Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Experimental study on earthquake-resistant design of confined masonry structures

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ABSTRACT: Results of a testing program of confined masonry structures are presented. One story specimens were designed and constructed according to the present Mexican practice. The experimental variable was the flexural coupling between two wall panels, henceforth, having different openings between them (door or window). It is concluded that the degree of coupling did not influence the failure mode of the specimens, which was governed by shear failure of the masonry panels. Diagonal cracking strengths were similar to all models, regardless of the form of the opening. Maximum measured strength was 75% higher, on the average, than that obtained from current code recommendations.

1 BACKGROUND

Masonry has been the material most widely used for construction of dwellings in Mexico. Low-cost housing projects, developed by Mexican housing agencies, are constructed using traditional methods for confined masonry. The structural system 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 tie-beams (TB's), respectively. Therefore, walls must resist the lateral loads. The wall panel can be made with solid clay or concrete brick units, or with hollow pieces which are reinforced both vertically and horizontally. Tie-columns have a square section whose dimensions typically correspond to the wall thickness (15 cm in general). Similarly, TB's width is the wall thickness and the depth is usually equal to 25 cm. Typically, both TC's and TB's have longitudinal reinforcement ratios, based on gross sectional area, of 1.2%. Confined masonry is used in buildings up to five stories high. For these cases, the TC's have higher longitudinal steel quantities to resist the overturning moments. Tie-columns and TB's are intended to confine the masonry panel, thus improving the wall deformation capacity and the behavior under alternated lateral loads (as compared with unconfined masonry panels), and to improve the connections among other walls and floor diaphragms. The floor system generally consists of cast-in-place reinforced concrete slabs, but very often, prefabricated units are used (such as prestressed concrete joists or planks). Confined masonry has evolved from unconfined unreinforced masonry, which has shown poor seismic behavior (even for large wall thicknesses) due to lack of adequate connections with other walls and confinement. Confined masonry is not only used in Mexico; several European and Latinamerican countries (Italy, Chile, Peru, for example) use this structural system for dwelling construction.

Masonry construction is code-regulated since 1976 (DDF 1976). Since that time, the code requirements follow an ultimate-strength approach to estimate the vertical and lateral carrying capacities of wall units. Therefore, partial safety factors, i.e. load and strength reduction factors, were developed from a probabilistic basis. To design masonry structures up to 13 m high, the code allows the use of a "simplified method", which assumes that in-plane wall deformations are governed by shear, and that the distribution of ultimate shear stresses across the wall is uniform. The later premise is based on plasticity principles. The approach established in the Mexican code has served as the basis for other codes (INN 1986, INPRES-CIRSOS 1983).

Code regulations for masonry structures were developed from results of a comprehensive research program carried out at the National Autonomous University of Mexico, over 20 years ago, intended to improve design recommendations for masonry structures. A summary of the program results can be found elsewhere (Meli 1975). One part of the study was aimed at developing material tests, and at studying the variability of mechanical properties of masonry components (brick units and mortar) and that of masonry units. From this stage, recommended design values for compression and shear strength were developed based on a 98% probability of exceedance. The objective of the other phase was to analyze the behavior of masonry walls subjected to lateral loads, monotonic and cyclic (Esteva 1966), and to vertical loads (concentric and eccentric). Over 100 full-scale load-bearing and infill walls were tested to study the effect on the response of different testing conditions, materials, amount of interior reinforcement (for hollow pieces), vertical load, etc.

2 RESEARCH SIGNIFICANCE

The seismic behavior of confined masonry buildings has been, in general, satisfactory, particularly in Mexico City (Meli 1990). Nevertheless, significant damages have been observed in near-epicentral regions during strong ground shaking. New building code requirements, enforced after the 1985 Mexico earthquake (DDF 1987 & 1989), are more stringent than those of previous codes, thus forcing dwelling designs to be revised, and in most cases, to be modified substantially to comply with the code. Since, construction programs for multi-family low-cost housing buildings in Mexico, involve prototype designs, which are repeated several times, the impact on construction cost is very large.

A research project on low-cost housing is underway at the National Center for Disaster Prevention in Mexico. The research is aimed at assessing the seismic safety and includes both analytical and experimental studies of full-scale confined masonry structures. An evaluation of the mechanical properties of construction materials will be done and the effect of changes in reinforcement and detailing will be studied in large-scale structures. This paper reports on some results obtained from testing of three full-scale confined masonry specimens. Each model consisted of two wall units made with clay bricks, linked by slabs or by slabs and parapets. Structures were tested applying cyclic loads that simulated earthquake-type motions.

3 EXPERIMENTAL PROGRAM

3.1 Specimen dimensions and materials

Specimens were designed and constructed following the requirements of the Mexico City Building Code (DDF 1987 & 1989). Models were two walls coupled, in which the variable was the type of flexural coupling between them. Specimens are shown in fig. 1. The first specimen, W-W, practically lacked of flexural coupling; walls were only connected through high-strength Dywidag bars, that transferred the lateral force between the walls. In the second model, WBW, walls were linked by a cast-in-place reinforced concrete tie-beam and slab, thus forming a door opening. Higher coupling was provided for specimen WWW, which had a parapet between the walls, thus forming a window opening, and a parapet above the slab to simulate the boundary conditions of an identical second floor. Specimens were 5 m long; walls were 2.4 m and 1.6 m long, separated by a 1 m opening. Specimen height, measured from the foundation beam top surface to the top slab surface, was 2.5 m. The

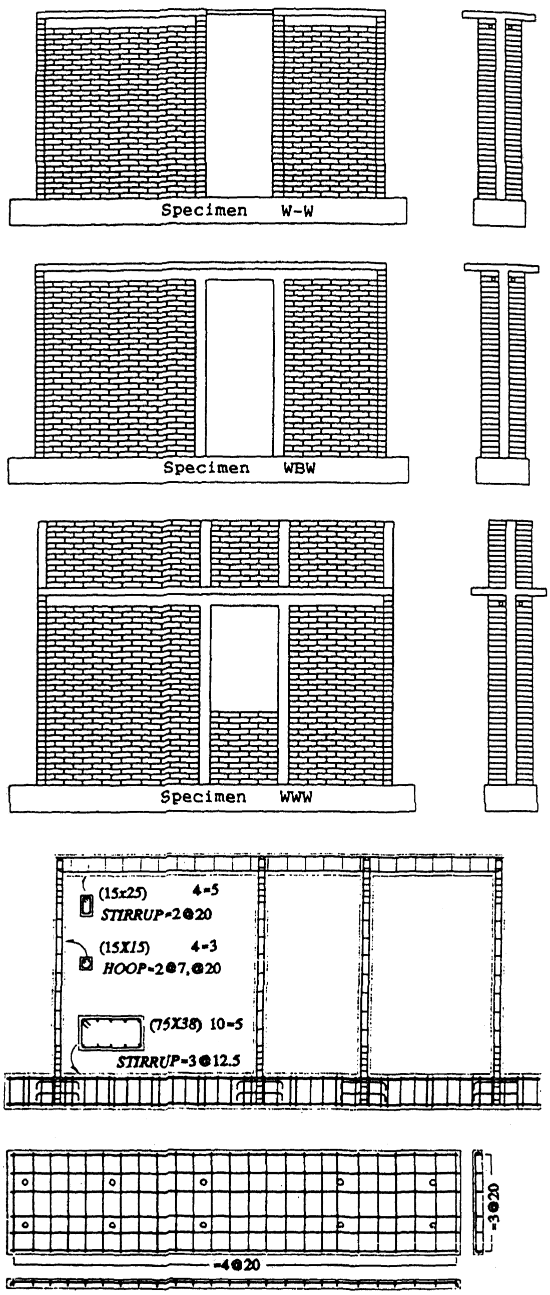

Figure 1. Specimen details.

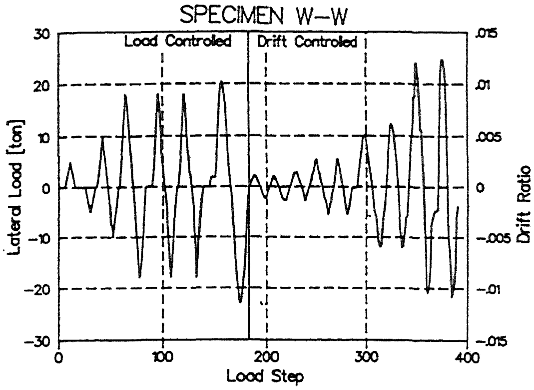

Figure 2. Loading history.

height of the second floor parapet for WWW was 1 m.

Walls were built with hand-made solid clay bricks. This material is typical for confined masonry in Mexico. Concrete mix design was for a compressive strength of 200 kg/cm². The mortar used to join the bricks had a cement:sand ratio of 1:3. The mortar was proportioned by volume and corresponds to that with a recommended compressive strength for design equal to 125 kg/cm² (DDF 1989).

Steel reinforcement with 4200 kg/cm² nominal yield strength was used throughout for the longitudinal reinforcement. For the tie-column and tie-beam hoops, #2 (0.6-cm diameter) mild steel, with 3000 kg/cm² nominal yield strength, was used. Dimensions and reinforcing details were the same of all specimens (fig. 1). Tie-columns with nominal sections of 15 x 15 cm, were reinforced with 4-#3 (0.95-cm diameter) longitudinal bars. The columns hoops were spaced at 20 cm, but spacing was reduced to 7 cm at the ends of the TC's over a 35-cm length. Tie-beams, with 15 x 25 cm section, were reinforced with 4-#4 (1.22-cm diameter). Hoop spacing was 20 cm for the TB's.

In the long direction, slab was reinforced with one layer of 6-#3 bars (0.95-cm diameter) at 20 cm spacing. In the short direction, #4 bars (1.22-cm diameter) were spaced at 20 cm. Slab thicknesses were 10 cm, with a short direction width of 120 cm. Specimens were constructed on reinforced concrete foundation beams.

3.2 Test and loading history

Models were densely instrumented with load, displacement and strain electric transducers. Instrumentation was designed to assess the behavior of different wall components. The lateral shear was applied through a static-type hydraulic jack. To simulate the effect of gravity loading, a constant compressive stress equal to 5 kg/cm² was applied during the test. This stress level can be considered typical for low-cost housing buildings. Data acquisition was performed by means of an automatic equipment controlled by a personal computer. Data was stored for further reduction and analysis.

Specimens were tested applying alternate lateral loads. At each cycle, several load steps were applied; data channels were read at each load stage. Loading histories showed two phases (in fig. 2 a typical load history is shown). First, tests were load controlled with maximum shears equal to 5 ton, 10 ton, 18 ton and that which caused the first diagonal crack in the masonry panel. In the second stage, after initial diagonal cracking, displacement controlled cycles, with ever increasing drift ratios, were applied up to 0.012. For purposes of this study, drift ratio was defined as the measured lateral displacement at the slab, divided by the displacement transducer's height, measured from the foundation beam top surface. In fig. 2, the left ordinate axis represents the lateral load applied during the load-controlled phase, while the drift ratio peaks applied throughout the remainder of the test, can be read from the right ordinate axis. To facilitate data reduction and analysis, and comparison among specimen responses, the same load/deformation history was tried to be applied for the three structures. Nevertheless, when sudden wall cracking occurred, different drifts were recorded for the three specimens.

4 TEST RESULTS

4.1 Crack patterns

Final crack patterns for specimens W-W, WBW and WWW are shown in fig. 3. Damage in all specimens was governed by wall diagonal cracking. First diagonal cracks appeared near the panel corners, and propagated very fast towards the wall center with increasing deformation levels during the test. In general, walls showed one or two large diagonal cracks (X-shaped). Almost no damage was observed in the rest of the masonry panel. In contrast, specimen W-W showed a more uniform crack distribution than the other models. At large deformations, X-shaped cracks opened several centimeters. Most cracks propagated through brick layers; however, few of them extended along the brick-mortar interface, which is indicative that the brick tensile strength was lower than the bond resistance between mortar and bricks. After the X-shaped crack had formed, a kinetic mechanism developed so that a masonry block above the diagonal crack, slid relative to the lower masonry portion, thus causing brick crushing. No mortar crushing was observed. By the end of the test, few vertical cracks

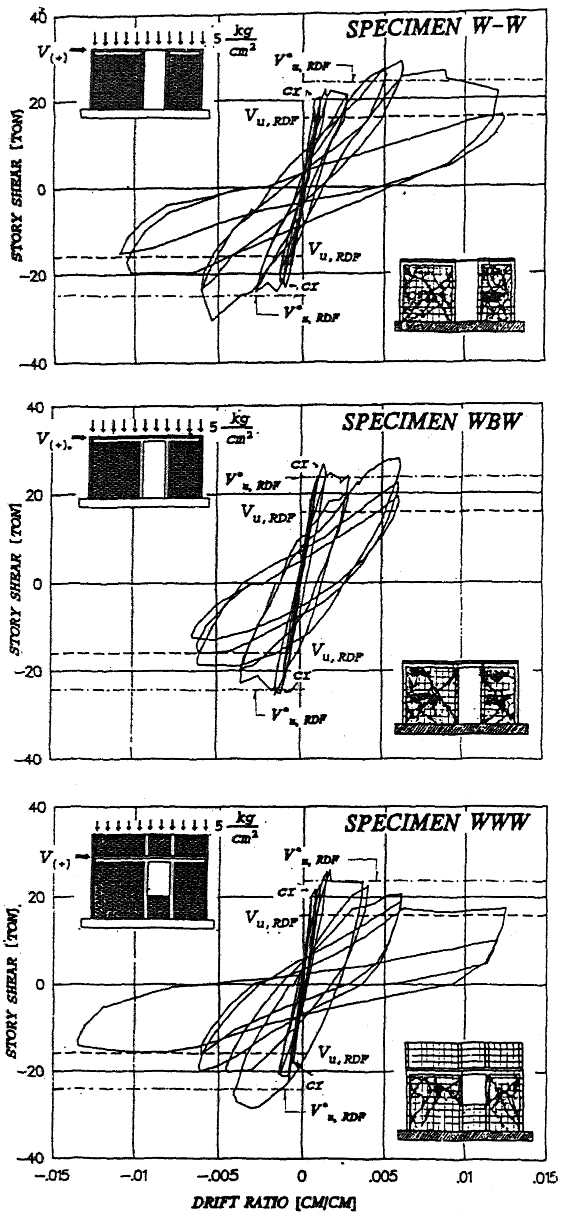

Figure 3. Response of specimens W-W, WBW and WWW.

were observed along the wall-TC interface for the three models. No cracks were observed in the floor systems (beams and slabs) for all specimens.

The effect of the type of opening between the walls on the crack patterns can be observed if models WBW and WWW are compared. Diagonal cracks extended through the walls from the corners of the opening to the diagonally opposite corner. No cracking was observed in the parapets of WWW. For specimens WBW and WWW, once the diagonal cracks fully extended across the walls, large forces concentrated in the corners and sheared off the TC's (drift ratios to 0.006). At large drift ratios (0.012), an S-shaped pattern due to the shearing forces was noted in the longitudinal reinforcement of the TC's that surrounded the opening. Cracks one-centimeter wide were observed at this stage. Damage was typical of plastic hinging. Shear cracks extended up to 20 cm outside the region that had hoops spaced at 7 cm. The exterior TC's only experienced flexural cracking uniformly distributed along the height. Since they were surrounded in three sides by the wall and two perpendicular wing walls, the presence of shear cracking could not be noted.

4.2 Hysteretic behavior

The story shear versus drift ratio curves for all models are shown in fig. 3. Graphs are shown to the same scale to allow comparison among the specimens. In the figures, the design shear forces, computed following the Mexican regulations for masonry structures (DDF 1989) are indicated as $V_{u,RDF}$ and $V^*_{u,RDF}$. Design shear forces were computed using the following expression (DDF 1989)

$$V_u = 0.5\, v^* A_T + 0.3\, P \qquad (1)$$

where
V_u = ultimate shear strength
v^* = shear stress
A_T = wall transverse area
P = applied vertical load.

To calculate both capacities, actual wall dimensions and P=25 *ton* were used. For $V_{u,RDF}$, the design shear stress (3.5 kg/cm^2) recommended by the code was used. For $V^*_{u,RDF}$, the shear stress was calculated from (DDF 1989)

$$v^* = \frac{\overline{v}}{1+2.5\, CV} \qquad (2)$$

where
$\overline{v}$ = average measured shear strength, 10 kg/cm^2
CV = coefficient of variation, 0.2.

First diagonal cracking is also indicated as "cr". Hysteresis curves were characteristic of confined masonry walls in which, after diagonal cracks occurred, strength and stiffness decay were observed, particularly at large drift ratios (0.012). Before cracking, specimens exhibited a linear elastic response. The onset of inelastic behavior was marked by wall cracking. Good energy dissipation characteristics were exhibited by all specimens up to drift ratios to 0.005, which are mainly attributed to the friction developed during brick relative movements, and

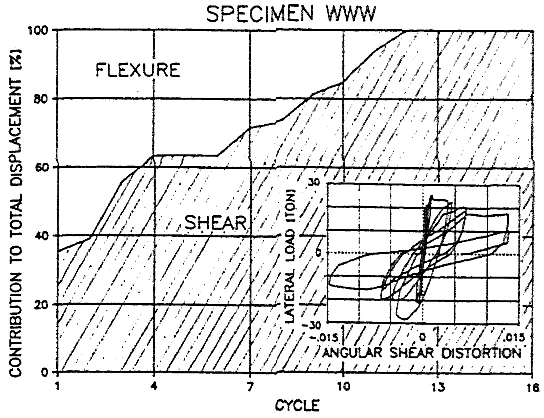

Figure 4. Total displacement contribution and angular strain for WWW.

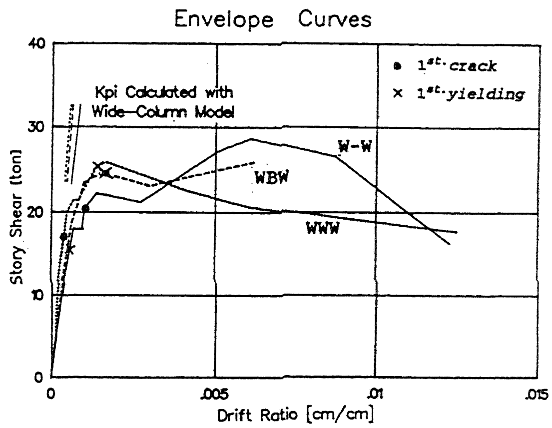

Figure 5. Response envelopes.

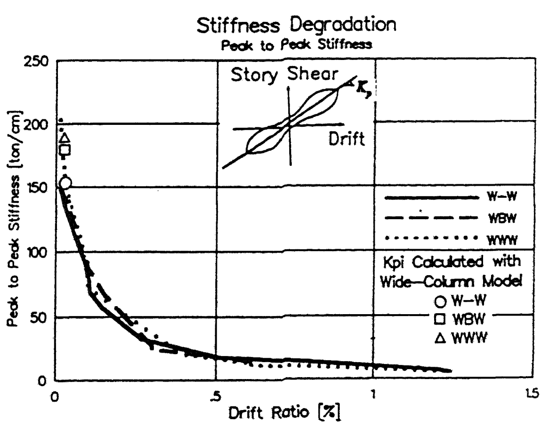

Figure 6. Stiffness decay.

to brick crushing.

After first diagonal cracking, all structures resisted higher shear forces. This is due to a reserve of capacity until the X-shaped diagonal cracks extended totally across the wall panel. Tie-columns and TB's improved the energy dissipation capabilities of the walls and reduced the strength and stiffness decay because of the confinement provided, but did not increase the capacity of the specimens. Should TC's and TB's not be provided, it is likely that severe degradation would have been observed. From fig. 3, it can be observed that both the diagonal cracking shear force and the maximum measured lateral load were greater than the design shear forces computed. Moreover, for almost all cycles, the measured shear force was higher than the design capacity, even when large deformations (0.006) were applied. This is indicative of the good deformation capacity of confined masonry walls, thus maintaining the strength at large deformation levels higher than the theoretical capacities. In fig. 4, the participation of shear deformations to the total deformation for WWW can be observed. The story shear versus angular distortion is also presented in fig. 4. Similar curves were obtained for WBW. It is clear that shear deformations governed the response. Note the resemblance of the angular distortion curves with the hysteresis loops (see fig. 3).

Response envelopes for all specimens are shown in fig. 5. First cracking and first yielding are indicated in the graph. First yielding was obtained from strain readings from the longitudinal reinforcement. First yielding always occurred at the base of TC's. Calculated initial stiffnesses with a standard wide-column model, using gross sectional properties and measured material properties, are also presented. The calculated initial stiffnesses are in good agreement with those measured. From fig. 5, it is clear that the initial stiffness varied with the type of flexural coupling. For WWW, the secant stiffness calculated to the first peak at 18 ton, is 53% higher than that of WBW, and 68% higher than that of W-W. It is important to note that the drift ductility factors (maximum drift divided by drift at first yielding) for WBW and WWW are 3 and 8.5, respectively, which are comparable to results reported elsewhere (Meli 1975).

4.3 Stiffness degradation

To assess the stiffness decay, peak-to-peak stiffness K_p was calculated for each cycle in all models. Drift ratio versus K_p curves are shown in fig. 6. Curves are very similar regardless of the degree of coupling. Stiffness degradation is observed at low drift ratios (before diagonal cracking) probably due to flexural cracking in TC's and adjustment of bricks. Stiffness shows further degradation when cycles at the same deformation level are applied. Decay was higher in cycles up to 0.005 drift ratio; at larger drift ratios, when the concrete TC's and TB's provide for the stiffness, K_p remains nearly constant. Stiffness decay is credited to cracking and crushing in walls and TC's.

5 CONCLUSIONS

Based on previous research programs and on the results reported herein, the following conclusions can be obtained.

The strength of masonry units is dependent on the strength of brick units, and is less dependent on mortar characteristics.

Vertical load increases the shear capacity and stiffness. However, large vertical forces reduce the available ductility of the structure.

Tie-columns and tie-beams provide adequate confinement to the masonry walls thus improving their energy dissipation characteristics and deformation capacity.

The form of the opening clearly affected the final crack pattern. However, the mode of failure for all specimens was governed by shear deformations in masonry panels and was not affected by the degree of coupling.

First diagonal cracking were caused by similar lateral forces, regardless of the type of coupling.

Maximum measured strengths were 75% higher, on the average, than the calculated capacities using code recommended masonry strengths.

Stable hysteretic loops were observed up to 0.005 drift ratio; this limit deformation is small compared with well-detailed r/c frame or wall structures. At larger deformations than 0.005 severe degradation was noted. Therefore, large reductions on the elastic spectral ordinates for this type of structures cannot be justified.

Although the type of opening affected the initial stiffness of the specimens, the stiffness decay was similar and follow a parabolic curve. A wide-column model can be used to predict the initial stiffness of a structure.

ACKNOWLEDGEMENTS

The financial support of the Instituto Nacional del Fondo para la Vivienda de los Trabajadores (INFONAVIT-Mexico) and of the Japan International Cooperation Agency is acknowledged. The help of the staff of the Large-Scale Structural Testing Laboratory at the National Center for Disaster Prevention is thankfully acknowledged.

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