AN INVESTIGATION OF THE SEISMIC BEHAVIOR AND
REINFORCEMENT REQUIREMENTS FOR SINGLE-STORY MASONRY HOUSES

by

R. W. Clough[I], P. Gülkan[II], and R. L. Mayes[III]

ABSTRACT

An investigation undertaken to determine the reinforcement requirements for single-story masonry houses in Uniform Building Code Seismic Zone 2 areas of the U. S. is described. The investigation consisted of testing four masonry house models measuring 16 ft. (4.90 m) square in plan dimensions on a two-component shaking table capable of horizontal and vertical motions. The dynamic response of each house was measured and careful observations enabled tentative recommendations to be made for two subregions defined within the current Zone 2 on the basis of effective peak ground accelerations. The tentative recommendations are that no reinforcement is necessary for single-story residences made from brick or concrete block in those subareas of Zone 2 where the effective peak ground acceleration is less than 0.1 g. Partial reinforcement is recommended for the remainder of Zone 2. Final recommendations will await the results of an additional test in which the walls of the house will be subjected to combined in-plane and out-of-plane loads.

BACKGROUND

Seismic design requirements specified by the U.S. Department of Housing and Urban Development (HUD) are referenced to "seismic risk zones" defined by the Uniform Building Code (UBC). Changes in the UBC maps were incorporated into HUD requirements and this resulted in the requirement for partial reinforcement for masonry houses in newly specified Zone 2 areas. These requirements were considered overly conservative by the construction industry in Phoenix, Arizona, one of the affected locations, and it was decided to study the question experimentally by subjecting assembled components of masonry houses to simulated earthquakes on the EERC shaking table. The primary objective was to determine the maximum earthquake intensity that could be resisted satisfactorily by an unreinforced house, and to evaluate the additional resistance that would be provided in the structure by partial reinforcing. Details of four of the test houses are given in [1] and [2]. The major conclusions and recommendations will be presented in a report after a fifth test is performed in which the walls will be subjected to both in-plane and out-of-plane forces.

STRUCTURES TESTED

The unique feature of the study was the testing of full scale components of typical masonry houses subjected to motions recorded in actual earthquakes. Masonry walls 8 ft - 8 in. (2.64 m) in height and up to 16 ft (4.90 m) long were constructed with commercially available 6 in. wide concrete block or clay

[I] Professor of Civil Engineering and Assistant Director, Earthquake Engineering Research Center, University of California, Berkeley, U.S.A.

[II] Associate Professor of Civil Engineering, Middle East Technical University, Ankara Turkey.

[III] Asst. Research Engineer, Earthquake Engineering Research Center, University of

brick units. The walls were assembled to form 16 ft. (4.90 m) square test "houses" built on strip footings. The individual wall units were connected at the top by a timber roof structure of standard construction. Concrete slabs were bolted to the roof structure to compensate for the reduction of mass resulting from scaling the plan dimensions. The weight of the slabs were chosen so that the ratio of total roof load to total wall peripheral length was similar to that of a 40 x 50 ft. (12 x 15 m) house with a specified roof load of 20 psf (1 kN/m^2).

Fig. 1 shows a three-dimensional view of a typical test house, one of four specimens tested. All models were designed so that transverse and in-plane response of both unreinforced and partially reinforced panels could be observed in a single test. All partial reinforcement consisted of vertical bars. In the last house a series of tests were conducted when all four wall panels were initially unreinforced; during the subsequent phase, all walls were partially reinforced with two No. 3 (10 mm) bars.

The test structures were generally subjected to a series of base motions with progressively increasing intensity. Some tests performed on Houses 3 and 4 included both horizontal and vertical components of motion. Three earthquake motions were used derived from the 1940 El Centro, 1952 Taft and 1971 Pacoima Dam accelerograms. Roof truss orientation and local repair of cracked walls were also included as test parameters.

OBSERVATIONS ON THE RESPONSE BEHAVIOR

The specimens used in this study were typical of "box" structures which derive their lateral force resistance from "membrane" action of the walls. The major part of the lateral force developed in these tests resulted from the concrete blocks bolted to the roof. Resistance to this force was provided by a mechanism dependent on the relative in-plane shear rigidity of the roof and wall components; the out-of-plane rigidity of the wall panels and the flexural stiffness of their connections to the roof were of negligible value in resisting the roof loads. The roof structure simply provided the top support for out-of-plane forces.

From this description it is clear that the out-of-plane walls of a masonry house must have sufficient flexural strength to resist their own inertia forces when acting as vertical beams, while the in-plane walls must have the capacity to resist the inertia forces of the entire roof system plus the top half of the walls.

In general, the observed behavior was consistent with this description of box structures subjected to lateral forces. During the tests, roof displacement amplitudes were directly related to the behavior of the in-plane walls (designated as A and B in Fig. 1). Differential displacements of the two in-plane walls were accomodated by "racking" distortions of the roof; relatively little in-plane distortion was observed in the out-of-plane walls, so it may be concluded that the roof structure did not rotate as a rigid unit. This is consistent with the usual design assumption that plywood diaphragms are much more flexible in shear distortion than are masonry walls.

A significant observation made from these experiments was that typical single-story masonry houses are so rigid that they do not develop complicated response mechanisms during an earthquake. Motions of the test structures followed the shaking table motions very closely, with distortions generally proportional to, and in phase with, the base accelerations. The peak input acceleration may therefore be cited as the dominant quantity controlling response. The most significant features of the observed response of the test structures taken as a whole may be summarized as follows:

For Unreinforced Wall Units:

(1) No cracking was observed in any major unreinforced wall unit for tests with peak accelerations less than 0.2 g. The lowest intensity shaking that caused cracking of a non-bearing in-plane wall occurred during tests with peak accelerations of 0.21 g; the minimum intensity to cause cracking of an out-of-plane wall was 0.25 g.

(2) Unreinforced out-of-plane walls continued to perform satisfactorily after cracking during several tests of increased intensity, but the displacements of these walls generally became excessive during tests with accelerations greater than 0.4 g. These large displacements involvedhinging at the horizontal crack line and exhibited potential instability.

(3) Cracking of unreinforced in-plane walls was of two-types: horizontal cracks in panels without openings, and a diagonal crack extending downward from the window corner in the wall units with window penetrations. Permanent displacements generally were not associated with the horizontal cracks; however, the diagonal cracks led to permanent displacements which became unacceptably large with further testing.

For Partially Reinforced Wall Units:

(1) Nearly all partially reinforced wall units performed satisfactorily in all tests. None of the partially reinforced out-of-plane components developed any significant cracks during any test, including several with peak accelerations in excess of 0.5 g.

(2) Partially reinforced in-plane walls also performed satisfactorily although some cracked when peak accelerations exceeded 0.3 g. Cracking in the pier units without window openings was associated with rigid body rocking, and included a horizontal crack due to uplift near the base of the wall. Residual cracks were easily repairable.

(3) The only partially reinforced wall which exhibited unsatisfactory behavior was the window wall of House 4 (unit A in Fig. 1). A typical diagonal crack extending from the window corner to the "toe" of the wall developed during the first phase of testing when this house was unreinforced. After the addition of two undowelled bars, the wall resisted a 0.32 g test without additional cracking. However, in subsequent tests with peak accelerations in the range of 0.47 to 0.68 g further cracking did develop as a result of uplift at the undowelled corner.

Extrapolation to Prototype Conditions

This general description of the observed behavior provides the basis for the tentative recommendations presented below concerning seismic design criteria for single-story masonry houses. However, before these observations may be applied, it is necessary to estimate the extent to which they represent the performance of real houses subjected to real earthquakes. Comparisons of shaking table test conditions with those existing in a prototype response to earthquakes were considered with regard to: (1) seismic input, (2) roof load, (3) foundation flexibility, (4) geometric effects, (5) roof diaphragm flexibility, (6) pre-existing state of stress in walls, (7) torsional response mechanisms, and (8) progressive damage. After evaluating each of these factors in detail, it was concluded that the behavior observed in the shaking table tests was quite similar to the performance expected of a real house subjected to a real earthquake with a similar peak acceleration. The only significant shortcoming of the shaking table tests was that only a single horizontal component of earthquake motion was applied, so that walls were subjected to either in-plane or out-of-plane forces. It is believed that the out-of-plane response of unconfined walls might have an unfavorable influence in their resistance to a simultaneous in-plane excitation, and it was decided that this negative effect should be investigated in an additional test before final recommendations are presented.

TENTATIVE DESIGN RECOMMENDATIONS

Seismic Input for Zone 2

From the earliest stages of this investigation, one of the most critical questions related to the intensity of shaking table accelerations that should be used to represent the maximum earthquake motions expected in UBC Zone 2. This correlation of shaking table motions to field excitation is required to relate the damage observed in the test structures to the expected behavior of real houses in Zone 2.

The best current estimate of expected earthquake intensity for the U.S. was developed by the Applied Technology Council (ATC) in preparing proposed seismic design regulations for buildings [3]. Figure 2 shows the ATC map of effective peak acceleration (EPA) contours superimposed on the 1976 UBC Seismic Zoning Map. The EPA contours are intended to represent effective ground motions with a 10 percent probability of being exceeded during a 50 year period. The EPA of a given ground motion is defined in terms the response spectrum of the motion evaluated for 5 percent of damping by drawing a line of constant spectral acceleration approximating the peaks and valleys of the spectrum in the period range of 0.1 to 0.5 seconds. The EPA is given by this spectral acceleration divided by 2.5, where the divisor is typical of the amplification for Western U.S. earthquakes. The concept of EPA was introduced in [3] to avoid overemphasizing the peak ground acceleration, the value of which often does not relate well with the damage induced by a given motion.

It will be noted in Fig. 2 that Zone 2 includes a wide range of EPA values from 0.05 to 0.2 g. It is not reasonable to impose design requirements suitable for the maximum EPA value of 0.2 g for all of Zone 2, and accordingly, two subzones were defined within it. Zone 2A is the part of Zone 2 indicated by the ATC map to have an EPA of less than 0.1 g while Zone 2B is the areas with EPA values of 0.1 to 0.2 g.

EPA values of the shaking table motions were determined by applying the above definition to the shaking table response spectrum. Because the tests were conducted with widely varying intensities, the table motions were all normalized to 1 g before the response spectra were constructed. The resulting combined average EPA value was 0.82 g for the three types of base motions used in the experiments. This means that a table motion having a peak acceleration of 1 g is assumed to have an EPA of 0.82 g, or conversely, the maximum EPA of 0.2 g indicated by the ATC map for Zone 2 is represented by a peak shaking table acceleration of 0.24 g.

Test Structure Amplification

Although masonry houses are relatively rigid, they do exhibit some vibratory amplification so that peak accelerations recorded on the structure are greater than the peak input acceleration. This amplification effect is represented in the definition of the EPA by the 2.5 divisor; that is, ATC has tacitly assumed an amplification factor of 2.5 to be appropriate for typical building structures.

Experimental data obtained during the course of this study demonstrated that the amplification varied considerably, from point to point on the test structures, and with differing test conditions. Amplification factors are important in the design of structures to resist earthquakes because the seismic load induced in any part of a structure is given by the product of the mass of that part multiplied by its local acceleration. In a single-story masonry house the principal seismic force results from the mass of the roof structure. Hence, the seismic load to which a house is subjected is given by the roof acceleration amplification factor multiplied by the product of the roof mass and the table acceleration. Careful review of test data indicated that an amplification factor of 2.5 was appropriate for estimating the seismic forces induced in the test structures by the given peak table acceleration.

Tentative Design Criteria

As noted earlier, the principal purpose of this investigation was to determine the amount and type of reinforcing that should be provided in single-story masonry houses constructed in Zone 2, and to recommend design provisions that will satisfy these requirements. Because two subzones having different earthquake intensities have been identified in Zone 2, it was necessary to formulate different recommendations for each subzone.

A. Criteria for Zone 2A

The maximum effective peak acceleration to be expected in this subzone is 0.1 g; this EPA is provided by shaking table tests with a peak table acceleration of 0.12 g. Concern about the performance of unreinforced walls subjected to combined in-plane and out-of-plane forces led to the recommendation for an additional test with the walls subjected to combined forces. The combined force effect was accounted for in the tentative recommendations by increasing the intensity of the single component by 30 to 50 percent. Thus, a single shaking table test with a peak acceleration of 0.16 g to 0.18 g is assumed to simulate the effects of a maximum Zone 2A earthquake on an unreinforced wall.

Review of test data [1] and [2] shows that no damage of any type occurred in any wall of any test structure during tests not exceeding this peak value of 0.18 g. Unreinforced walls which had been cracked during more severe tests performed satisfactorily in subsequent tests of 0.18 g or less. Based on this observation, the following tentative code provision is presented. "For Zone 2A, no reinforcing is required for earthquake resistance in single-story residential buildings of standard clay brick or concrete block construction provided the ratio of shear wall length to roof load is similar to that included in the tests."

B. Criteria for Zone 2B

For Zone 2B, the maximum expected EPA of 0.2 g is provided by a shaking table peak acceleration of 0.24 g. For unreinforced walls this intensity was increased by 30 to 50 percent to account for the damaging effect of the second horizontal motion component.

Review of response observations revealed that the only unreinforced wall that withstood this intensity of shaking without damage was the in-plane wall of House 2 for which the mortar strength was measured to be 4,700 psi (32.4 MPa). The unreinforced in-plane walls of all other test structures, and the unreinforced out-of-plane walls of all other test structures exhibited damage after tests with peak accelerations less than 0.36 g. Also, the performance of cracked unreinforced walls was unsatisfactory during tests with less than 0.36 g peak accelerations. Based on these observations it was concluded that partial reinforcement is necessary in the walls of masonry houses built in Zone 2B.

When walls are partially reinforced little coupling is expected between in- and out-of-plane response mechanisms. Accordingly, the intensity of the single-component shaking table motions was increased by only 20 percent to account for the orthogonal motion effect. Thus, a shaking table motion with a peak of 0.29 g was taken as the basis for judgement of adequate performance. Test data reveals that no cracking damage developed in any of the partially reinforced walls during tests with peak accelerations of 0.29 g or less. In fact, no damage to the partially reinforced out-of-plane walls occurred in any test including peak accelerations greater than 0.6 g. Also, no requirement for dowels of such walls was indicated.

On the other hand, some cracking was observed in the partially reinforced in-plane walls of all test structures. Generally, this cracking was at the base of the piers and above the ends of the door and window lintels. It was associated with rigid-body rocking of the piers, and does not represent a serious damage condition.

The final step in formulating the design recommendations for Zone 2B is to generalize the essential factors of partial reinforcement included in the test structures. These recommendations are presented in the form of minimun standards whichensure adequate resistance to out-of-plane forces. These standards also pertain to the in-plane resistance, and it is believed that adequate in-plane resistance could be achieved by prescribing such minimum standards.

The principal recommendations concerning in-plane strength are presented in the form of a design procedure which involves first estimating the lateral force that would be developed in the structure due to the maximum expected Zone 2B earthquake. The acceleration inducing this force is given by the maximum EPA of Zone 2B increased by a factor of 2.5. Thus, the acceleration acting on the roof system is 0.5 g, and each in-plane wall resists half the total load.

The seismic force developed at the roof level must be resisted by shear stresses in the in-plane walls, and for the purpose of the following recommendations it is assumed that only panels that are at least 6 ft. (1.8 m) wide and without window penetrations will provide the required resistance. Maximum shear stresses calculated for wall panels which performed satisfactorily during the tests were 34, 38, 40, and 39 psi in Houses 1 to 4, respectively. Because these did not necessarily determine the limit of good performance, it is likely that the effective strength is higher than these values so the value of 40 psi (kN/m^2) was selected as the allowed shear stress. It should be emphasized that the assumption of satisfactory performance with this magnitude of shear stress is based on the premise that the resisting panel has vertical reinforcement at each end capable of accomodating rocking rigid-body displacements. To account for the ductile response of the shear wall reinforced as recommended and for the forces resisted by the interior partitions of the house, it is recommended that the design load be 0.5 g times mass/1.5.

In conclusion, the following criteria are recommended for Zone 2B:

Single-story houses of clay brick or concrete block masonry built in Zone 2B must be partially reinforced. For the purpose of providing adequate seismic resistance, partial reinforcement must meet the following conditions:

(1) Minimum reinforcing bar size is NO. 3.

(2) Each exterior corner of the house must be reinforced by at least one doweled bar; dowels are not otherwise necessary.

(3) For out-of-plane resistance:

(a) At least one bar is required in each pier extending from floor to lintel or ceiling height.

(b) Maximum bar spacing is 8 ft. (2.5 m) except that shear panels selected for in-plane resistance up to 12 ft. (3.5 m) long need not have more than two bars.

(4) For in-plane resistance:

(a) The in-plane resistance is provided by shear panels which are defined as a wall or a portion of a wall extending from floor to lintel or ceiling height, at least 6 ft. (1.8 m) wide and without penetrations.

(b) A vertical bar is required at each edge of a shear panel.

(c) The total length of shear panels oriented along each axis must be sufficient to resist a horizontal force equal to half the weight of the roof system divided by 1.5 with the net shear stress not to exceed 40 psi (21 kN/m^2).

ACKNOWLEDGEMENTS

The investigation described in this paper has been funded by the U.S. Department of Housing and Urban Development. Planning of the research was done with the cooperation of an Applied Technology Council Advisory Panel.

REFERENCES

[1] Gulkan, P., Mayes, R. L. and Clough, R. W., "Shaking Table Study of Single-Story Masonry Houses - Volume 1, Test Structures 1 and 2, Earthquake Engineering Research Center Report No. UCB/EERC-79/23, August 1979.

[2] Gulkan, P., Mayes, R. L. and Clough, R. W., "Shaking Table Study of Single-Story Masonry Houses - Volume 2, Test Structure 3 and 4," Earthquake Engineering Research Center Report No. UCB/EERC-79/24, August 1979.

[3] "Tentative Provisions for the Development of Seismic Regulations for Buildings," Applied Technology Council Publication ATC3-06, (NSF Publication 78-8, NBS Special Publication 510), U. S. Government Printing Office, June 1978.

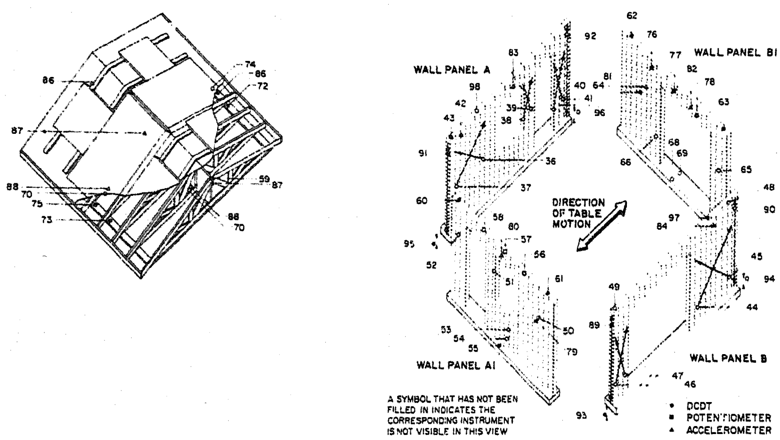

FIG. 1 TYPICAL TEST STRUCTURE

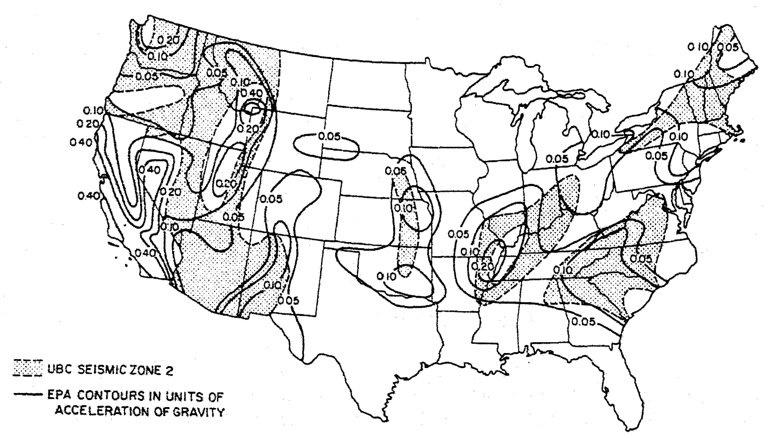

FIG. 2 ATC EFFECTIVE PEAK ACCELERATIONS FOR THE UNITED STATES