VARIATION OF DYNAMIC PROPERTIES OF MEXICAN LOW-COST HOUSING BUILDINGS WITH LEVEL OF SEISMIC MOTION

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ABSTRACT:

In order to evaluate the dynamic behavior of housing buildings made of confined masonry in Mexico, a series of shaking-table tests on small-scale models were conducted at the Institute of Engineering at the UNAM. Specimens were half-scale models made of handmade solid clay bricks. A similarity model for ultimate strength was selected as the basis for scaling. These were subjected to a series of seismic motions typical to the epicentral region along the Mexican Pacific. The experimental response of one-, two- and three-story models is compared. Resistant mechanisms were identified through the analysis of recorded and observed results. Structural capacity was assessed in terms of strength, stiffness, deformation capacity and energy dissipation. Shear deformations controlled the response of the models. Based on the failure mode observed, the analytical model for design and assessment could be simplified by assuming that all inelastic deformations would take place at the first story which performed as a soft-story with a shear governed mechanism.

KEYWORDS: shaking-table tests, confined masonry structures, solid clay bricks, dynamic behavior.

1. INTRODUCTION

Masonry, concrete and steel are the most common materials used for construction in Mexico. Confined masonry, CM, is a traditional method for low-cost housing projects still in seismic regions. It consists of load-bearing walls surrounded by small cast-in-place reinforced concrete columns and beams, hereafter referred to as tie-columns, TC´s, and bond-beams, BB´s, respectively.

In recent decades, a comprehensive research program of quasi-static tests of isolated masonry walls, systems of walls and three-dimensional confined masonry structures, CMS, in natural scale was carried out in Mexico. A few experimental research studies have attempted to assess the dynamic response of CMS. Previous efforts have focused on tested confined masonry systems of walls subjected to harmonic excitation [Alcocer et al, 1996]. To obtain more evidence about the dynamic response of CM, a series of tests were conducted at the shaking table facility of the Institute of Engineering at UNAM. The research program involved construction and testing of one-, two- and three-story small scale specimens in which the walls' dimensions, layout and details are comparable to typical prototypes. All these structures were built to half scale.

This paper reports on the response of the one, two and three-story models, hereafter referred to as M1, M2 and M3, respectively. Model M2 was created after the modification caused by testing M3 (this modification was the demolition of the first story).

2. EXPERIMENTAL PROGRAM

2.1. Description of the models

The shaking table system at UNAM is capable of controlling five degrees of freedom and operating in frequencies ranging from 0.1 to 50 Hz. Due to the physical characteristics of the table (size is 4.0 x 4.0 m, and the maximum weigh of the specimens is 196 kN), models were such constructed that the materials for both, the model and the prototype, were identical, following the concept of simple similarity, [Tomaževic and Velechovsky, 1992]. Structures dimensions and reinforcement layout are shown in Figure 1.

Walls that were made of handmade solid clay bricks were confined by reinforced concrete TC´s and BB´s. In the direction of the earthquake simulator motion (E-W), three wall axes were built. The facade's walls had door and window openings, whereas the middle walls didn't. In the prototype, the middle wall axis divides the two adjacent dwellings. In the transverse direction (N-S), four walls were built to improve the gravity load distribution among walls, and to control possible torsional deformations. Floor systems that were cast-in-place are reinforced concrete solid slabs supported on BB´s.

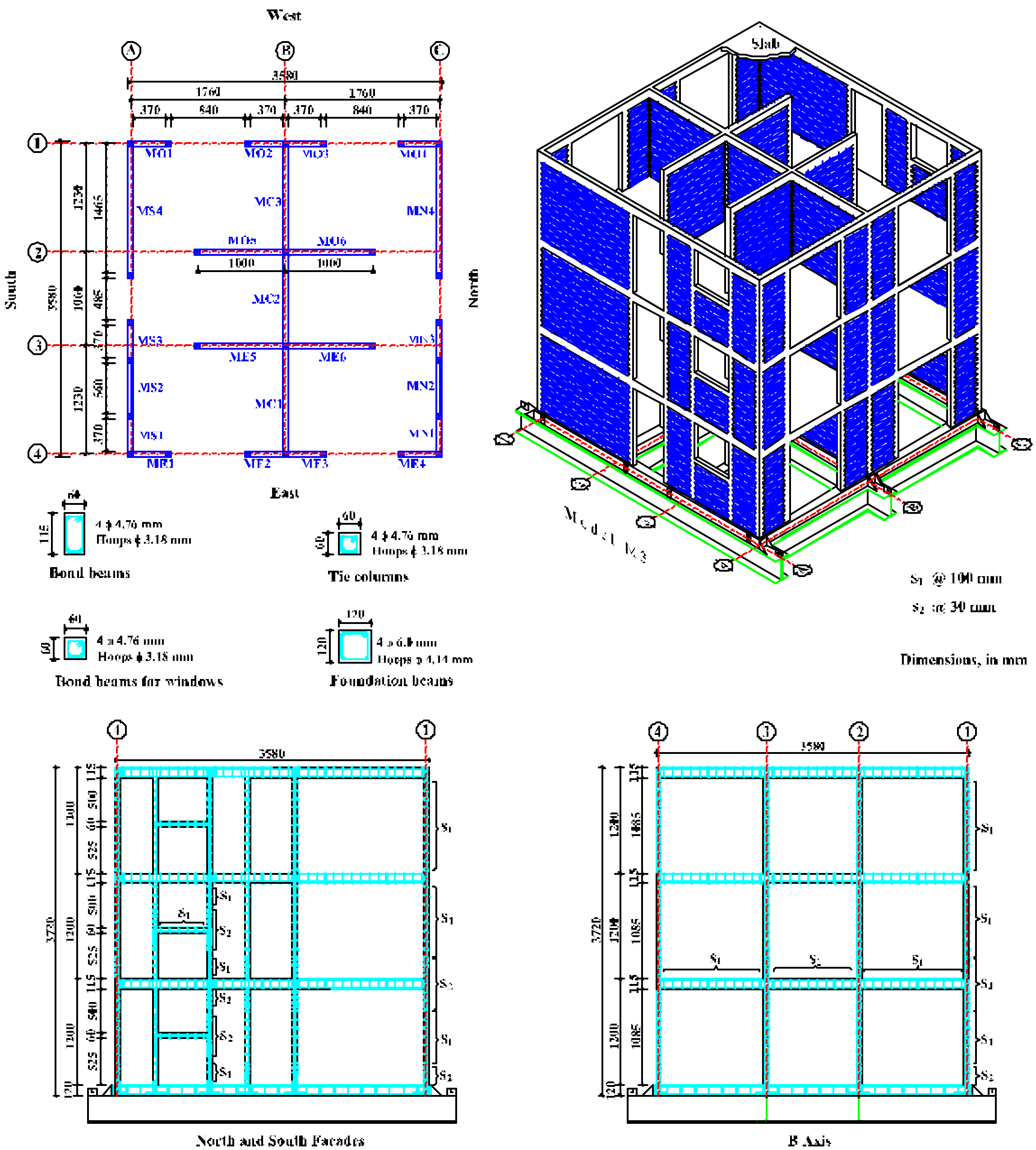

Figure 1. Characteristics of the specimens

2.2. Instrumentation and test program

Two earthquake motions recorded in epicentral regions in Mexico were used as basis for the testing program. One was recorded in Acapulco, Guerrero, in April 25, 1989 (M=6.9 earthquake with PGA=0.34g). The other was recorded in Manzanillo, Colima, in October 10, 1995 (M=8.0 quake with PGA=0.40g), [Sociedad, 1997]. Both records were considered as Green's functions to simulate larger magnitude events.

All models were subjected to a sequence of seismic excitations by increasing gradually the intensity of the motion at each test run up until the final damage state was attained. A total of 28, 7 and 13 test runs were applied to M1, M2 and M3, respectively. Between each test run, a random acceleration signal (white noise) of 50 cm/s^2 (0.05g) was applied to identify the changes in the dynamic properties.

Because expected performance levels were not attained during the testing, caused for the large lateral strength and stiffness of the specimens, several modifications were done to the models.

The phases of testing in M1 were: original model (M-1); walls MC1 and MC3 were fully eliminated (M-1M); walls ME5, MO5, ME6, MO6 were cut into small narrow walls (M-1A); a 36 percent of extra mass was added (M-1B); and the last modification was an addition of 11 percent of extra mass (M-1C). In M2 since the beginning the walls MC1 and MC3 were eliminated for both floors. For M3 two modifications were done: original model (M-3) and just the walls MC1 and MC3 of the first floor were eliminated (M-3M).

3. TEST RESULTS

To facilitate comparison among these specimens and among others, tested under dynamic or static conditions, four limit states were defined: initial, I, (when the test was started); elastic, E, (first diagonal cracking in the masonry wall); maximum or strength, M, (maximum base shear was resisted); and ultimate, U, (last test run, or the highest lateral drift ratio for the first story). Test results for prototypes are summarized in Table 1, where M1, M2 and M3 represent prototypes of one, two and three story, respectively.

Table 1. Response characteristics for prototypes of one, two and three story

Model	Limit State	V_b [kN]	? [%]	c [g]	D. A.	C. E. D. [kN-m]	f [Hz]	? [%]	K_p/K_0	M_T^1 [kN-m]	M_T^2 [kN-m]	e [m]
M1	I	340.1	0.12	0.8	1.15	8.2	13.8	7.3	1.00	38.8	243.5	0.12
	E	776.2	0.34	1.9	1.56	367.2	8.9	17.6	0.54	226.2	555.8	0.31
	M	1182.5	0.67	2.1	1.48	1252.8	4.8	26.5	0.36	807.3	846.7	0.76
	U	1030.0	1.83	1.8	1.35	2054.5	1.8	-	0.10	1179.9	737.5	1.28
M2	I	804.2	0.14	1.2	1.43	33.2	6.8	2.3	1.00	140.1	575.8	0.20
	E	844.6	0.18	1.3	1.38	87.8	6.5	5.5	0.81	170.8	604.8	0.20
	M	1159.8	0.94	1.7	1.84	654.9	4.1	11.2	0.22	454.6	830.4	0.43
	U	1213.0	1.30	1.8	1.64	961.0	2.6	12.1	0.16	399.7	868.5	0.41
M3	I	851.5	0.16	0.8	1.47	35.6	5.1	4.6	1.00	187.7	609.7	0.22
	E	972.5	0.23	0.9	1.58	206.8	5.0	5.4	0.74	301.9	696.3	0.34
	M	1178.6	0.42	1.1	2.08	1994.6	4.3	7.6	0.54	405.0	843.9	0.34
	U	703.0	1.70	0.7	1.30	2654.1	2.1	10.2	0.09	328.2	503.3	0.49

V_b. Base shear
?. Lateral drift ratio of the first story
c. Seismic shear coefficient
D. A. Dynamic amplification at top slab
C. E. D. Cumulative energy dissipated
f. Frequency
?. Viscous damping
K_p/K_0. Stiffness degradation of the first story
MT^1, MT^2. Experimentally obtained and calculated torsional moment
e. Accidental eccentricity

3.1. Crack patterns

Final patterns of cracking are shown in Figure 2. Models with their original wall configuration suffered only minor cracking, suggesting a behavior within the elastic range. At this stage, the response was characterized by horizontal cracks at the base of the walls. After removing the wall above mentioned, and at larger intensity motions, the damage in specimens was characterized by inclined cracking of the N and S facades. Simultaneously, horizontal cracks were observed, uniformly distributed over the TC´s and walls on the E and W sides. Slabs also showed perpendicular cracking to the direction of the base motion which is attributed to slab bending at the door openings. At the end of the tests, damage was characterized by the crushing of the masonry walls, cracking and crushing of the concrete TC´s and by kinking of the longitudinal steel at the TC´s ends (dowel action). Similarly, shallow cracks in walls MO1 and MO4 at the West side, and out-of-plane sliding in walls MS4 and MN4 were observed, which suggests that some torsional response took place.

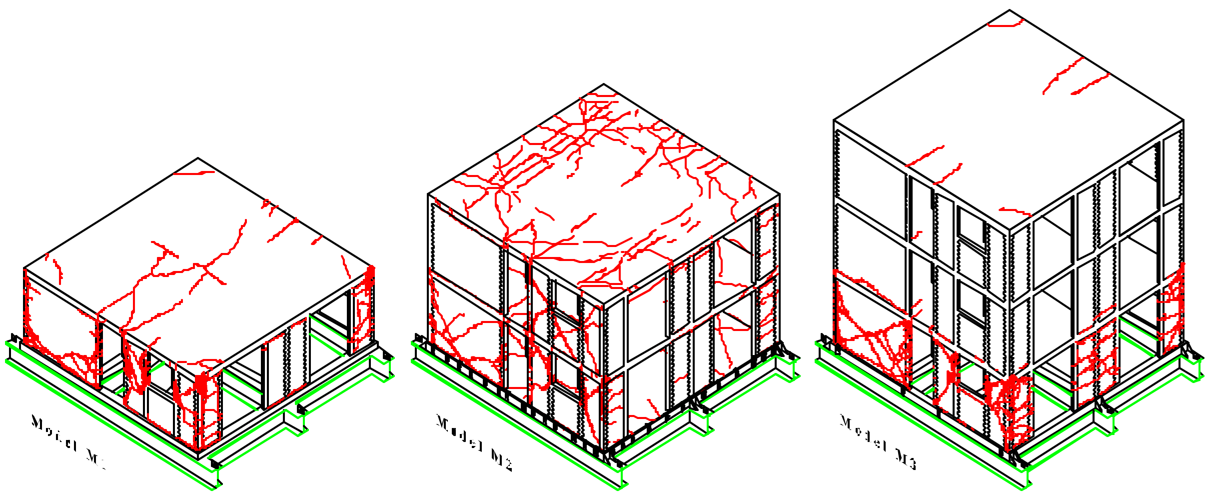

Figure 2. Final cracks patterns

3.2. Hysteretic curves

The hysteretic curves in terms of the base shear and lateral drift ratio at the first story are shown in Figure 3. It is also drawn on it, the strength predictions using the Mexico City Building Code requirements, MCBC, [Gobierno, 2004]. The envelope curves are drawn with different colors and markers depending on the specimen wall configuration. The base shear was calculated from measured accelerations at each floor slab center of gravity and by considering the specimen's mass and extra mass from lead ingots. Hysteretic loops were typical of CM structures.

Cycles within the elastic limit experienced some hysteresis attributed to wall flexural cracking at initial stages. As it is common in CMS, specimens attained their strength to higher loads than those that are associated to the first inclined cracking. As it is customary in shear-governed members subjected to inelastic deformations, response curves exhibited severe pinching, especially at very large lateral drift ratios associated to failure of the structure. In M3 at the ultimate limit state, a fast degrading process, involving sliding along the first story, inclined cracking and crushing of masonry and concrete, was clearly observed. It was apparent that stories 2 and 3 laterally deformed very slightly, suggesting a rigid body motion over the first story. This phenomenon led to a concentration of deformations and damage at the first story which performed as a soft-story with a shear-governed mechanism.

Comparing the shape of the envelope curves, as well as the force and drift ratio values for the four identified limit states, the similarity is apparent in the response. The latter supports the idea that the performance of M2 and M3 was controlled by the first story which was characterized as in M1, by shear deformations.

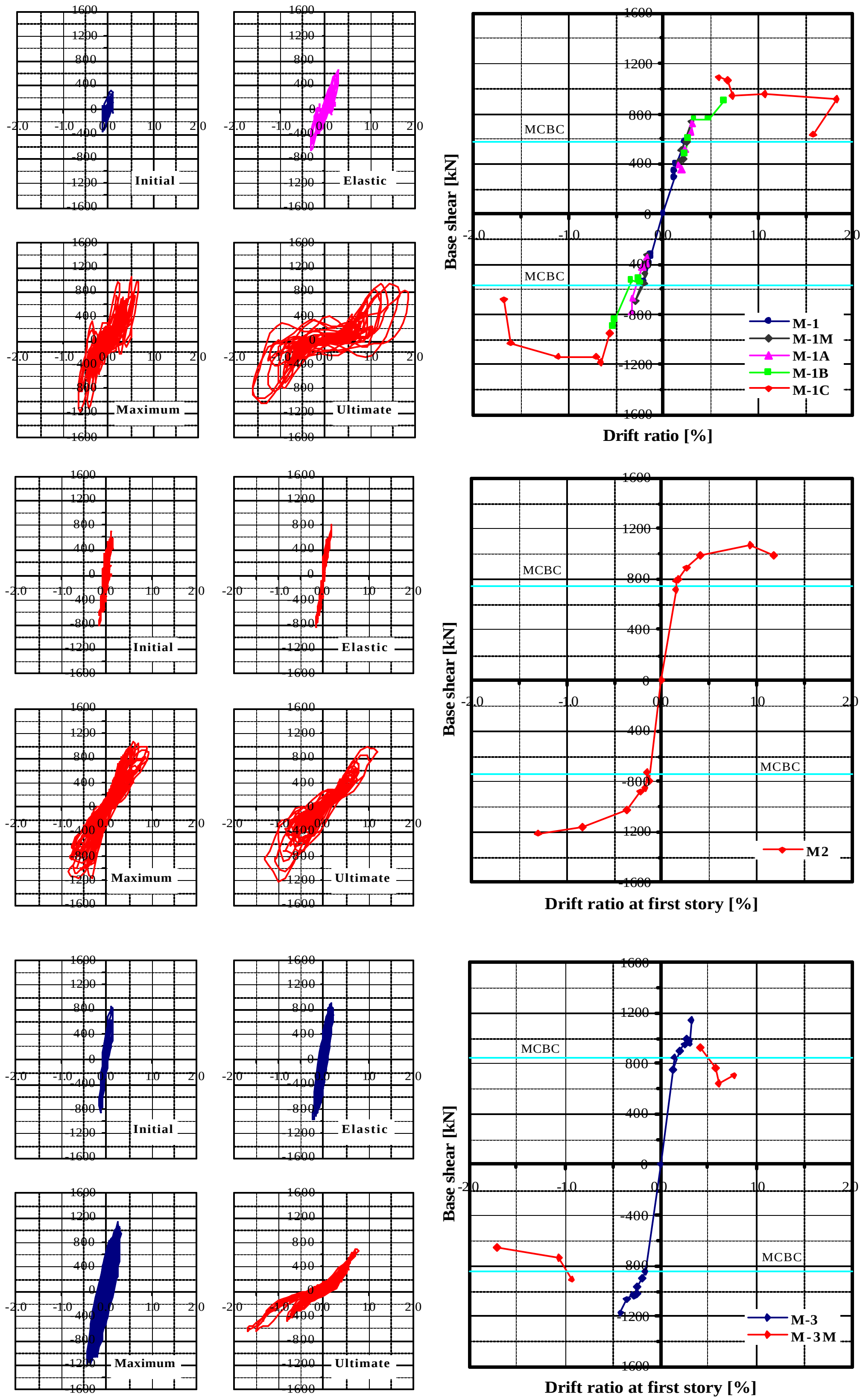

Figure 3. Hysteretic curves and response envelope

3.4. Seismic shear coefficients

The tested models represented a realistic prototype, so that the results obtained could be directly applied to real CMS [Krawinkler and Moncarz, 1982]. Seismic shear coefficients and first story drift ratios at the limit states selected for prototypes are shown in Figure 4. For comparison, the results of two-story full-scale 3D structures tested under static lateral loading are also included [Sánchez, 1996].

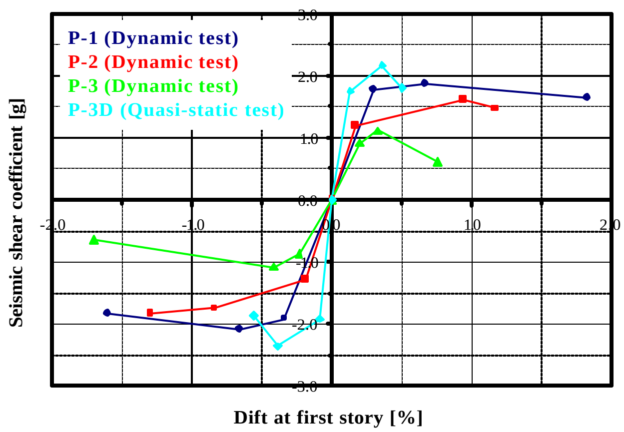

Figure 4. Seismic shear coefficient for prototypes

3.5. Stiffness degradation

To assess the stiffness degradation phenomenon, peak-to-peak stiffness, K_p, it was calculated. Normalized peak-to-peak stiffness and first story drift ratio curves for prototypes are shown in Figure 5. The peak-to-peak stiffnesses were normalized with respect to the initial stiffness, K_p/K_0.

Stiffness decay was observed at low drift ratios, even before first inclined cracking became visible. This phenomenon is attributed to incipient wall flexural cracking, and perhaps, to some micro-cracking in masonry materials, local loss of mortar bond and adjustment of brick position. After first inclined cracking, but before reaching the strength, the decay increased with the drift ratio. At larger drift ratios, K_p remained nearly constant. At this stage, stiffness decay is associated to cracking and crushing in masonry walls and the RC confinement members.

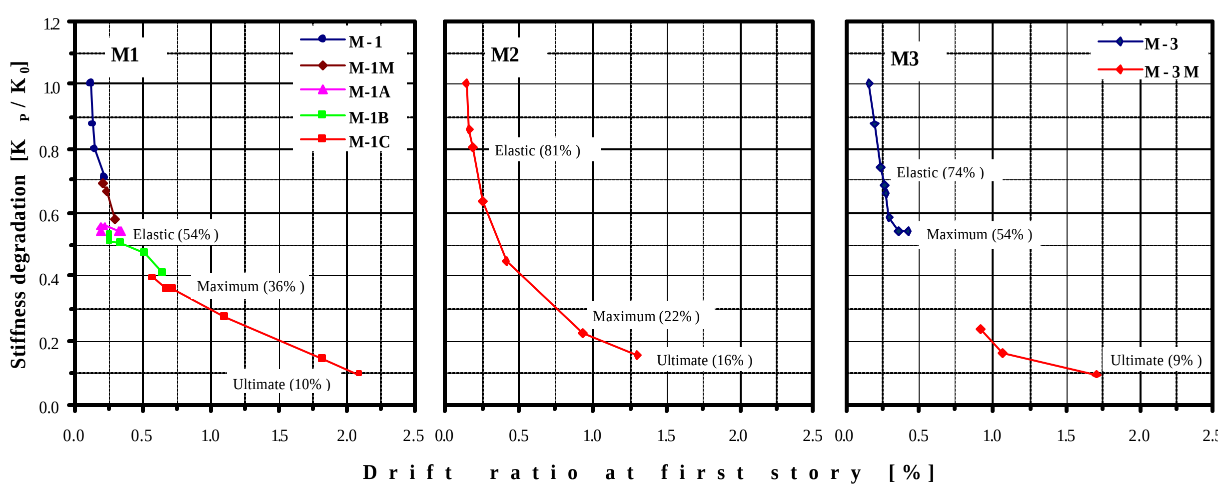

Figure 5. Stiffness degradation

3.6. Frequency and damping

The values of the first natural frequency of vibration and the percentage of viscous damping during the test runs are shown in Figure 6. It can be seen that as the natural frequency decays in correlation with the increasing amount of damage occurred to the models, the values of the viscous damping increases.

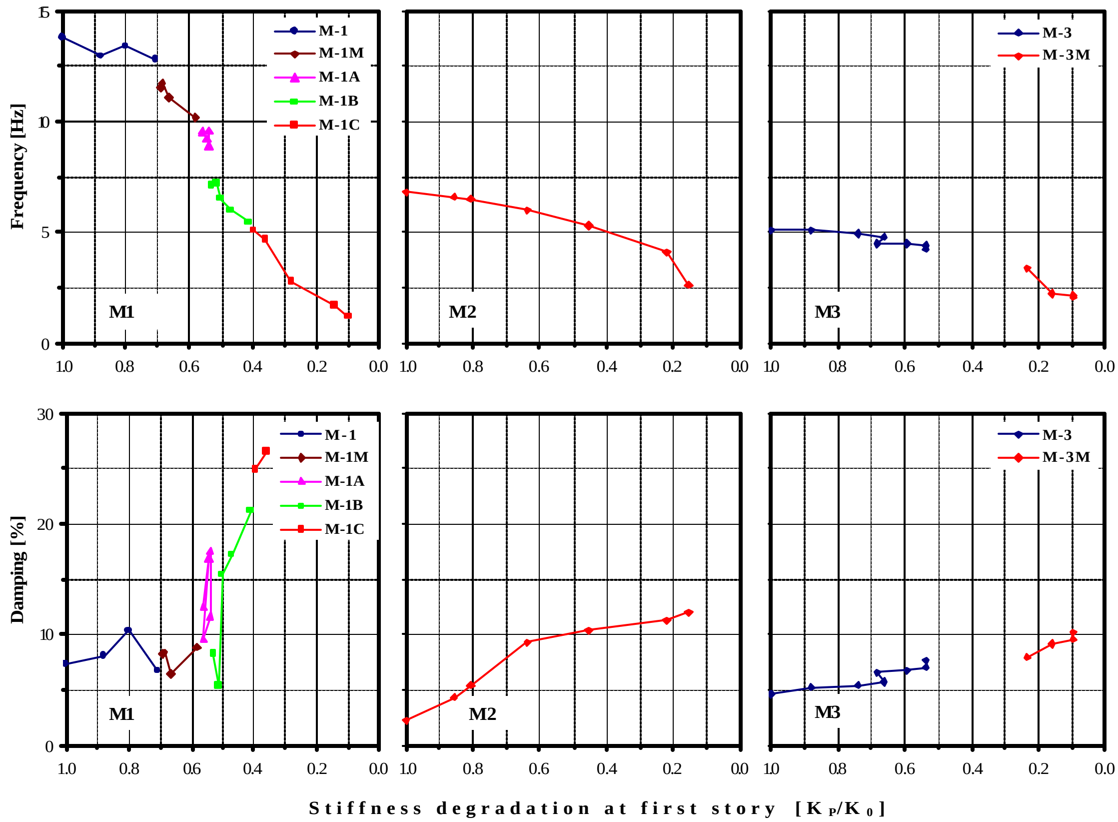

Figure 6. First natural frequency and percentage of viscous damping

3.7. Torsional response

Accidental eccentricity for models during test run is shown in Figure 7. The hysteretic curves in terms of torsional moment and base shear for maximum limit state, and torsional moment according to MCBC [Gobierno, 2004], are shown in Figure 8.

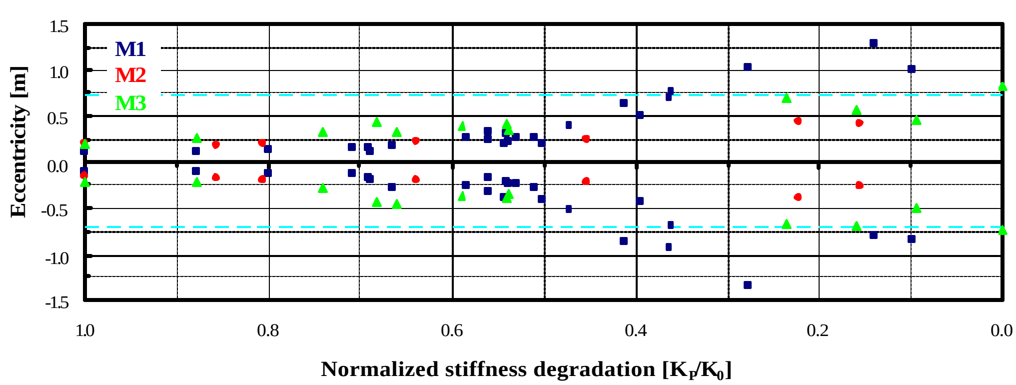

Figure 7. Accidental eccentricity

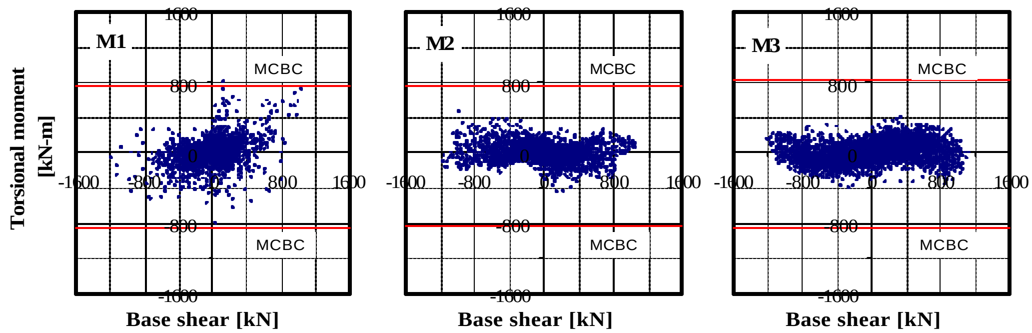

Figure 8. Torsional moment and base shear hysteretic loops for maximum limit state

CONCLUSIONS

Shear deformations controlled the response of the models. Damage was characterized by crushing of the masonry walls, cracking and crushing of the concrete TC´s and by kinking of the longitudinal steel at the TC´s ends. Based on the failure mode observed, the analytical model for design and assessment could be simplified by assuming that all inelastic deformations would take place at the first story which performed as a soft-story with a shear-governed mechanism.

Comparing the calculated and the measured strengths, it was found that the level of overstrengths were 1.97, 1.50 and 1.36 for M1, M2 and M3, respectively.

Lateral force distributions up to the elastic limit clearly followed the commonly assumed triangular shape distribution. In contrast, for higher lateral drift ratios, when the maximum load carrying capacity (strength) and ultimate were reached, force distributions changed by concentrating the forces in the first story, again suggesting the formation of a soft-story mechanism.

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