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CALCULATED RESPONSE OF CONFINED MASONRY STRUCTURES

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ABSTRACT

Aimed at studying the response of confined masonry wall structures, nonlinear dynamic analyses of typical Mexican low-cost housing buildings were performed. Variables were the frequency content of earthquake records and the number of stories (one and three). The effect of using horizontal steel on the structure behavior was assessed.

Calculated force-displacement relations followed the hysteresis rules developed for confined masonry walls (with and without horizontal reinforcement) from tests of full-scale specimens. Hysteretic loops were limited by a tri-linear envelope curve. Calculated stiffnesses for the loading and unloading branches followed a decay law which depended on the deformation history. Results indicated that for high-frequency records, horizontal wire steel in walls provides a feasible solution for improving the dynamic characteristics of structures allowing them to withstand severe ground shaking without collapse.

KEYWORDS

Response; nonlinear analysis; analytical model; hysteresis rule; confined masonry; walls; horizontal steel; steel wires; Green's function.

INTRODUCTION

Masonry is the construction material most widely employed for dwellings and apartment buildings up to 5-stories high in Mexico. Due to the enforcement of new codes with higher seismic-induced force levels, and due to the lack of design standards for all cities of the country, a widespread heterogeneity in the design criteria for confined masonry (CM) structures has been found in Mexico (Meli *et al.*, 1994). For instance, similar designs are produced for multi-family buildings in regions with distinctly different seismic risk. Structures in highest seismic risk areas require large wall densities to resist the design forces. Options for increasing the strength and deformability characteristics have been experimentally evaluated (i.e. walls reinforced with horizontal high-strength cold-drawn deformed wires, and walls jacketed with steel welded wire meshes shotcreted with cement mortar).

Aimed at developing design guidelines for low-cost housing buildings in high-seismic areas, an analytical research was undertaken. This study is part of a comprehensive program underway at the National Center for

Disaster Prevention in Mexico City (Alcocer and Meli, 1995). Nonlinear dynamic analysis were performed on typical Mexican apartment buildings with one and three stories. The frequency content of the earthquake motions and the amount of wall horizontal reinforcement were varied. This paper reports on the general results obtained until date; further information may be found elsewhere (Flores, 1995).

ANALYTICAL MODEL

Hysteresis rules used for the nonlinear dynamic analysis were developed and calibrated from tests carried out on full-scale confined masonry specimens at CENAPRED. Results of these experiments can be found elsewhere (Ishibashi *et al.*, 1992; Alcocer and Meli, 1995; Aguilar *et al.*, 1996). In all structures, walls were built with hand-made clay bricks and cement mortar with a cement:sand ratio equal to 1:3 or 1:4 (by volume). Typically, brick dimensions were 240x120x60 mm. Average masonry prism strength was between 5,4 and 3,5 MPa and diagonal compression strength was between 0,52 and 0,25 MPa. Mortar cube strength was between 10,8 and 6,7 MPa. Grade 42 steel (f_y= 412 MPa) was used for tie-column, bond-beam and slab longitudinal reinforcement. Hoops were made of #2 mild steel (f_y= 216 MPa). Horizontal high-strength cold-drawn deformed wires were of 4,0 and 6,0 mm in diameter and had a nominal yield stress equal to 589 MPa. Tie-column and bond-beam cross section were 120x150 mm and 120x250 mm, respectively.

Typical Behavior Observed

A typical load-deformation envelope curve for CM structures subjected to alternated cyclic lateral loads is shown in Fig. 1. Drawings in the graph are shown for clarity for lateral loads applied in the positive direction; similar crack patterns and distress are exhibited when loaded in the opposite direction. In walls with no horizontal reinforcement, initial behavior is linear-elastic until first inclined masonry cracking occurs (point A). With further cycling at higher levels of deformation, cracking concentrates near the diagonals, thus dividing the wall in triangular blocks limited by the main cracks. At this stage (point B), wall stiffness has considerably reduced; strength is provided by friction and brick interlock in the masonry panel and by the shear resistance of tie-column (TC) ends. Shear deterioration of wall strength (point C) is credited to brick crushing and spalling, and to shearing of TC ends (concrete crushing and kinking of longitudinal reinforcement). Indeed, the slope of the descending branch (B-C) is strongly influenced by the detailing of TC ends. If TC hoop spacing is reduced at the ends, dowel resistance of longitudinal bars is augmented, thus leading to a more stable strength and stiffness behavior.

For walls with horizontal high-strength cold-drawn deformed wires in the panel, initial behavior is similar to that described before. The amount of horizontal steel does not affect the initial stiffness nor the cracking load (Aguilar *et al.*, 1996). At the onset of masonry cracking, participation of horizontal steel to the wall lateral load carrying capacity starts. Tests have shown a remarkably improved behavior of walls with horizontal reinforcement in terms of strength and deformation capacity (Aguilar *et al.*, 1996). At wall strength, a diagonal compression strut forms within the masonry. This strut is balanced, in the horizontal direction, by forces resisted by the wires. The participation of wires in the load carrying mechanism leads to a more uniform inclined cracking than in specimens with no horizontal wires. After reaching the structure strength, the descending branch in the envelope curve may show a sharp reduction (severe slope) if wires fracture. Typically, if reinforcement steel ratios are low (around 0,007), specimen will fail in shear following the fracture of wires. If steel ratios are close to 0,020, brick crushing in the panel is likely to occur prior to yielding of horizontal steel.

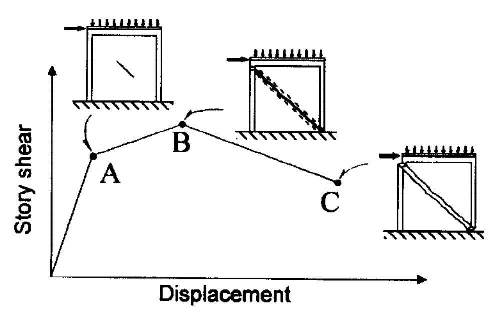
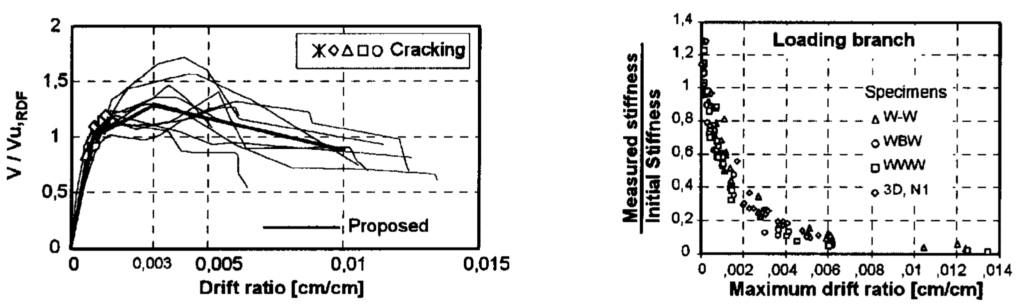


Fig. 1 Typical envelope curve from tests of confined masonry walls

Normalized Envelopes and Stiffness Decay Curves

Response envelopes of tests were normalized by the design shear strength, $Vu_{,RDF}$, calculated from current masonry code requirements in Mexico City (DDF, 1995). In Fig. 2 these curves are shown for positive and negative cycles for specimens with no horizontal reinforcement. A similar plot was obtained for walls reinforced horizontally (Flores, 1995). Drift ratio was calculated by dividing the specimen horizontal displacement by the structure's height. First inclined cracking is indicated with symbols. It is apparent the small variation of cracking loads and the good prediction obtained with the code design equation. After cracking, a large scatter of results is observed; this phenomenon is typical of CM structures, for which material properties also show large variations. Typical coefficients of variation of compression strength and diagonal tension resistance are about 20 to 30 percent.

Measured stiffness of the loading and unloading branches in the hysteresis loops were normalized by the measured initial stiffness. Measured stiffness was obtained as the slope of the secant drawn from the point with zero load of the previous half-cycle to the peak in the next half-cycle. Initial stiffness was the slope of the secant line drawn at a point where the slope of the envelope curve changes significantly. Normalized stiffness against the maximum drift ratio reached in the previous half-cycle is shown in Fig. 2 for the loading branch in specimens with no horizontal reinforcement. Similar plots for the descending branch and for horizontally reinforced walls are available elsewhere (Flores, 1995).



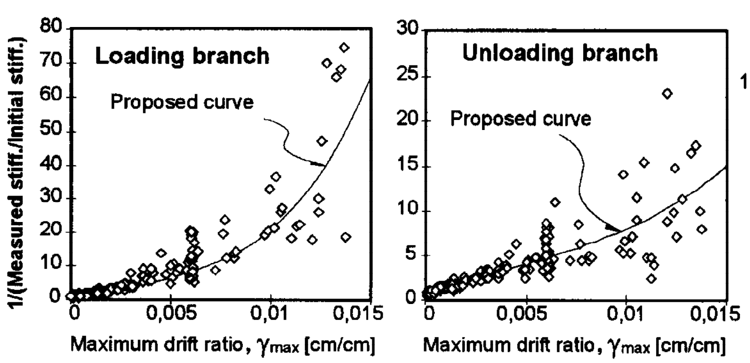
a) Normalized envelopes

b) Stiffness decay curves

Fig. 2 Normalized envelopes and stiffness decay curves

Model Proposed

Based on the normalized envelopes (Fig. 2a) and on the typical behavior characteristics reported, a tri-linear envelope was proposed (see dark line in Fig. 2a). For the hysteresis rules for walls with and without horizontal reinforcement, stiffnesses of the loading and unloading branches follow a best-fit curve of the data (Fig. 3).



Proposed curve:

$1/(\text{Measured stiff./Initial stiff.}) = a\,(\gamma_{max})^4 + b\,\gamma_{max} + 1$

Parameter	Without horizontal reinforcement		With horizontal reinforcement	
	Loading	Unloading	Loading	Unloading
a	1×10^9	1×10^8	1×10^8	1×10^7
b	1000	600	1000	300

Fig. 3 Normalized stiffness degradation

According to the model proposed, loading stiffness is constant until reaching the envelope. If higher deformation is imposed, the loading branch will follow the envelope. To compute the stiffnesses for the loading and unloading branches for the next half-cycle, the maximum drift ratio reached must be recorded. This implies that a stiffness decay in the loops will not be calculated if drift ratios are smaller than a value reached before.

CALIBRATION OF THE ANALYTICAL MODEL

The shape of hysteresis loops and envelopes, stiffness decay and the cumulative energy dissipated were the criteria used to qualify the adequacy of the analytical model. Calculated load histories were compared to those obtained in the laboratory. Analytical and experimental stiffnesses were also compared. "Cycle stiffness", which is the slope of the secant which joins peaks of a particular cycle, was used. Energy dissipation was calculated as the area within a loop.

Drift ratio histories and cracking loads were used to calculate the response of CENAPRED specimens. A sample of measured and calculated hysteresis loops and load histories is shown in Fig. 4. In general, agreement was very satisfactory. Calculated loops are thinner than those measured because the loading and unloading branches are idealized by straight lines and not by curves as experimental results suggest. The latter leads to a lower amount of calculated energy dissipated when compared to the value obtained in laboratory.

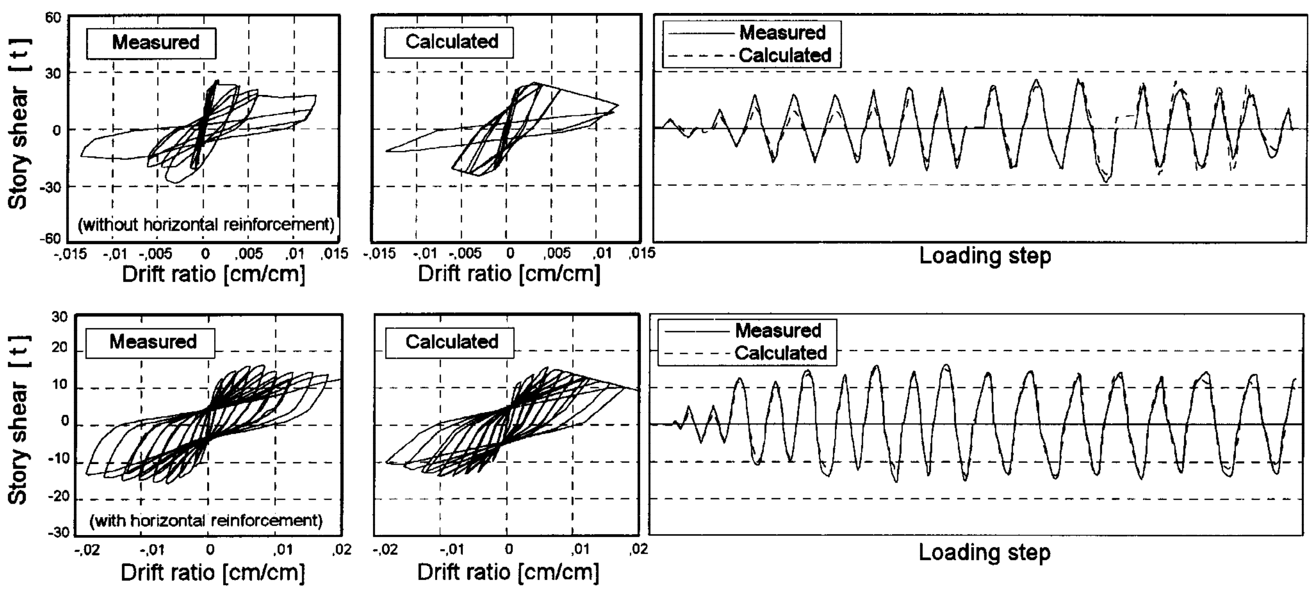


Fig. 4 Comparison between measured and calculated response (1 t = 9,81 kN)

ANALYSIS OF STRUCTURES

Confined masonry wall structures built with hand-made clay bricks were analyzed. Variables were the number of stories, earthquake records, and the amount of horizontal reinforcement in the wall (zero, minimum and maximum values). The minimum value is that specified in the current Mexican Building Code (DDF, 1995). The maximum steel ratio was computed with a formula developed to avoid masonry crushing prior to yielding of horizontal reinforcement (Aguilar *et al.*, 1996). Structures were considered fixed at the base. Soil-structure interaction was not accounted for.

Number of Stories

One- and three-story structures were analyzed (Fig. 5). Structure dimensions and details were obtained from drawings of typical low-cost housing buildings in regions of high seismic risk. Buildings were constructed by the largest Mexican housing agency of the Federal Government (INFONAVIT). Walls had no horizontal reinforcement. In this graph, calculated wall densities in the short direction (critical direction) are compared to those required by different Mexican codes. Wall density was calculated as the sum of transverse wall area divided by the floor plan area. Wall density is related to building lateral strength. Both types of structures fulfilled the design force requirements of the current Mexico City Building Code; however, for the three-story building wall density is scarce for the Guerrero Code. As a reference, Acapulco is located in the state of Guerrero, in the zone of highest seismic risk in Mexico (Fig. 6). Analyses were performed in the short direction.

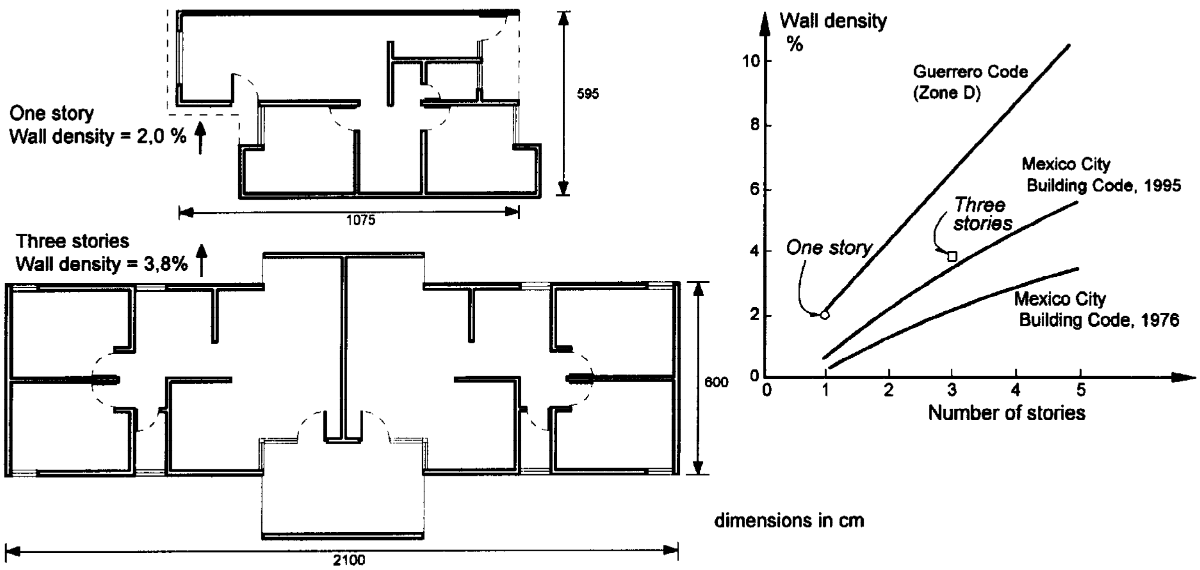


Fig. 5 Floor plan of the structures analyzed and their wall density compared to code requirements

Earthquake Records

For analyses, records with different amplitude, duration and frequency content were used (Fig. 7). For long-period records, the famous SCT accelerogram of September 19, 1985 was selected. The accelerogram recorded at the Kobe Meteorological Agency (January 17, 1995) was used as a high-frequency input. Also, an artificial accelerogram was obtained from the record of April 25, 1991 in Acapulco. The calculation procedure used the 1991 record as a Green function to obtain a postulated M8,2 earthquake record (Flores, 1995). Response spectra for the records are also shown in the graph. A five-percent value of the critical damping ratio was used for computation. This value of damping ratio is commonly assumed for masonry structures. Damping ratios between 2 and 5% have been obtained through ambient vibration tests in CM structures (Ruiz, 1995).

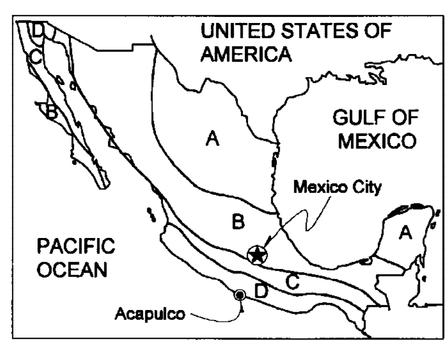


Fig. 6 Seismic regionalization in Mexico

Wall Panel Reinforcement

Walls with and without horizontal reinforcement made of steel high-strength cold-drawn deformed wires were considered (f_y=589 MPa). For horizontally reinforced walls, two steel amounts were used: the minimum value recommended by the code (DDF, 1995) which was close to 0,007, and the maximum value calculated with a formula intended to avoid web crushing prior to yelding of horizontal reinforcement (Aguilar *et al.*, 1996). In the mathematical model, all walls in a particular story were considered either horizontally reinforced or unreinforced. For the 3-story building, walls were reinforced at the ground story.

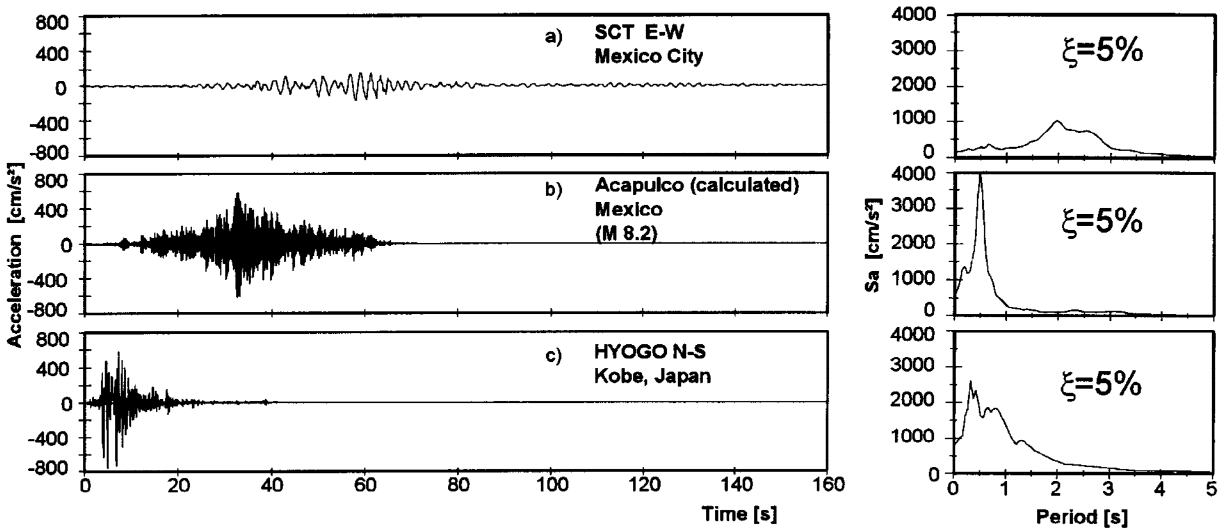


Fig. 7 Earthquake records used in the analysis

RESULTS OF ANALYSES

Cases studied are identified by a combination of numbers and letters: NXY: N refers to the number of stories (1 or 3); X to the record used (S=SCT, A=Acapulco, K=Kobe); and Y to the amount of horizontal reinforcement (0=none, C_1=minimum amount, C_2=maximum amount).

One-Story Structure

Total weight of the structure was 518 kN (52,8 t), ultimate lateral shear strength was 176,6 kN (18,0 t) (nominal strength affected by a strength factor less than one). Design base shear was 73,6 kN (7,5 t) and 245,3 kN (25,0 t) for the lake bed zone in Mexico City and for the southern coast along the Pacific Ocean, respectively. The design base shear coefficient was 0,13 and 0,43 for the lake bed zone in Mexico City and for Zone D (see Fig. 6), respectively. It is clear that this structure would not satisfy the force-level requirements of zones of high seismic risk. Results of the analyses are shown in Table 1. In parenthesis, values for the linear-elastic response are shown. The structure survived unscathed the SCT record. This agrees with the response of CM structures observed in Mexico City during the 1985 earthquakes. When subjected to the artificial accelerogram in Acapulco (M8.2), the structure would have collapsed. The analysis was repeated but considering walls with horizontal reinforcement. For the minimum steel ratio (0,007), higher force demands were resisted but finally the structure would have collapsed. When the maximum amount of horizontal steel was used (0,018), calculations indicate that the structure would have survived both the Kobe and Acapulco records. Hysteretic response and roof acceleration histories are shown in Fig. 8 for cases 1S0 and $1AC_2$. As expected, the response of 1S0 was small and remained in the linear-elastic range. Under the artificial accelerogram in Acapulco, loops were ample and with good energy dissipation capacity.

Table 1 One-story structure response

Case		Maximum acceleration [cm/s²]	Maximum drift ratio [%]	Base shear Vb_{max} [kN]
1S0	✓	179 (179)	0,02 (0,02)	94,4 (94,4)
1A0	⇩	- (1281)	- (0,12)	- (675,6)
$1AC_2$	✓	756 (1281)	0,17 (0,12)	399,1 (675,6)
$1KC_2$	✓	756 (907)	0,27 (0,09)	398,9 (478,3)

✓ = survived;
⇩ = collapsed.

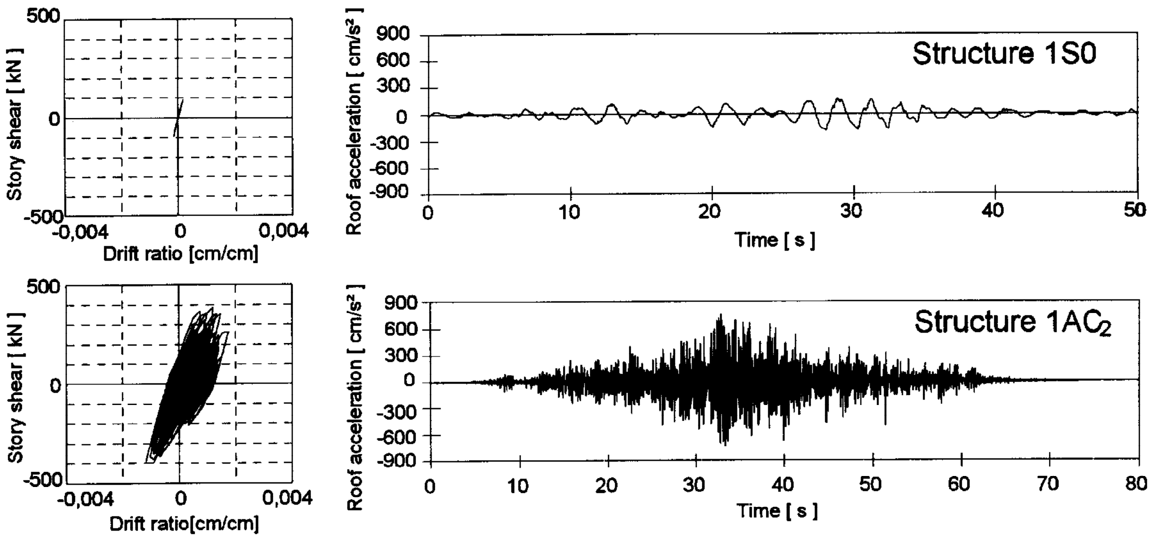


Fig. 8 Response of the one-story structure

Three-Story Structure

The building analyzed was designed according to the design spectrum in the lake bed zone in Mexico City (DDF, 1995). Design base shear coefficient was 0,19. Each story weight was 1332 kN (135,8 t), so that the total weight was 3997 kN (407,4 t). Ultimate base shear strength was 911,3 kN (92,9 t); design earthquake base shear was 834,8 kN (85,1 t). For Zone D, the design base shear was 1890,4 kN (192,7 t). Results of analyses are tabulated in Table 2. The base shear versus ground story drift ratio curves for cases 3A0 and $3AC_1$ are shown in Fig. 9. The structure would have survived the SCT record; again, this agrees well with the observed behavior. Under the Acapulco's artificial accelerogram, the structure would have survived only if the minimum amount of horizontal reinforcement were provided.

Table 2 Three-stories structure response

Case		Roof acceleration [cm/s²]	First story drift ratio [%]	Base shear Vb_{max} [kN]	Vb/W_T
3S0	✓	269,5 (263,7)	0,08 (0,05)	938 (922)	0,23 (0,23)
3A0	⇩	- (2830,2)	- (1,02)	- (7339)	- (1,84)
$3AC_1$	✓	1133,1 (2830,2)	0,67 (1,02)	2388 (7339)	0,60 (1,84)

Lateral load distribution over the structure's height varied with the type of earthquake record. For the soft-soil SCT record, a nearly uniform load distribution was obtained; this finding contrasts with the code assumption of a triangular distribution of forces. For low-period accelerograms, force distribution could be approximated by a triangle.

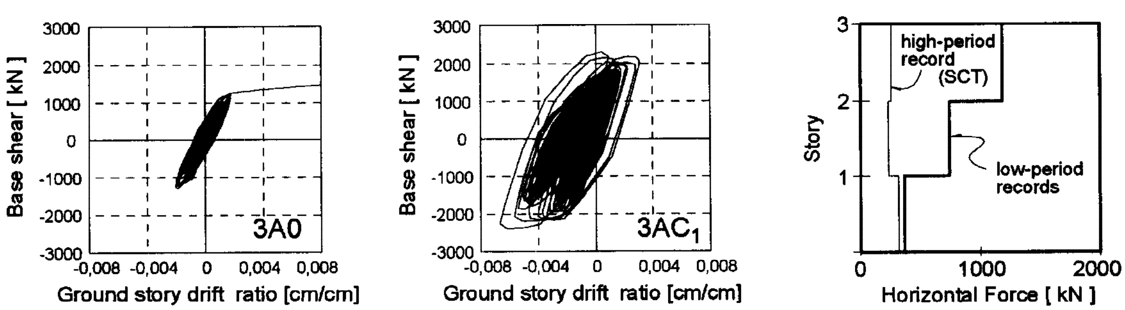
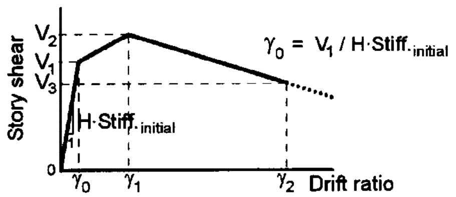


Fig. 9 Response of the three-stories structure

CONCLUSIONS

1. A mathematical model was developed to represent the nonlinear behavior of confined masonry structures built with hand-made clay bricks.
2. The envelope curve of the model is a tri-linear force-deformation curve, which is calculated from material properties and wall geometry (Fig.10).



Without horizontal reinforcement	With horizontal reinforcement
$V_2 = V_1 + \eta V_d$	$V_2 = V_1 + \lambda V_s$
or $V_2 \approx 1{,}25\, V_1$	or $V_2 \approx 1{,}5\, V_1$
$V_3 = 1{,}12\, V_1$	$V_3 = 1{,}32\, V_1$
$\gamma_1 = 0{,}003$	$\gamma_1 = 0{,}006$
$\gamma_2 = 0{,}005$	$\gamma_2 = 0{,}01$

$$V_1 = 0{,}5v^* A_T + 0.3P \leq 1{,}5 v^* A_T$$

$$V_d = 1{,}26\, d_b^2 \sqrt{f_c' \, f_y} \; ; \quad \eta = 0{,}3$$

$$V_s = \rho_h \, f_{y,h} \, A_T \; ; \quad \lambda = 0{,}35$$

$$\text{Stiff.}_{\cdot initial} = \left(\frac{h^3}{\beta EI} + \frac{h}{GA} \right)^{-1} ; \quad 3 \leq \beta \leq 12$$

β depends on the boundary conditions of the walls
v^* is the design diagonal compression strength
A_T is the wall transverse area
P is the applied vertical load

Fig. 10 Parameters for the envelope curve

3. The loading and unloading stiffness are calculated following a rule which depends on the maximum drift ratio reached in previous excursions (Fig. 3).
4. Calibration of the mathematical model was very successful when results obtained through experiments were compared with those calculated.
5. Results of analyses of one- and three-story structures subjected to the SCT record were consistent with the behavior observed in the aftermath of the 1985 earthquakes.
6. Horizontal reinforcement in walls at lower stories increased the structure strength and deformation capacity, particulary under high-frequency motions.
7. Highest damage in the three-story structure was concentred at the ground level; this phenomenon coincides with observations made during earthquake reconnaissance.
8. Force distribution over the building height when subjected to the SCT record (long-period accelerogram) departed from the commonly-used code prescription of a triangular distribution.

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REFERENCES

Aguilar, G. *et al.* (1996). Influence of the horizontal reinforcement on the behavior of confined masonry walls. *Proceedings of 11WCEE,* Mexico.

Alcocer, S.M. and R. Meli (1995). Test program on the seismic behavior of confined masonry structures. *The Masonry Soc. J.*, 13, 68-76.

Alcocer, S.M. *et al.* (1996). Retrofitting of confined masonry walls with welded wire mesh. *Proceedings of 11WCEE,* Mexico.

Departamento del Distrito Federal, DDF (1995). Reglamento de Construcciones para el Distrito Federal (Mexico City Building Code), Mexico.

Flores, L.E. (1995). Estudio analítico de estructuras de mampostería confinada. *B.Sc. Thesis*, National Autonomous University of Mexico (UNAM), 106.

Ishibashi, K. *et al.* (1992). Experimental study on earthquake resistant design of confined masonry structures. *Proceedings of 10WCEE,* Spain, 3469-3474.

Jordán, R.M. and M. Kreger (1991). LARZ user's manual. University of Texas at Austin, 40.

Meli, R. *et al.* (1994). Características estructurales de la vivienda de interés social en México. *Cuadernos de Investigación.* Centro Nacional de Prevención de Desastres, 17, *25-52.*

Ruiz, J. (1995). Reparación y refuerzo de una estructura tridimensional de mampostería confinada de dos niveles a escala natural. *M. Sc. Thesis.* National Autonomous University of Mexico (UNAM), 251.