

## EXPERIMENTAL RESEARCH AND SEISMIC DESIGN PROPOSAL FOR CONFINED ADOBE MASONRY

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### ABSTRACT

The traditional constructions of unreinforced adobe are highly vulnerable to earthquakes, mostly affecting people of low income. The experimental research done in order to mitigate this problem has tried to reinforce adobe constructions with internal cane meshes, external welded-wire meshes or similar. In recent years, laboratory tests have been developed to study adobe confined with reinforced concrete elements under seismic loads, with good results. The tests include cyclic lateral loads on full scale walls and shaking table tests on one and two-story models.

Based on the results of these experiments, a seismic design proposal for confined adobe houses up to two stories was developed. The theory of the proposal uses the criteria of ultimate strength and performance-based design used in confined masonry of fired clay bricks, acknowledging the poor quality and low resistance of the traditional adobe masonry. The application of this theory leads to low-strength concrete columns and beams and minimum reinforcement. In this way, the design proposal can be useful for people of low income.

**KEYWORDS:** codes, adobe, confined adobe, seismic design, experimental tests

### INTRODUCTION

In Peru, a few one or two-story houses can be found in which the adobe walls are confined by reinforced concrete elements (Figure 1). However, such constructions do not have a rational structural design, and usually have too much steel reinforcement, becoming an unnecessarily expensive construction. On the other hand, experimental tests carried out in the Structures Laboratory at the Catholic University of Peru indicate that it is possible to make a rational confined adobe design under seismic forces.



**Figure 1: Examples of confined adobe walls in Peru.**

Also, there are evidences of good behavior exhibited by these structures under real earthquakes, like the Mexican Omotepec 1995 earthquake, as shown in Figure 2. The purpose of this paper is to present a seismic design proposal for confined adobe structures. The experimental tests in which this proposal is based are previously summarized.

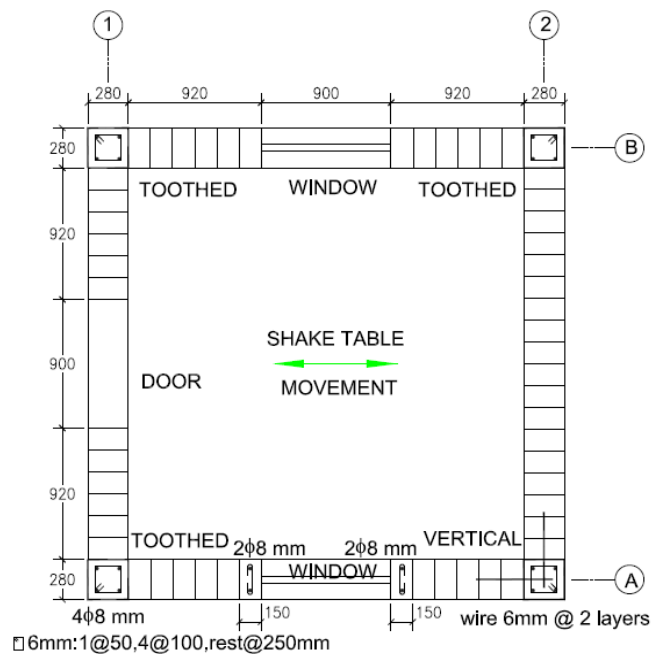


**Figure 2: Unreinforced collapsed house (left) and Confined adobe house (right) after the 1995 Omotepec earthquake (EERI, 1995).**

### PREVIOUS EXPERIMENTAL RESEARCH ON CONFINED ADOBE

A set of experimental projects on confined adobe structures are briefly summarized. All were subjected to seismic loads at the Structures Laboratory at the Catholic University of Peru.

The first experimental project consisted on seismic simulation tests on a shaking table with one horizontal degree-of-freedom to a one story full scale model (Matos et.al., 1996). The model had four adobe walls with one door and two window openings, as shown in Figure 3.



**Figure 3: Confined Adobe 3D model**

In each corner, a confining square concrete column was added, with 280mm side, reinforced with 4-8mm bars. Three corner connections between adobe walls and concrete columns were toothed, while one had small wires anchored in the walls in two perpendicular directions. Surrounding the window opening in the A-axis, two small RC columns were included, with 2-8mm bars. This model was subjected to the acceleration history of May 31, 1970 earthquake recorded in Lima, Peru, in six steps up to a peak base acceleration of 1.6g.

The confined adobe model behaved elastically without any crack (Figure 4a). After that, the adobe walls on axes A and B were removed in order to observe if the concrete columns were responsible of the high strength observed, and the specimen was tested again (Figure 4b). In this second stage, for step 2 (0.6g of peak acceleration) plastic hinges were developed in the column borders and the specimen become unstable. In Figure 4c a comparison between the confined adobe masonry walls and the RC frame is given, and it was concluded that the adobe wall was greatly responsible for the strength and rigidity.



(a) (b)  
(c)

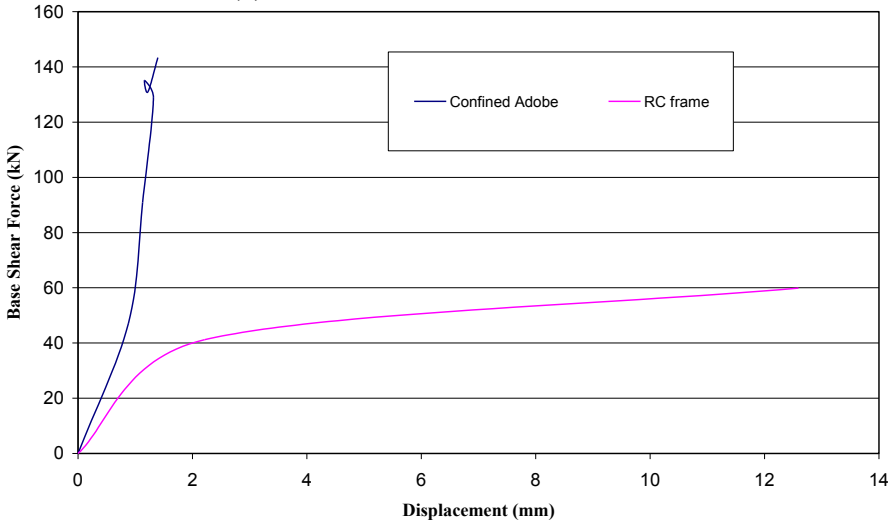
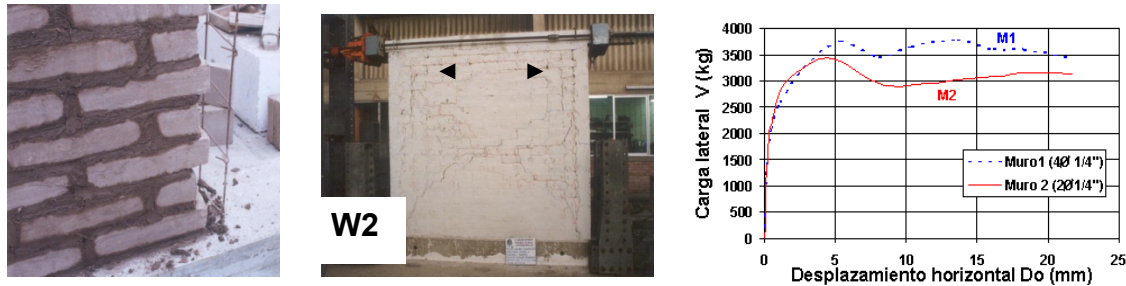


Figure 4: Confined Adobe 3D model and Concrete Frame, 1996

In the second experimental project, two confined adobe walls were subjected to cyclic lateral loads (San Bartolomé, 2003). The confinement RC elements were designed to resist the diagonal cracking load, according to the procedure given hereafter in the design proposal. The design required only 2-6mm bars in each column, but in order to study the effects of the vertical reinforcement, 4-6mm bars were used in each column in wall W1, while only 2-6mm bars were used in wall W2. The hysteretic loops for both walls are displayed in figure 5, in which almost no difference can be appreciated. This is attributed to the dominant shear deformation that prevailed in both walls under the cyclic loads.



**Figure 5. Lateral Load Test and Envelope V-D (Angel San Bartolomé, 2003).**

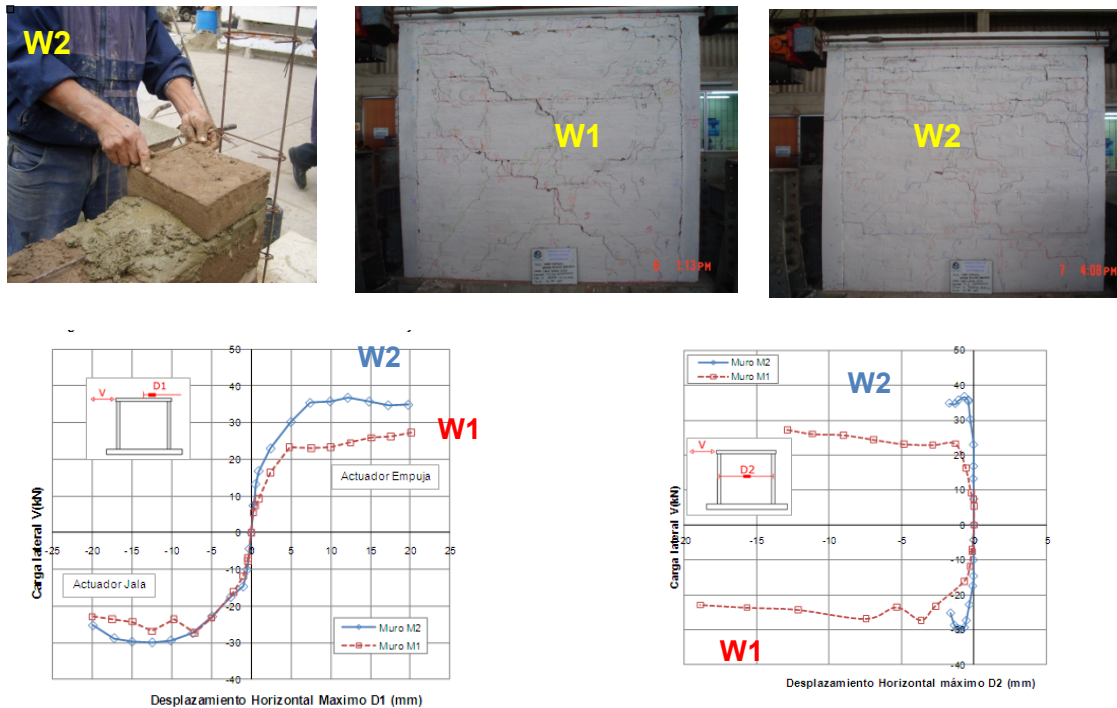
For the third experimental project, a two-story model was constructed using a  $\frac{3}{4}$  scale, and was subjected to a shaking table test (Figure 6, San Bartolomé et. al. 2009). The reinforcing bars in the confining columns were obtained using the specifications shown hereafter in the proposal, except that the horizontal reinforcement within the layers was omitted in the second floor. Also, the spacing between columns was slightly larger than the proposed design. For the columns, 4-6mm ( $\frac{1}{4}$ " ) bars were required and for the collar beams, 2-6mm ( $\frac{1}{4}$ " ) bars were required. The model was subjected to the acceleration history of May 31, 1970 earthquake recorded in Lima, Peru, in six steps up to a peak base acceleration of 1.0g. In step 4 (0.8g), some adobe units of the second story flexural wall fell down, producing a "V" shape failure. In the last step, the whole wall collapsed due to the out-of-plane forces and the lack of horizontal reinforcement, while the shear walls only had a few small cracks.



## Figure 6. Shaking table test and horizontal reinforcement proposed

In the fourth and last experimental research project completed, the effects of horizontal reinforcement in two confined adobe walls subjected to cyclic lateral loads were studied (Torres, 2011).

The two walls had RC confinements, having 2 6mm bars (1/4") reinforcing the columns and collar beams. Wall W1 had no horizontal reinforcement. Wall W2 had one horizontal 6mm bar every 5 layers, anchored in the columns and placed in a cement-sand 1:8 mortar, while the rest of the layers had mud mortar (Figure 7, up left). The hysteretic curves (Figure 7, bottom left) show that in the negative branch of the envelopes the influence of the reinforcement is negligible over the shear resistance, but in the other positive branch it increased. In terms of the overall behavior, the horizontal reinforcement controlled the cracks, making them very thin in wall W2 and keeping the wall integrity. For a horizontal displacement  $D=20\text{mm}$ , the vertical cracks in the column-wall intersection had a maximum width of 3.5mm in W1 and 0.5 mm in W2.



**Figure 7. Cyclic lateral load in Walls W1 (unreinforced) and W2 (with horizontal reinforcement); hysteretic envelope V-D; and relative displacement between columns**

## SEISMIC DESIGN PROPOSAL FOR CONFINED ADOBE

The following seismic design proposal is based on the experimental tests performed, and on the observations that confined adobe walls have a similar seismic behaviour to clay brick masonry with a low strength: a predominant shear deformation under in-plane loads, and a weakness in

the wall-confinement connections for out-of-plane loads. The equations are similar to those used in the Peruvian Masonry Code (San Bartolome and Quiun, 2007; SENCICO Norma E.070 “Albañilería”, 2006).

### **GENERAL CONSIDERATIONS FOR CONFINED ADOBE**

The confined adobe buildings may be up to two stories or 6m high. In this proposal, an adobe wall is considered to be confined by RC elements whenever it meets the following specifications:

- a) The masonry adobe wall must be completely surrounded by RC elements cast after the masonry was built;
- b) For the first story, if the foundation is made of plain concrete, it can be considered as a horizontal confinement; the foundation height should be enough to let a proper anchorage of the vertical column bars plus a 75 mm cover.
- c) The distance between confinement columns shall not exceed twice the distance between the horizontal confinements (collar beams) nor 3.5m.
- d) The openings for doors and windows wider than 1m, must be surrounded by RC columns. The lintel beams in such openings must be of reinforced concrete with a cross section similar to that of the collar beams over the walls.
- e) The minimum width of the wall shall be 250 mm. The confinement width (“t”) shall be equal or greater than the wall width. The minimum area of the confinement cross sections shall be  $100t$  (in  $\text{mm}^2$ ).
- f) The wall-column interfaces shall be vertical.
- g) All the walls must have continuous horizontal reinforcement, consisting on at least 1 deformed 6mm bar every 5 layers, placed in the wall axis over a mortar bed of 1:8 cement-coarse sand (in volume), during the adobe masonry construction. The bars should enter 120 mm into the column core with a  $90^\circ$  bend of 100mm length. In the case of the column dimension being larger than 150 mm, the anchorage of these bars can be mechanical, bent over a stirrup or hook.
- h) The confinement concrete shall have a compressive strength  $f'c = 10$  MPa or more. The concrete should be compacted using a 12mm bar or a vibrator. If voids are detected, the loose particles will be discarded and the affected area will be watered and manually filled with 1:4 cement-sand mortar.
- i) The reinforcement in the confining column should be ductile steel deformed bars, with diameters equal or larger than 6mm, having a yield stress of 420 MPa. The bars cover should be 20mm if an overall cement cover is applied, or 30 mm otherwise.
- j) The minimum longitudinal reinforcement in the confinements shall be two 6mm bars, while the minimum stirrups should be 6mm hooks, 1 at 50mm, 4 at 100mm, and the rest at 250 mm. The hooks shall be bent  $180^\circ$  over the longitudinal bars.

### **PROPOSAL FOR SEISMIC ANALYSIS OF CONFINED ADOBE**

The proposal for the seismic analysis of confined adobe buildings has the following specifications:

- a) The strength design force for seismic design (V for severe earthquakes) shall be given by the Seismic Code (In Peru, the Seismic Code is given by SENCICO E.030, 2003). The reduction factor for the elastic forces will be 3.
- b) It is accepted that continuous RC beams and lintels provide a rigid diaphragm in each level. Wood or steel roofs can be used properly connected to the beams. Besides, a plan ratio of length to width for the building must be less than 4.

- c) The plan and elevation irregularities shall be avoided. In such a case, the building should be divided into several blocks by seismic joints, and each separate block should be analyzed independently.
- d) The walls that carry seismic loads must have vertical continuity and their length should not be less than 1.2m.
- e) The elastic modulus for adobe masonry can be taken as  $E_m=650$  MPa, and the shear modulus can be taken as  $G_m=0.4 E_m$ . The elastic modulus of concrete (normal weight) shall be given by the expression  $E_c = 4700 \sqrt{f'_c}$  (in MPa).
- f) To determine the lateral stiffness of confined adobe walls, the confining RC columns shall be transformed to equivalent adobe using the ratio of elastic modulus  $E_c/E_a$ . In addition, the transverse walls shall be included using an effective width of 4 times the thickness of the transverse wall. If small walls under windows openings are not isolated from the main structure, they must be considered in the analysis.
- g) The internal forces in each wall under severe earthquakes are called the flexural moment  $M_u$  and shear force  $V_u$ . For the determination of  $V_u$  and  $M_u$ , rational calculations are required (manually or computational) taking into account the eccentricities of the seismic forces specified by the Seismic Code.
- h) The maximum inelastic story drifts shall not exceed the value of 0.005. If this limit is surpassed, the stiffness of the building must be increased.

### **PROPOSED SEISMIC DESIGN FOR CONFINED ADOBE WALLS UNDER IN-PLANE FORCES**

The proposal for the in-plane seismic design of confined adobe walls has the following specifications:

For step a) the Shear Strength  $V_R$ , for the ultimate strength condition of adobe units laid with mud mortar shall be obtained using Equation 1:

$$V_R = 0.5L t + 0.2P \quad (1)$$

In which: L is the total wall length including the columns; t is the wall width without cover, and P is the accumulated gravity load.

In case the adobe units were laid using cement mortar,  $V_R$  shall be increased by a 1.3 factor. If a cement cover is placed with a mesh properly connected to the masonry, the width "t" can include the cover thickness.

In step b) the wall density in each direction of the building and in each floor, shall be calculated through the sum of shear force strength ( $\Sigma V_R$ ) for all the walls that carry seismic loads in the direction of the analysis. The sum of these wall shear capacities should be equal or greater than the actual seismic load acting in the story under analysis (Equation 2), specified by the Seismic Code.

$$\sum V_R \geq V \quad (2)$$

In the case that  $\Sigma VR > R \cdot V$ , in which  $R=3$ , the walls will behave elastically under severe earthquakes. Such walls will be designed according to step “e” of this section.

Step c) is the Check for Moderate Earthquakes. It is assumed that moderate earthquakes produce a lateral force equal to 50% of that of severe earthquakes. Complying with equation 3 will ensure that none of the walls exceeds 60% of their shear resistance under moderate earthquakes. This means that for moderate earthquakes none of the walls should have shear cracks.

$$0.5Vu \leq 0.6 VR \quad (3)$$

Step d) is the design of the walls with shear cracking. Under severe earthquakes, it is assumed that all the walls reach their shear capacity  $VR$ , except when  $\Sigma VR > R \cdot V$  (see step b of this section). For the walls that develop shear cracking, the RC confinements shall be designed with the following procedure:

The internal forces in the RC confinements are three: Tension (T), Compression (C) and Shear ( $V_c$ ). There is no flexure moment as the column cannot bend because it is connected to the wall. For design purposes, the compression C may be neglected, due to the fact that the walls are relatively low (up to two stories), and the cross section of the columns is large enough to take this compression. For the Tension and Shear Forces, the expressions given in Table 1 are proposed.

**Table 1: Internal Forces in Columns**

COLUMN	$V_c$ (shear force)	$T$ (tension)
Interior	$\frac{VR \cdot L_m}{L(N_c + 1)}$	$VR \frac{h}{L} - P_t$
Border	$1,5 \frac{VR \cdot L_m}{L(N_c + 1)}$	$F - P_t$

In which:

$F = M / L$  = axial load in border columns produced by M.

$M = Mu1 (VR / Vu)$  = flexural moment at shear cracking.

$L_m$  = length of the longer span or 0.5 L, whichever is larger.

$L$  = total wall length, including the columns

$N_c$  = total number of columns in the wall under analysis

$h$  = story height under analysis

$P_t$  = vertical load coming from the transverse wall into the column

In walls of only one span, there are two border columns,  $N_c = 2$ , and  $L_m=L$ .



The confinement column design deals with the cross section area ( $A_c$ ) and the longitudinal reinforcing steel area ( $A_s$ ). The confinement column should be able to resist the combined tension  $T$  and shear friction  $V_c$ , for which equations 4 are used.

$$A_c = \frac{V_c}{0.2 f'_c \phi} \geq 100t \text{ (in } mm^2) \quad A_{sf} = \frac{V_c}{f_y \cdot \mu \cdot \phi} \quad A_{st} = \frac{T}{f_y \cdot \phi} \quad (4)$$

$$A_s = A_{sf} + A_{st} \geq \frac{0.1 \cdot f'_c A_c}{f_y} \quad (4) \text{ continued}$$

In which the minimum column reinforcement  $A_s$  is 2-6mm bars.

$\phi = 0.85$  for the combined action of tension  $T$  and shear  $V_c$

$\mu = 0.80$  is the friction coefficient for concrete-concrete

The column stirrups shall be closed ties or hooks made of 6mm bars, with spacing 1 at 50mm, 4 at 100mm, and the rest at 250 mm.

The collar beam reinforcement shall be calculated using equation 5. The concrete area of the collar beam  $A_{cs}$ , shall be a minimum given by 100 t (in  $mm^2$ ). The stirrups shall be closed ties or hooks of 6mm bars, spaced 1 at 50mm, 4 at 100mm, and the rest at 250 mm.

$$T_s = VR \frac{L_m}{2L} \quad A_s = \frac{T}{\phi f_y} \geq \frac{0.1 f'_c A_{cs}}{f_y} \quad (5)$$

In the collar beam design,  $\phi = 0.9$ . The minimum reinforcement  $A_s$  shall be 2-6mm bars. The longitudinal beam bars shall be anchored at the nodes; also, additional stirrups can be placed in the column to anchor the beam bars with a 90° hook.

In the cracked walls, continuous horizontal reinforcement should be placed, anchored into the columns. This reinforcement should be placed on horizontal joints filled with cement mortar. The amount of horizontal reinforcement,  $A_{sh}$ , shall be calculated using equation 6, in which “s” is the vertical spacing between horizontal bars. The minimum reinforcement in any story, either cracked or not, shall be 1-6mm bar every 5 layers.

$$A_{sh} = \frac{VR \cdot s}{f_y \cdot L} \quad (6)$$

The design of non-cracked walls shall be done whenever the building has enough shear resistance, that is when  $\Sigma VR > R V$ . In the interior columns and the collar beams of non-cracked walls, a minimum reinforcement may be used. Also, in such walls, it is not necessary to add shear

friction reinforcement in the columns. Only the border columns need to be designed, using equation 7.

$$T = F - P_i \qquad A_s = \frac{T}{\phi f_y} \geq \frac{0,1 f'_c A_c}{f_y} \quad (7)$$

In which,  $\phi=0.9$ , and  $F = Mu / L$ , is the axial force in the border columns, produced by moment  $Mu$ . Also, the minimum reinforcement  $A_s$  shall be 2-6mm bars.

The lintel beams shall be designed using a combined action of gravity loads and seismic loads. Such beams should be designed in a ductile manner, so that a flexural failure is dominant, as specified in the Peruvian Reinforced Concrete Code (“Norma E.060”, SENCICO 2009).

### **PROPOSAL DESIGN FOR OUT-OF-PLANE SEISMIC LOADS**

The confined adobe masonry and with horizontal reinforcement, does not need any further design for out-of-plane seismic loads. Only the braces need to be designed, for which the following specifications shall be used.

The seismic out-of-plane load  $w$ , shall be calculated using equation 8, according to the Peruvian Seismic Code (in Peru, “Norma E.030”).

$$w = Z.U.C_1 \gamma e \quad (8)$$

$Z, U$  = zone factor and Importance factor, specified in the Seismic Code.

$C_1$  = seismic coefficient specified in the Seismic Code.

$e$  = gross width of the wall, including the covers if any.

$\gamma$  = volumetric unit weight of adobe masonry, which can be assumed as  $160 \text{ kN/m}^3$ .

The RC confinements act in this case as braces for the adobe wall. The load that goes from the wall “ $w$ ” to the braces can be obtained using the envelope rule, which consist in lines with a  $45^\circ$  slope from the wall vertices that intercept a horizontal line drawn at mid height of the wall.

Examples of braces that can have flexure deformations are a collar beam without a roof connection, and a column at a free border of the wall. In such braces with flexural deformations, the loads (either triangular or trapezoidal distributed), will produce flexural moments and shear forces, which must be resisted by the cross section and its reinforcement, as specified in the Reinforced Concrete Codes (“Norma E.060” in Peru).

The cross section requirements or its reinforcement obtained in the RC elements acting as braces, need not to be added to those values when these elements act as confinements. Only the larger values should be adopted.

### **CONCLUSIONS**

A rational design proposal for the seismic design of confined adobe buildings has been presented, limited to two stories high. The proposal is based on several experimental static and dynamic tests on full scale models, and the knowledge of brick confined wall seismic behavior.

In order to improve the confined adobe design proposal presented in this paper, some additional experimental research must be done. For instance, important research pending is the verification under out-of-plane seismic loads of adobe walls located on a second story, with the reinforcement specified in this proposal. Up to date, the observed seismic behavior under real earthquakes and in the lab tests indicates that it is possible to use this proposal safely.

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