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Pseudo dynamic tests of confined masonry buildings

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ABSTRACT: Pseudo dynamic tests were carried out to investigate the behavior of confined masonry structures. Two-story one-bay specimens were used for the tests here reported. One full scale specimen and one half scale model were built following current practices. The half scale model was identical to two other specimens previously tested statically and in a shaking table. This paper summarizes results from the pseudo dynamic tests and compares them with those obtained using different experimental techniques. There is reasonable agreement between test results; however, pseudo dynamic tests tend to give lower strength values due to stress relaxation.

1 INTRODUCTION

Most housing units in urban areas of Latin America are masonry structures. With few exceptions, four to five story masonry buildings have been used for all large housing projects of the past decades. Since this trend is expected to continue, a substantial part of the work at research institutions in the region deals with masonry.

This paper summarizes results from tests of confined masonry structures made at the Japan Perú Center for Earthquake Research and Disaster Mitigation (CISMID) of the National University of Engineering, in Lima, as part of a joint research program with the Pontificia Universidad Católica del Perú (PUCP). The term confined masonry is used here to denote a building system with clay brick load bearing walls confined by reinforced concrete elements.

The main objective of the research program was to compare different testing techniques to simulate earthquake loads.

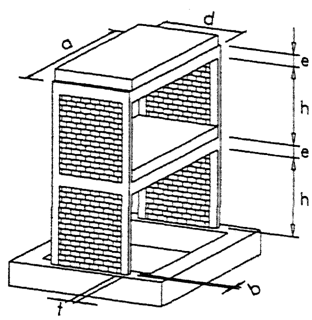

Figure 1. Test specimen.

Table 1. Specimen dimensions (cm)

	a	d	h	e	b	t
Full scale	247	214	210	20	25	14
Half scale	247	107	98	15	10	7

2 TEST PROGRAM AND PROCEDURES

2.1 *Specimens*

Two-story one-bay specimens, with two parallel walls connected by stiff horizontal slabs, were used for the tests. Design was based on Peruvian standards for confined masonry. One full scale specimen (tested at CISMID) and 3 approximately half scale models (two of them tested at PUCP and one at CISMID) were built.

Figure 1 and Table 1 show the dimensions of the specimens. Slab dimensions were not in the ratio 1:2. This was corrected by adding masses to the full scale specimen, in order to have the same vertical stress.
Scale factors were 2 for displacement, 1 for acceleration as for strain, angular distortion, stress and elastic moduli, $\sqrt{2}$ for time, 4 for mass and force.

The mass of the full scale specimen was 15,26 t (including added masses of 2,54 t on each slab), with a ratio of 4 with respect to the original half scale model. However, operational limitations of the shaking table used in one of the tests made it necessary to decrease the fundamental frequency of the half scale models. Additional masses of 1,42 t were added for that purpose at each floor level. The same masses were added to the other half scale models tested with different techniques.

2.2 *Materials*

Nominal dimensions of the clay brick units used in the full scale specimen were 140 mm x 140 mm x 280 mm, with a net area of 61%. They were laid with a 1:4 (cement : sand) mortar, with an average joint thickness of 10 mm.

The units used for half scale models were cut from 60 mm x 120 mm x 250 mm bricks using a mechanical saw. Final dimensions of the cut units were 60 mm x 70 mm x 120 mm, with a 66% net area. The average joint thickness was 5 mm. The mean prism strength was 10,8 MPa.

The concrete used for footings, columns and slabs had a nominal strength f'c of 20 MPa.

Columns were reinforced with 4#3 longitudinal bars and stirrups #2 at 14 cm, except near the joints, were a 10 cm spacing was used. The yielding stress of the steel was 410 MPa. In the half scale models the columns were reinforced with 4#2 bars; stirrups had a net area of 12 mm and a yielding stress of 200 MPa.

2.3 *Test Program*

The joint CISMID-PUCP research program included:

1. Static test of half scale model under monotonic loading, with equal horizontal forces at both floor levels.
2. Shaking table test of half scale model, with harmonic base acceleration of varying amplitude.
3. Pseudo dynamic (PD) tests of one full scale specimen and one half scale model.

This paper deals mainly with the PD tests conducted at CISMID; results from preliminary forced vibration tests are also presented. Results from the static and shaking table tests conducted at PUCP have been reported by San Bartolomé et al. (1991); some of their findings are quoted here for purposes of comparison.

3 FORCED VIBRATION TESTS

3.1 *Equipment and instrumentation*

Steady-state resonance tests were performed using a small rotating eccentric weight exciter. The exciter was set at the center on the top slab of the specimen being tested, producing a horizontal sinusoidal force, parallel to the walls.

Four accelerometers were located on the slabs, near the center of each wall, with their sensitive axis horizontal, parallel to the walls. The analog signals were filtered with 100 Hz low-pass filters and measured with a digital storage oscilloscope.

3.2 *Natural periods, damping and modal shapes*

Figure 2 shows typical resonance curves, obtained for the full scale model. Damping was estimated by considering the specimen as a one degree of freedom system and using the bandwidth (half power) method.

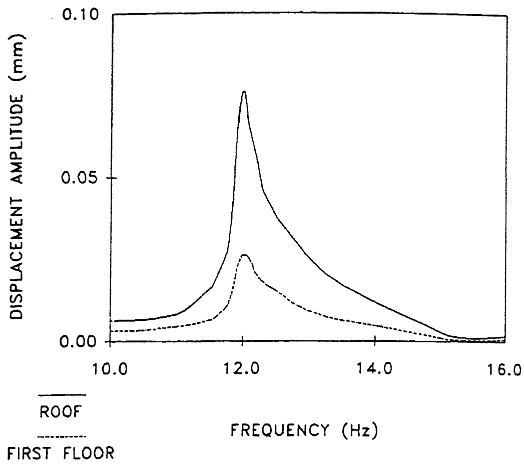

Figure 2. Resonance curves for full scale specimen.

Table 2. Natural periods, frequencies and damping

	f (Hz)	T (sec)	ß (%)	u max (mm)
Full scale specimen				
First mode	12,5	0,080	1,5	0,038
Second mode	28,3	0,035		0,021
Half scale model				
First mode	13,5	0,074	1,5	0,097
Second mode	18,5	0,054		0,018

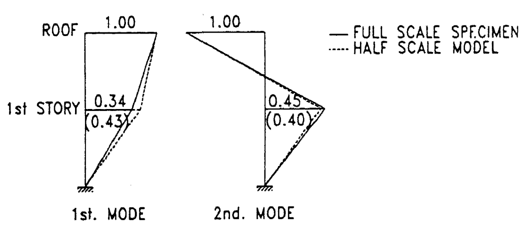

Figure 3. Modal shapes.

Table 2 lists natural periods, frequencies and percentage of critical damping of the specimens tested at CISMID. The maximum average displacement at the top of the specimen is also tabulated.

Modal shapes were obtained from the ratios of acceleration amplitudes at resonance. There were differences as large as 30% between accelerations at the top of both walls; to be consistent with the hypothesis of a 2 DOF model used in the PD tests an average value was considered.

Considering the fundamental period of the full scale specimen and taking into account the time scale factor and the additional masses in the half scale model, the fundamental period of the latter should be 0,077 sec.

The 4% difference may be due to material properties and workmanship.

An average fundamental period of 0,05 sec was reported by PUCP researchers for the half scale model which was tested statically. This result was obtained from free vibration tests at the microtremor level; hence the large difference may be explained by the nonlinear behavior of the specimens.

For the other half scale model, used for the shaking table test, a fundamental period of 0,062 sec was reported. This result was obtained from free vibrations after step base motions, with maximum base accelerations of the order of 0,1 g.

4 PSEUDO DYNAMIC TESTS

4.1 *Implementation at CISMID*

Pseudo dynamic tests combine analytical and experimental techniques. They can be considered as a numerical integration of the equilibrium differential equations, except that restoring forces are directly measured from the specimen rather than computed using a mathematical model.

Different systems interact during PD tests. At CISMID two separate micro computers control the integration of the equations and the data acquisition. Displacements or loads are applied to the specimen by means of servo hydraulic actuators. Load cells, LVDTs and strain gages are used as transducers. The interaction of computers and mechanical equipment is through 12 bit A/D and 16 bit D/A converters, connected by GPIB.

The software allows for tests with an arbitrary number of degrees of freedom. A summed form of the central difference method is used; an analysis of this procedure may be found in Shing et al. (1983).

The tests are usually controlled by "external" LVDTs attached to the specimen. Since the actuator displacement ("internal" LVDTs) may be different from that of the specimen, the software provides for a correction, imposing target displacements slightly different from those obtained from numerical integration. To avoid instabilities, the correction is numerically damped by averaging the difference between internal and external LVDT readings with the correction considered in the previous iteration.

There is good agreement between theoretical and PD test results. For example, figure 4 compares time histories for the top displacement in the half scale model resulting from a 0,1 sec pulse of 100 gal. The fundamental period from the PD test was 0,075 sec, almost the same obtained with forced vibration tests. The theoretical time history was computed for a 2 DOF system, with the same lumped mass matrix used for the PD test. Viscous damping was assumed as 2,5% of critical. The stiffness matrix was obtained from previous measurements of flexibility coefficients.

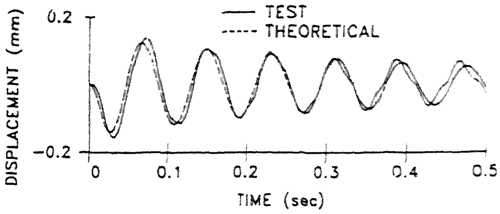

Figure 4. Experimental vs theoretical results.

However, it should be pointed out that when running a PD test with an earthquake record large relative errors occur at the beginning of the test, while displacements are small, sometimes of the same order of magnitude of the digitizing error.

4.2 *PD test conditions*

The specimens were modelled as 2 DOF systems with lumped masses. Perfect symmetry was assumed, hence the degrees of freedom were associated to the horizontal displacements at the center of each slab.

Preliminary tests were conducted to obtain flexibility matrices of the specimens. For this purpose a small load, of the order of 1 kN, was applied at one of the floor levels while the other floor level was free. The fundamental periods computed from these results were 0,086 sec for the full scale specimen and 0,067 sec for the half scale model.

Viscous damping was introduced in the form of a matrix proportional to initial stiffness, with 1,5% of critical damping for the first mode, and remained unchanged during the test.

For the PD test of the half scale model the input signal was the same used for the shaking table test. It consisted of a series of 5 Hz sine waves with different amplitudes (figure 5). During the first stage of the test, while the specimen (with a 13,5 Hz fundamental frequency) had little damage, this 5 Hz base motion was equivalent to a static loading. The maximum acceleration in the input signal is 1,3 g, although the specimen failed during the stage with maximum acceleration of 1,06 g. The integration time interval was 0,004 sec. The total duration of the test was about 16 hours.

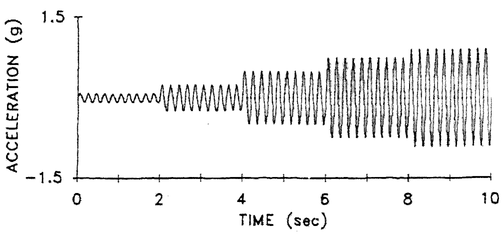

Figure 5. Input signal for PD test of half scale model.

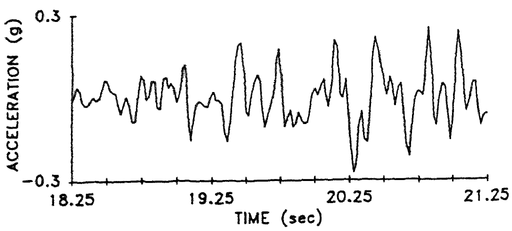

Figure 6. Input signal for PD test of full scale model.

The full scale specimen was tested with a ground acceleration corresponding to the N08E component of the Lima earthquake of October 10 1966. A portion of the record, from 18,25 to 21,25 seconds, which contains the peak acceleration, was selected (figure 6). The test was repeated four times, scaling the record as required to have a maximum ground acceleration of 293,6 gal (original record), 400, 800 and 1200 gal. The integration time interval was 0,004 sec. The average duration of these tests was 12 hours.

4.3 *Test results for half scale model*

Thin cracks were observed at the base of the walls from the beginning of the test. Diagonal cracks developed during the second stage and became increasingly important after 4 seconds. A large strain increment in the longitudinal steel reinforcement of the columns occurred at the same time. The failure mode was by shear, involving both masonry units and mortar joints. Diagonal cracks were also observed in the second level, in contrast with what was reported for the static and shaking table tests. The higher damage in the second level was clearly related with construction defects in one of the walls.

A plot of base shear versus first story displacement is shown in figure 7. The behavior was almost linear during the first two stages of test, while the story drift angle was less than 1/1000. Stiffness degradation and hysteresis were important from the third stage.

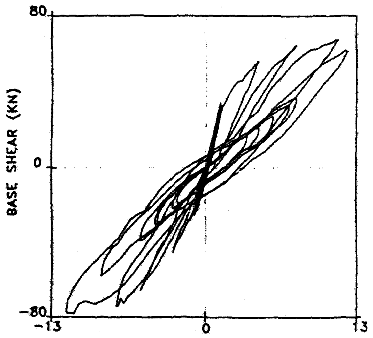

Figure 7. Base shear vs first story displacement, pseudo dynamic test of half scale model.

The maximum base shear was 78,3 kN, corresponding to a 0,0118 angular distortion. The maximum average shear stress was 0,52 MPa.

The relationship between critical damping ratio and displacement amplitude for each cycle is illustrated in figure 8. As expected, hysteretic damping increases with displacement.

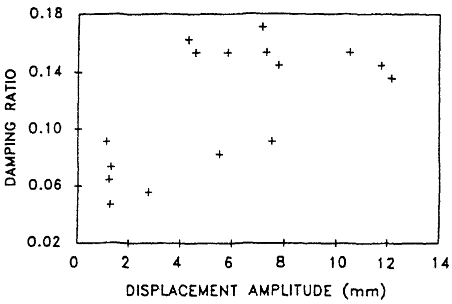

Figure 8. Critical damping ratio as a function of first story displacement amplitude.

Figure 9 compares envelope curves from static and dynamic tests, reported by San Bartolomé et al. (1991), with those from the PD test. Good agreement was found between results of static and shaking table tests. Lower values obtained in the PD test; this may be due to strain rate.

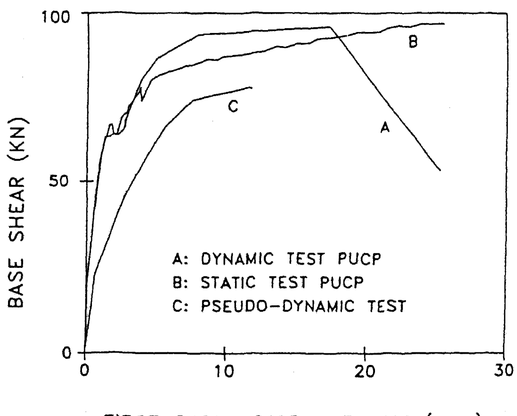

Figure 9. Envelopes of base shear vs first story displacement, half scale models.

4.4 *Test results for full scale specimen*

The behavior of the full scale specimen was similar to that observed in the static and shaking table tests of half scale models. The failure mode was by shear. Cracks were noticeable in the first story walls after the 400 gal earthquake. The second story walls remained practically undamaged.

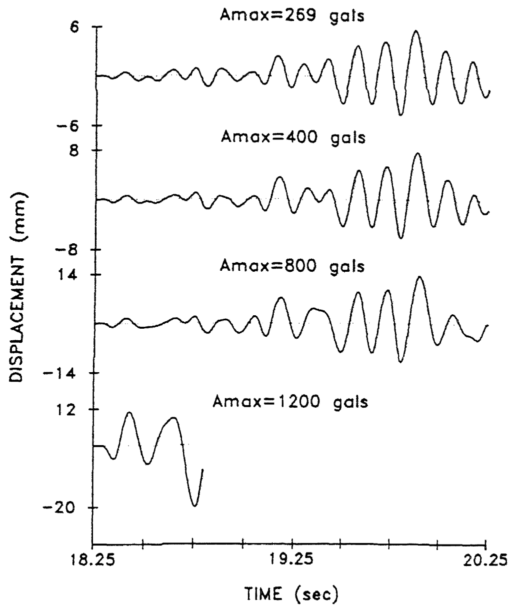

Figure 10. First story displacement time histories, full scale specimen.

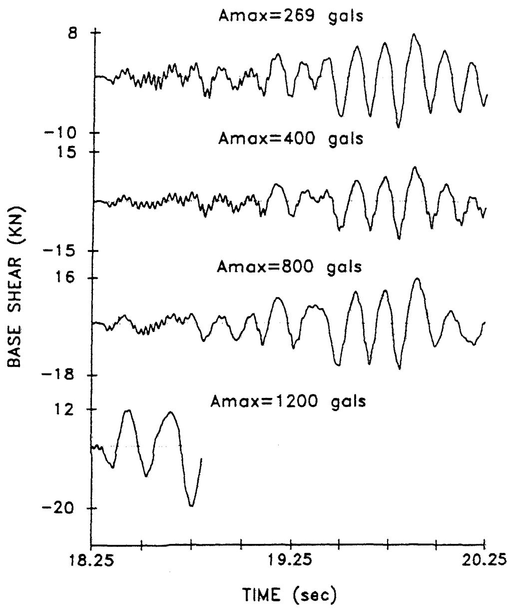

Figure 11. Base shear time histories, full scale specimen.

Table 3 lists maximum first floor displacement (u), maximum base shear (V), and predominant response period (T), for different levels of ground acceleration (a). Although maximum displacements and base shears correspond to only one point of each record, their relative magnitudes and the period elongation reflect the importance of nonlinearities in the response.

Table 3. First floor displacement, base shear and predominant period as a function of maximum ground acceleration.

a max (gal)	u max (mm)	V max (kN)	T (sec)
269	2,5	89	0,14
400	3,6	104	0,15
800	10,8	157	0,19
1200		192	0,28

The specimen failed at an average shear stress in the first level of 0,32 MPa, considerably lower than that reached by the half scale model. The allowable design stress in the current Peruvian code is 0,16 MPa. Part of the difference between experimental results may be explained by the vertical stress (0,23 MPa in the full scale specimen, 0,35 MPa in the half scale model). Other possible cause is the different frequency content of the input signals. Further research is needed to clarify this point.

5 CONCLUSIONS

Reasonable agreement was found between results using different testing techniques, although a lower strenght was obtained from pseudo dynamic tests, possibly because of stress relaxation effects. Additional research is required to incorporate proper corrections while integrating the equilibrium equations in pseudo dynamic tests.

The behavior of confined masonry structures is markedly nonlinear, hence seismic analysis and design procedures should consider material properties consistent with the expected strain level.

If nonlinearities in the behavior are taken into account, there is consistency between results for fundamental periods obtained from free vibration, forced vibration and pseudo dynamic tests.

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