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SEISMIC EVALUATION OF UNREINFORCED MASONRY BUILDINGS

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ABSTRACT

The paper provides an overview of analysis methods that are presently used, or are being proposed, for seismic evaluation of unreinforced masonry buildings. Comments regarding the accuracy and usefulness of various computational methods are based on measured response of reduced-scale test structures subjected to simulated earthquake motions on a shaking table. Various analytical methods are discussed in terms of their applicability for modeling relevant traits of dynamic response of systems with perforated shear walls and flexible diaphragms. Information given in the paper is also intended to support numerical modeling tasks done to assess urban loss in the Memphis area as a result of damage to masonry buildings.

KEYWORDS

building systems, clay-unit masonry, dynamic response, flexible diaphragms, loss assessment, Memphis, rocking, seismic evaluation, structural analysis, urban, unreinforced, walls

INTRODUCTION

Earthquakes always excite buildings dynamically, and much of the time in a manner in which forces are not linearly related to displacements. Yet, analysis methods used to represent seismic response are principally based on simple static and linear behavior. Whereas these approximate procedures are deemed appropriate for design of new, ductile frame and wall systems with rigid diaphragms, they may not necessarily be applicable for evaluation of existing, unreinforced masonry buildings with flexible diaphragms. The purpose of this paper is to provide insight into whether these approximate analysis procedures can be used to estimate true nonlinear dynamic response of unreinforced masonry buildings. Results from various computational models are correlated with measured response of two, reduced-scale test structures constructed of unreinforced clay-unit masonry, and subjected to an array of simulated earthquake motions.

Complete descriptions are not given of the experiments, the computational methods, or the correlations between computed and measured data. However, conclusions as stated in the paper are relevant to the development of improved evaluation and loss assessment procedures, and should prompt an interested reader into seeking the more lengthy reference material.

One direct application of the research is to improve the accuracy of computational techniques used to generate fragility curves for unreinforced masonry buildings in the Memphis region, as part of an integrated program of research at the U.S. National Center for Earthquake Engineering Research. The NCEER *Loss Assessment of Memphis Buildings* (LAMB) Project is a coordinated research program that combines talents from structural engineering, seismology, risk/reliability and socioeconomic researchers. The effort provides a demonstration of how these various disciplines can be integrated to estimate economic losses for a scenario earthquake in the Memphis area.

DESCRIPTION OF SHAKING TABLE EXPERIMENTS

Two reduced-scale, unreinforced masonry test buildings were subjected to an array of simulated earthquake motions on a shaking table. Each two-story test structure was three-eighths scale and constructed of clay masonry units and Type O mortar placed in a two-wythe, running bond pattern (wall thickness equal to 94m). For the first test structure, S1, perforations in each of the two parallel shear walls (Fig. 1) were chosen so that lateral stiffness and strengths of the two wall elements were similar. For the second test structure, S2, the size and placement of perforations (Fig. 2) were varied to result in dissimilar stiffnesses and strengths for the two parallel shear walls. This was done to examine the load sharing and possible torsional effects, if any, between the two walls. Pier dimensions and aspect ratios are given in Table 1.

Table 1. Dimensions (mm) and aspect ratios of piers

Structure	Wall	Exterior Piers			Interior Piers		
		h	L	h/L	h	L	h/L
S1	door	812	440	1.85	812	686	1.18
S1	window	456	240	1.90	456	340	1.34
S2	door	812	240	3.38	812	340	2.39
S2	window	456	440	1.04	456	686	0.66

Model units were cut from solid clay paver units, and had an average compressive strength of 46.4 Mpa. Model mortar was fabricated by sifting sand free of large particle sizes (larger than a #30 screen or 600μ) to be consistent with the scale factor. The nominal thickness of mortar joints was 5 mm. Average compressive strength for a population of 38 test prisms was 13.5 MPa with a c.o.v. of 0.15. Flexural tensile strength normal to the bed joints was determined from tests of simply supported masonry beams. The average of three tests was 0.28 MPa with a c.o.v. equal to 0.09. In-place shear tests were done on undamaged portions of the test walls following the earthquake simulation test runs. Shear values, adjusted for vertical stresses, averaged 2.49 MPa with a c.o.v. equal to 0.20. The in-place shear strength value exceeded by 80% of the tests (10 out of 12) was 2.06 MPa.

Shear walls were attached to each other with flexible diaphragm elements. These elements were constructed of steel beams of rectangular cross section. The diaphragm beams were sized so that they would be strong enough to support both gravity and lateral inertial forces without yielding while being sufficiently flexible so that the diaphragm lateral frequency would be approximately one third that of a system with rigid diaphragms. This was done to investigate different amplifications of base accelerations for walls and diaphragms. Transverse masonry walls were attached to the end diaphragm beams so that their deflected shape would be equal with that of the flexing diaphragms.

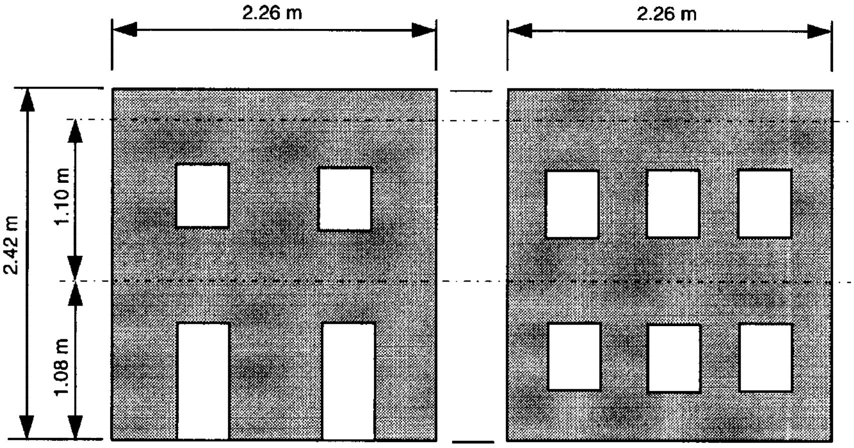
Supplemental mass was added at each of the two floor levels so that inertial forces would be sufficiently large to damage shear walls at a base acceleration within the limits of the earthquake simulator. The total weight of each test structure was 68.5 kN with 65% of the weight supported by the two diaphragms and the remaining 35% of the weight in the masonry walls. The gravity compressive stress at the base of each pier ranged from 0.23 to 0.33 Mpa.

Each test structure was subjected to scaled-versions of the motions measured during the 1985 Nahanni earthquake in the NW Canadian territories. This record was chosen because it had similar characteristics of eastern United States earthquakes such as shallow depth, intraplate center and shifted spectrum towards higher frequencies. The time scale of the recorded earthquake motion was compressed by a factor of 1.6

which was equal to the square root of the length scale factor of 2.5. Base accelerations were progressively increased from 0.1 to 1.3 times the acceleration of gravity to investigate response to an array of different seismic intensities.

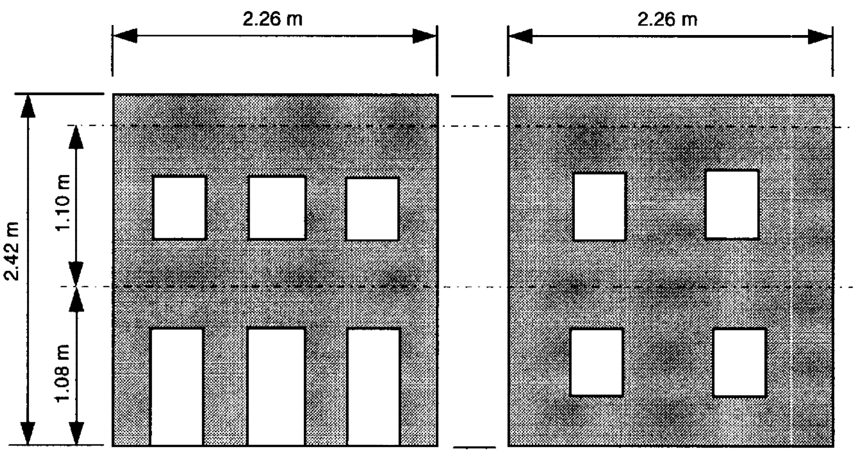
An overall summary of response for each test structure is shown in Fig. 3 where peaks in measured base shear are plotted vs. peaks in measured first-story lateral deflection for all test runs. Despite having the same sum of pier cross-sectional areas, test structure S1 was stronger than S2 because height-to-length aspect ratios were less for the more vulnerable weaker wall of the S1 pair than for the weaker wall of the S2 pair. Initial bed-joint cracking at top and bottom of first-story piers was observed at drifts of nominally 0.1%. Because rocking of base-story piers was induced subsequent to this cracking, behavior of the test structures was ductile to drifts of approximately 0.9% before the experiments were concluded due to limitations of the earthquake simulator.

Additional information on the experiments can be found in Costley and Abrams (1995a and 1995b).



a) door wall (b) window wall

Fig. 1 Elevations of test structure S1



a) door wall (b) window wall

Fig. 2 Elevations of test structure S2

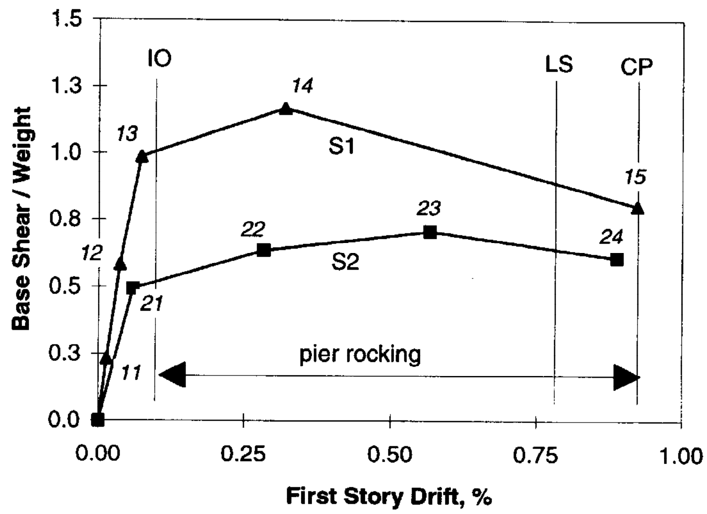


Fig. 3 Summary of measured peak response for test structures S1 and S2

ANALYTICAL MODELS

The accuracy and convenience of various computational models was investigated by modeling seismic response of the two test structures. Models ranged from simple and approximate linear static procedures to nonlinear dynamic procedures. Brief reviews of four models are given in this section emphasizing the merits and shortcomings of each method.

Linear Static Analysis

A conventional analytical method for seismic evaluation or design of building systems is the equivalent base shear method. An approximate base shear is estimated with an equation of the form,

$$V = \frac{ZIKCSW}{R} \tag{1}$$

where C is a spectral seismic coefficient equal to the spectral acceleration, S_a, W is the weight of the structure and R is a force reduction factor that varies with the ductility of the system. The terms, Z, I, K and S represent the seismic zone, importance of structure, redundancy of structural system and soil flexibility, and can all be assumed equal to 1.0 for the shaking table experiments.

The value of R can be deduced from the measured response of the shaking table test structures by taking the weight times the acceleration, S_a, from spectral response curves of measured base motions for the observed building period, and dividing by the measured base shear maxima, V_b. A summary of this operation for all test runs for each test structure is given in Table 2. A maximum inferred R value of 4.34 was found. Lower values were associated with test runs that did not reach the ultimate limit state, and could be used in performance-based evaluations for various drift levels.

Linear Dynamic Analysis

One limitation of the equivalent base shear equation (1) is that the total weight is used rather than the effective modal weight which can be determined from shape of vibration. This is an important distinction, particularly for a flexible diaphragm system where floor or roof displacements can be much more than lateral wall deflections.

Table 2 Spectral acelerations, base shear maxima and first-story drifts

Test Run	Δ/h, %	S_a, g	V_b/W	R
11	0.01	0.41	0.23	1.78
12	0.04	1.24	0.58	2.14
13	0.07	1.72	0.99	1.74
14	0.32	2.80	1.17	2.39
15	0.92	1.90	0.81	2.35
21	0.06	0.82	0.49	1.67
22	0.28	1.00	0.64	1.56
23	0.57	1.40	0.71	1.97
24	0.89	2.65	0.61	4.34

In accordance with linear structural dynamics principles, the first-mode base shear is related to the spectral acceleration as follows.

$$V_{b1} = \Gamma_1 \frac{S_{a1}}{g} \sum_{i=1}^{p} w_i \phi_{1i} \tag{2}$$

where Γ_1 is the participation factor for the first mode which is a function of the mode shape and mass distribution, w_i is the weight associated with each degree of freedom, and ϕ_{1i} is the first-mode coordinate for degree of freedom *i*. The effective modal weight is then the product of the participation factor times the summation term. Assuming the measured deflections to be primarily in a first-mode pattern provides direct information of the mode shape, and thus the effective weight. Because the deflected shape changed with intensity of base motion as wall deflections increased relative to the diaphragms with rocking, the effective modal weight varied from 82.4% to 98.7% of the total weight. Thus, the elastic base shears according to (1) should reduce slightly as would the *R* values given in Table 2. Further discussion on measured and estimated force maxima can be found in Abrams (1996).

Another aspect of linear response that can only be revealed with a dynamic analysis is the differential amplification of base accelerations for the shear walls and diaphragms. Fast Fourier transforms of wall acceleration indicated dominant frequencies at approximately 23 Hertz for the stiff masonry walls and 9 Hertz for the more flexible diaphragms. Base shear and moment forces resulting from inertial loadings were not in direct proportion with the relative wall and diaphragm masses since the diaphragms were responding to input frequencies differently than the wall mass. This is clearly seen in Fig. 4 where the measured base-moment response histories for the diaphragm inertial forces are constrasted with the moment histories for the wall inertial forces (test run 11).

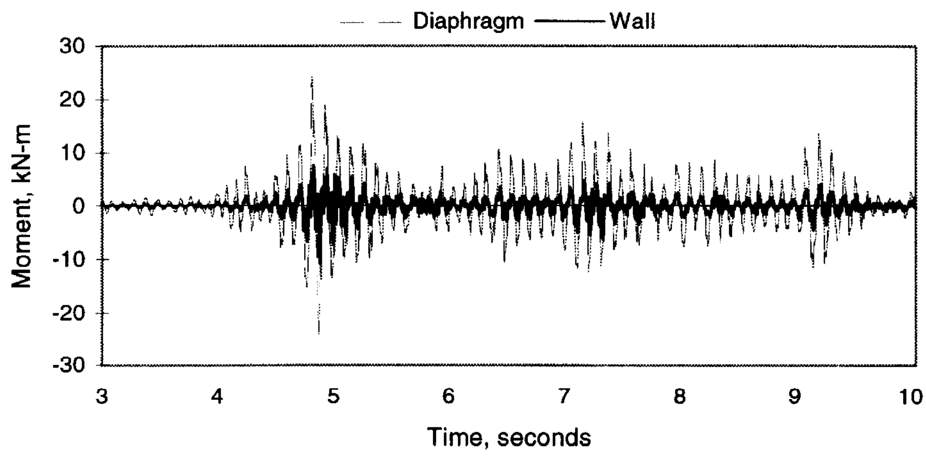


Fig. 4 Measured base moment histories for wall and diaphragm mass

Though the wall mass comprised 35% of the total mass, the higher frequency moments from wall inertia were disproportionately small relative to moments from diaphragm inertia. The lower frequency found in the wall response was obviously a result of the diaphragms driving the walls. This dynamic effect is not modeled with a static analysis.

Nonlinear Static Analysis

An analysis procedure known as the "push over" method has become popular for seismic evaluation of steel and concrete frame structures because the softening of an overall system can be modeled as various beam, column or bracing members become plastic. Nonlinear deformation demands on critical components can be identified for comparison with deformation capacities to judge whether rehabilitation of individual components should be done. This method has also been suggested for evaluation of unreinforced masonry buildings in a new set of guidelines for systems comprising all types of construction materials.

A model of the test structures was envisioned where pier and spandrel beam elements were modeled with line elements connected at zones of infinite stiffness. Pier elements were modeled with an elasto-plastic behavior whose elastic stiffness was based on uncracked sections, and strength was based on the rocking strength. Gravity forces were sustained while lateral forces, applied equally at the first and second floor diaphragms, were progressively increased. A pier was removed from the model when the applied shear force reached the pier cracking or rocking strength. A force equal to the pier rocking strength was applied to the remaining structural system. Gravity forces were still assumed to be resisted by piers after cracking or rocking.

The global force-deflection relation for the shear wall system was essentially bilinear despite the iterative approach. As the first pier tended to rock, additional shear force was attracted to adjacent piers which then rocked soon after. This was because the aspect ratios of all piers was similar for the weaker of the shear wall pair. An approximate push-over curve could have been constructed from the elastic stiffness of the wall, and the combined rocking strengths of all the piers.

The push-over curves did not agree well with measured force-deflection curves, particularly because deflections were underestimated with the overly stiff frame model that did not represent softening effects related to progressive cracking. In actuality, the perforated walls resisted in-plane shear forces as a two-dimensional continuum where principal stresses flowed between the openings at directions which varied from point to point. As gravity compressive stresses were exceeded by tensile stresses from lateral forces, local cracking developed and considerable stress redistribution occurred. These effects cannot be modeled with a conventional beam or column element. An elastic finite element model was used to simulate these effects, but was not useful for modeling post-cracked behavior. Cracked elements could not be removed because they still resisted gravity stresses. An inelastic finite element model is necessary for a push-over analysis of the shear walls.

One other shortcoming of the push-over method for modeling response of the test structures is that diaphragm inertial forces occurred non-concurrently with wall inertial forces because of their different frequency. Furthermore with pier rocking, deflected shapes did not remain invariant with all amplitudes of motion, and thus effective modal mass varied as well as the distribution of lateral forces.

The push-over method is used either with the capacity-spectrum or the coefficient method to identify the maximum target displacement a system is likely to encounter for a particular base motion. Forces in various components resulting from this global target displacement are determined, and judged to be acceptable for a given level of performance. With rocking, the test structures could displace to very large lateral displacements without attracting any additional force. Thus, reaching a specific target displacement has no relevance on the amount of force any component is likely to attract, or the amount of damage that should occur. Provided that a pier component remains stable to support gravity forces, the only damage it should incur with a rocking mechanism are bed-joint cracks that will close under vertical compressive stress following an earthquake, no matter what the amplitude of the lateral drift is.

Nonlinear Dynamic Analysis

Because of the lower frequency diaphragm effects, the floor mass and the wall mass did not produce concurrent inertial forces. A dynamic analysis was found to be necessary to represent the two distinctly different frequencies of the wall and diaphragm. When shear forces exceeded cracking or rocking strengths of piers, these components did not loose strength or collapse, but revealed a highly ductile response. A nonlinear dynamic analysis was found to be necessary to represent both the flexible diaphragm and the post-cracking effects.

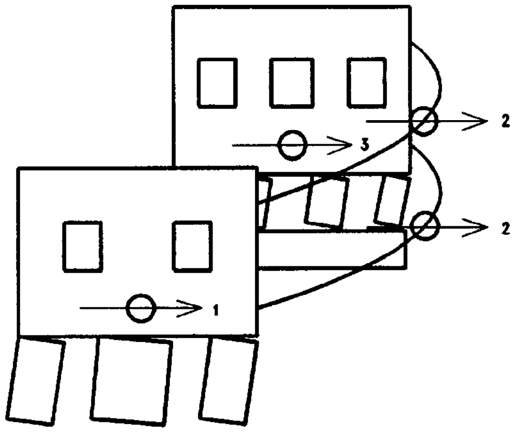


Fig. 5 Nonlinear dynamic analysis model

The simplest model that was explored to represent the measured response was the three-degee-of-freedom system shown in Fig. 5. This model was intended to represent the interaction of diaphragm inertial forces and rocking effects in the piers. It was not necessarily intended to represent low-amplitude response where a linear, dynamic analysis model would suffice. Therefore, the same degree of freedom was assigned to each of the two diaphragms. This assumption was justified because the nonlinear deformation was concentrated in the first story, and the stiffness and mass were the same for each diaphragm. In addition, degrees of freedom were assigned at the top of the first story for each wall to represent differential wall motions. A computer program was written to integrate the equations of motion for each time step based on assumed force-deflection curves for each wall and the diaphragm, and base motions as recorded during the earthquake simulation tests. Diaphragm elements were modeled as linear to match the steel beams of the test structures. Shear walls were modeled as elastic, but nonlinear to represent an idealized rocking behavior after cracking.

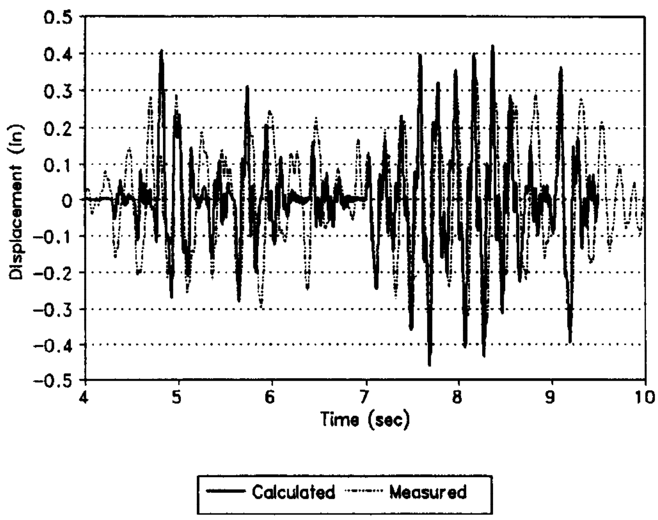


Fig. 6 Computed vs. measured first-story deflections of S1 door wall (run 15)

A sample comparison of computed and measured wall displacements for the door wall of test structure S1 during test run 15 is shown in Fig. 6. Portions of the measured record where rocking occurred were reproduced well with the computational model including the sequence, frequency content and amplitude of waveforms. The simple three-degree-of-freedom system provided an accurate depiction of peak displacements, instants when rocking commenced, reductions in diaphragm displacements with rocking, and frequencies of the system with rocking. Shear forces were not modeled as accurately as displacements which is common with a time-step integration approach. Because the walls were modeled with an elastic, nonlinear behavior, no accumulation effects were represented.

SUMMARY AND CONCLUSIONS

A brief summary of modeling concerns for unreinforced masonry buildings with flexible diaphragms has been presented based on dynamic response measurements of two reduced-scale, two-story brick building systems subjected to simulated earthquake motions on a shaking table.

Findings from several investigations have been collected to provide insight to modeling of masonry buildings for seismic evaluation and loss assessment studies. The following conclusions were made:

- An equivalent base shear analysis will be conservative because the total building weight is used instead of the effective modal weight, and the wall and diaphragm inertial forces are not necessarily concurrent. Force reduction factors as inferred from the tests exceeded four for the unreinforced masonry shear wall systems.
- A linear dynamic analysis will depict different amplifications of base accelerations for shear walls and flexible diaphragms of unequal frequencies, and will characterize the effective mass vibrating with a particular mode.
- A nonlinear static analysis will represent the progressive softening of unreinforced masonry shear walls if a nonlinear finite element program is used with realistic estimates of post-cracked behavior.
- A push-over analysis of a flexible diaphragm system may be conservative because the phasing of wall and diaphragm inertial forces is not represented.
- A push-over analysis of a rocking-pier system may lead to meaningless results because lateral deflections are not necessarily related to imposed strains or damage.
- A nonlinear dynamic analysis is needed to represent the interaction of flexible diaphragms with rocking piers.

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