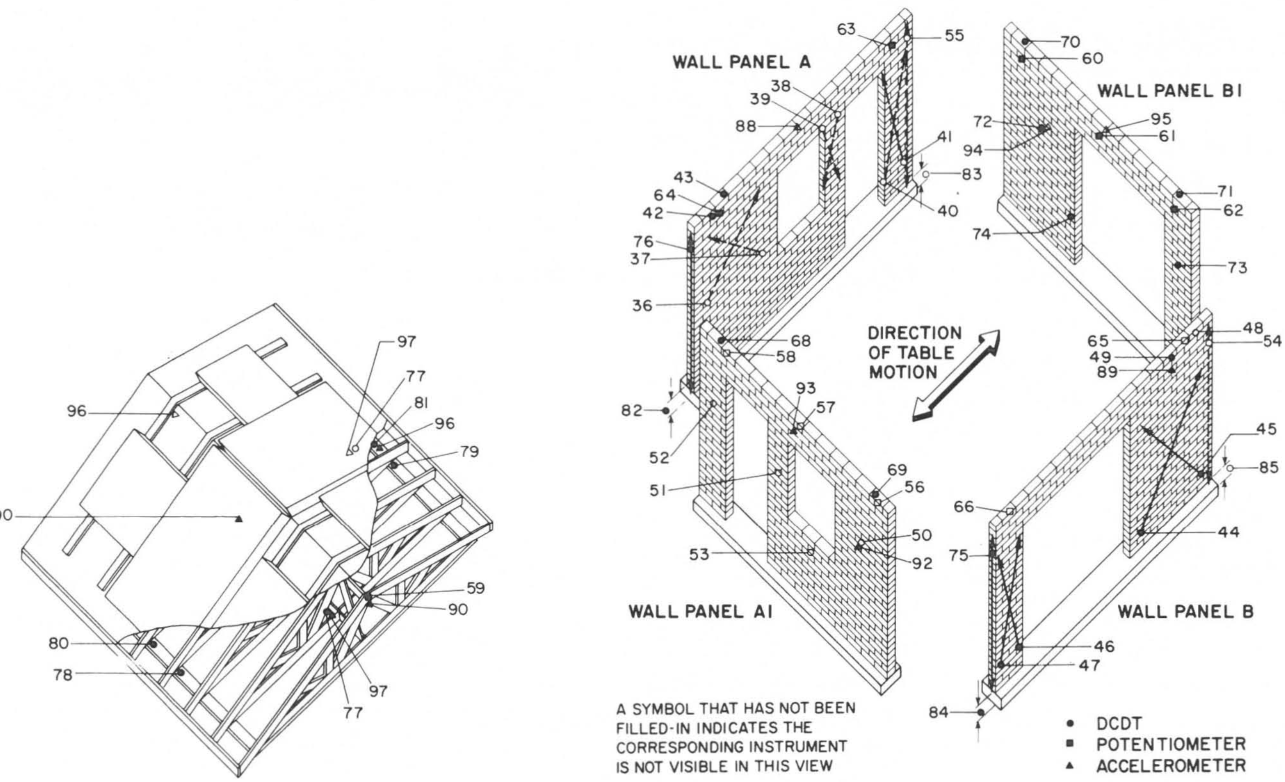


Figure 1. The Test Structure

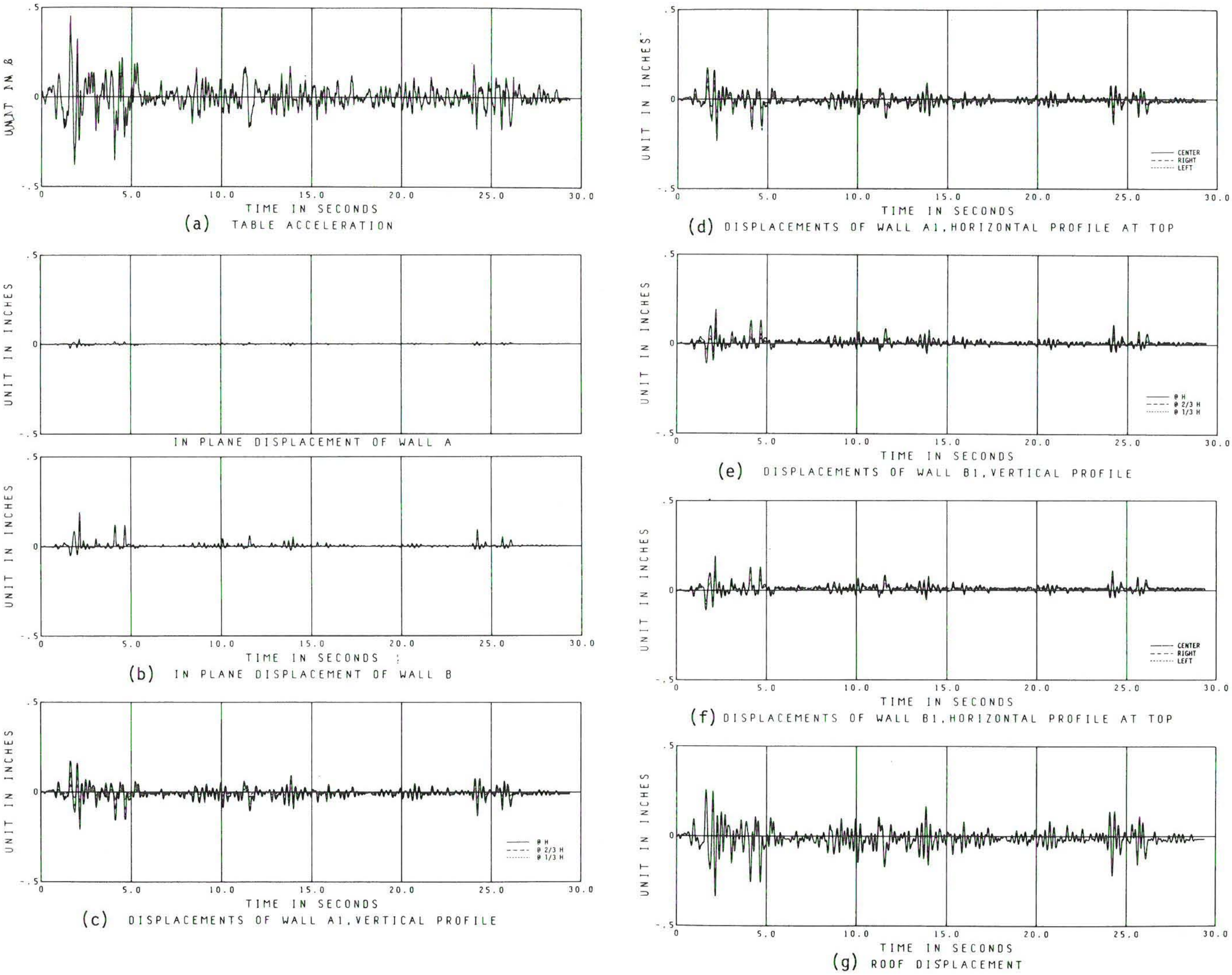


Figure 2. Response Measured During Test E-0.453

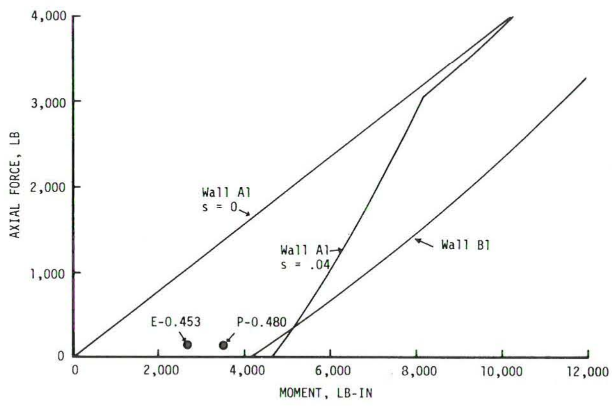


Figure 5. Strength of Transverse Walls

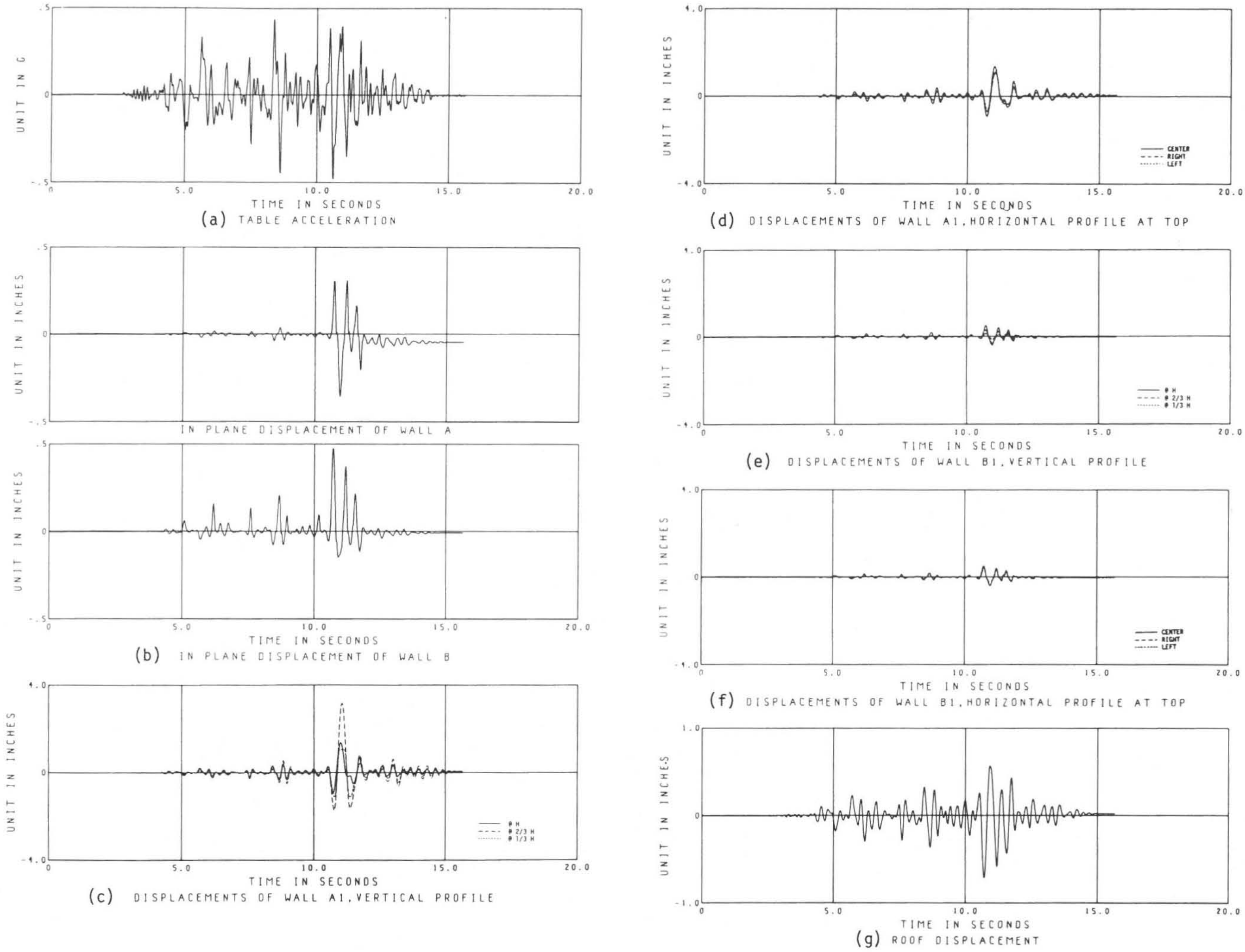


Figure 3. Response Measured During Test P-0.480

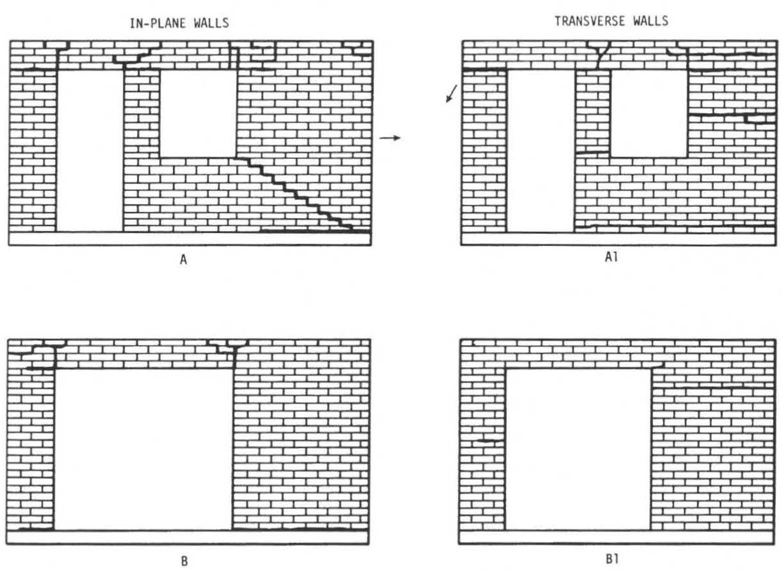
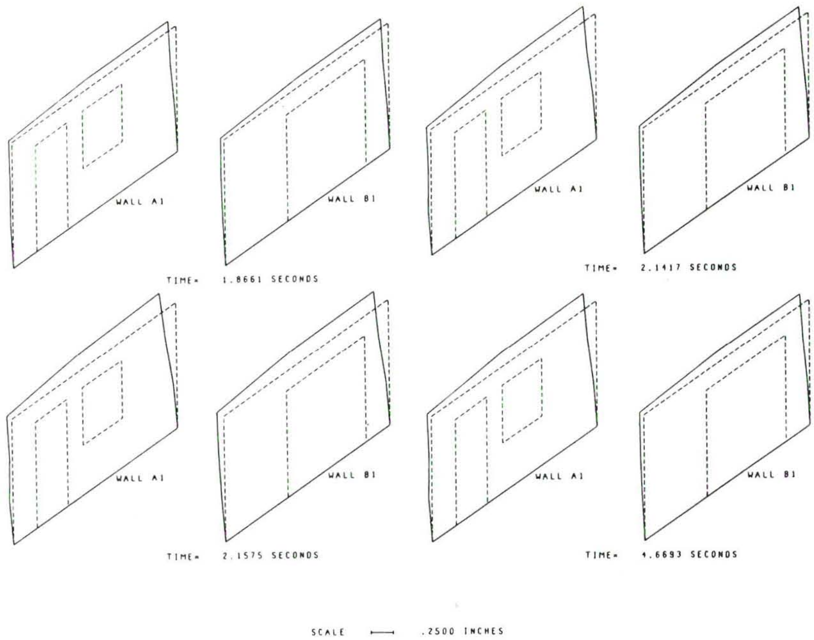
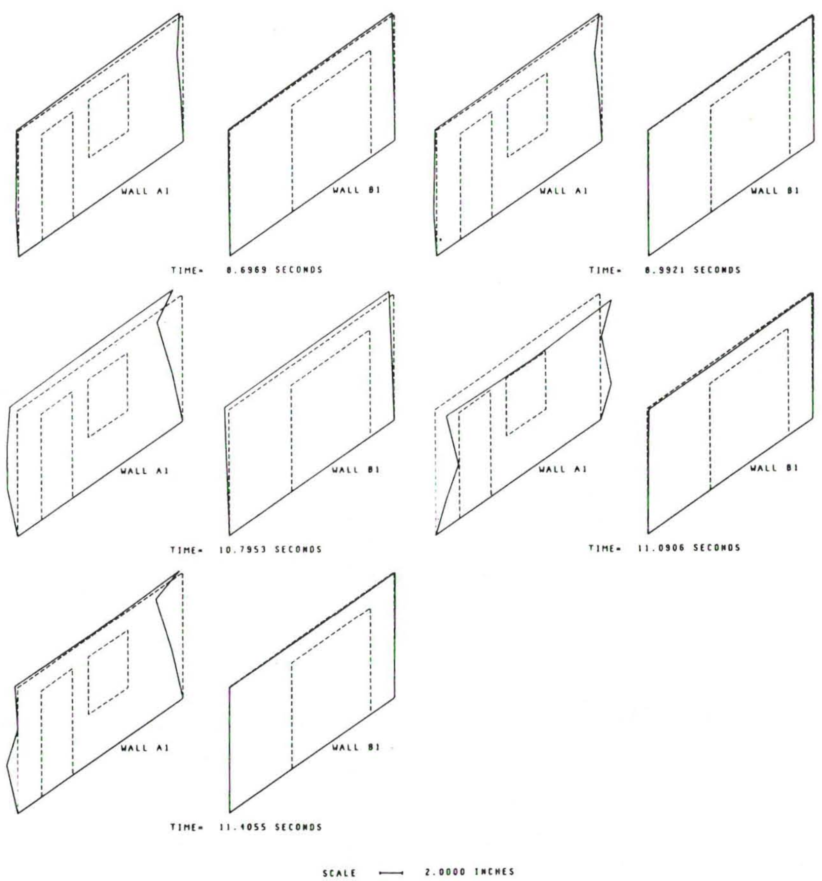


Figure 6. Distribution of Cracks After Test P-0.480



Test E-0.453



Test P-0.480

Figure 4. Deflected Shapes of Transverse Walls