SEISMIC BEHAVIOR OF A TWO STORY MODEL OF CONFINED ADOBE MASONRY

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ABSTRACT

Many traditional adobe houses located in the Andean highlands are seismically vulnerable due to lack of reinforcement. Of these houses, many have two or more stories, which makes them even more vulnerable. A research project was conducted applying well known confined masonry walls concepts used with clay bricks, now to adobe masonry. A two story full size model was constructed and tested at the Structures Laboratory of the Pontifical Catholic University of Peru. The model was designed using low resistance concrete with minimum reinforcement in the confinement elements. Horizontal bars were also used as reinforcement in the first story.

Several tests, which are common for brick masonry specimens, were also performed on adobe specimens. These included compressive resistance of adobe units, axial compression on small prisms and diagonal compression on small square walls to obtain the shear resistance.

The two story model was tested on a shaking table under horizontal movements of increasing amplitude, completing a total of 5 steps until partial collapse. The behavior observed was good in some aspects, such as shear capacity and adequate flexural resistance to out of plane forces in the first story, horizontally reinforced with steel bars. However, the walls of the second floor, without horizontal bars, had a partial collapse due to out-of-plane forces, indicating that some other aspects have to be improved.

KEYWORDS: confined adobe, confinement, adobe, shaking table, reinforcement, out-of-plane.

INTRODUCTION

In the Andean highlands, especially in Peru, many traditional adobe houses have two or more stories (Figure 1). These constructions usually have no reinforcement, and when they have been hit by an earthquake, they had collapsed causing injuries or even death of their occupants. However, adobe is still the best solution for poor people housing in highland areas, due to economic reasons; also, earth units are the only available material suitable for walls due to their thermal properties. This project was thought on how to reinforce such two story houses, by using a seismic resistant technique that can be simple, economic and able to inspire confidence in the inhabitants so they can accept it easily.

Figure 1: Traditional two-story adobe houses in Peru highlands

The reinforcement technique thought for the adobe houses was similar to the ones used in brick confined masonry, with low resistance concrete and minimum reinforcement, adding horizontal steel bars in order to integrate the wall with the confinement columns. The use of confinement RC elements with adobe walls proved to be successful when subjected to lateral cyclic loads [1]. A structural design method was proposed by San Bartolome [2], which was applied to the two-story model of this project.

The Peruvian Adobe Code of 2000 [3] allows the use of reinforced concrete elements, which in one-story houses have shown good earthquake behavior. Reports on the Ometepec, Mexico 1995 Ms=7.2 earthquake [4], and the Nazca, Peru 1996, M=7.5 earthquake [5] include comments on such observations (Figure 2). Therefore, the objective of this project was to determine if the proposed reinforcement for two-story houses has good behavior and/or drawbacks.

Figure 2: Good behavior of one-story adobe houses confined by RC elements, Ometepec Mexico [4] (left) and Nasca Peru [5] (right)

MATERIALS

The adobe units used for construction of the specimens had dimensions of 310x180x85 mm, with a compressive resistance of 1.8 MPa. This value exceeds the minimum specified by the Peruvian Adobe Code which is 1.2 MPa.

Mud mortar was used in a mix volume proportion of 3 parts of soil, 1 part of coarse sand, and 1 part of straw. In the horizontal joints of the first story where reinforcement bars were included (every 3 layers) the mortar was changed to a cement-sand 1:5 volume proportion. In this way, the steel bars are protected from corrosion and adherence is provided.

Columns and collar beams were built using a low resistance concrete. The average resistance of the test cylinders was 14 MPa.

Steel bars with a nominal yield stress of 420 MPa, usually used in RC elements, were used in the model. All bars were 6.3 mm (1/4”) in diameter, as horizontal reinforcement for the first story walls, as well as the longitudinal bars and ties for columns and collar beams in the two stories.

Compression tests were performed on 4 prisms (Figure 3). These prisms had 7 layers with 20 mm joint thickness. The average compressive resistance was f’m=0.9 MPa. Four small walls with 0.80m side were also constructed in order to perform diagonal compression tests to evaluate the shear resistance. However, some units of these small walls had such a weak adherence that they detached from the wall during the handling prior to the test. Therefore, the shear resistance values were very low and with high dispersion, so those values are not useful.

Figure 3: Adobe prisms and small walls

MODEL CHARACTERISTICS

A two-story adobe model was constructed for testing on the shaking table. The table capacity at the Structures Laboratory is 160 kN of weight and has a single horizontal degree of freedom. The Peruvian Adobe Code limits the wall thickness in confined masonry to 250mm, but due to the table limitation, a ¾ length scale was used. The adobe wall thickness used was 180 mm. The two walls parallel to the movement (shear walls) had a length of 1.36m, while the wall subjected to out of plane movement (flexure wall) had 3.16m in length, the maximum that could be accommodated in the table platform. In this way, the specimen weight without foundation, was 71.2 kN. The confinement elements were designed using Peruvian Seismic Code forces [6] and the proposal for confined adobe [2] for the prototype.

The reinforcement should be able to resist the diagonal cracking load of the walls parallel to the movement. Then, using the same steel ratio, the reinforcement for the scaled model was obtained. Columns had a section of 180 mm square, with 4 – 6.3 mm (1/4") longitudinal bars, and closed ties 1 at 50mm, 4 at 100 mm, rest at 250 mm. The collar beams had a 180x150mm section with 2 – 6.3 mm (1/4") longitudinal bars, and hooks 1 at 50mm, 4 at 100 mm, rest at 250 mm. The first story featured 1 bar 6.3 mm (1/4") every 5 layers, along the wall and anchored in the columns (Figure 4).

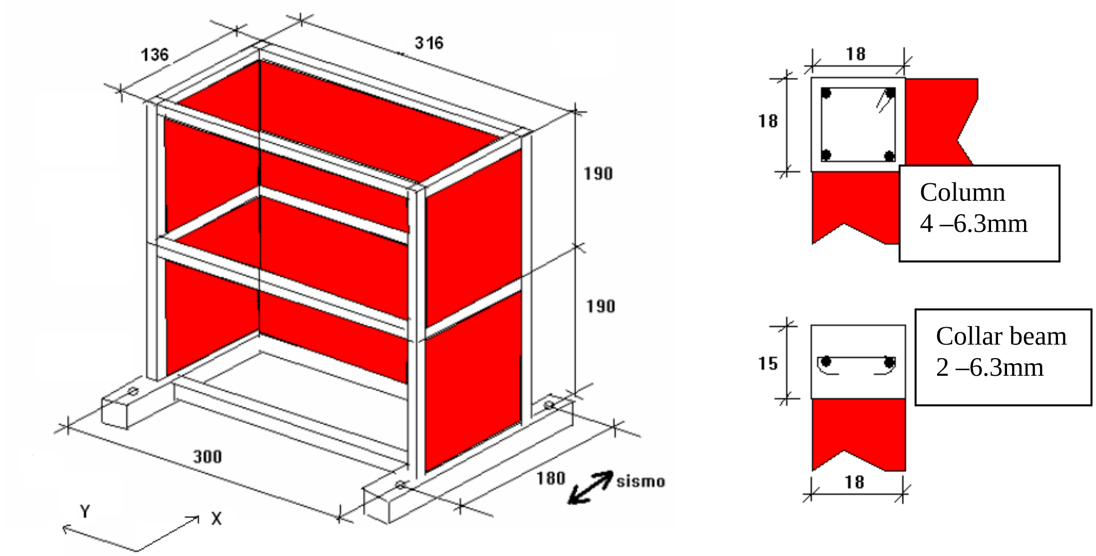

Figure 4: Adobe two-story specimen

MODEL CONSTRUCTION

The construction procedure is described as follows (see Figure 5 for the first story and Figure 6 for the second story). First, the adobe units were wetted prior to placing to reduce their suction. The adobes were placed using mud mortar and had 20 mm joints controlled with a stick. The adobe-column connection was vertical and a one-6.3 mm horizontal bar was placed inside a joint of 1:5 cement mortar every 5 layers. After the adobe wall was finished, the concrete of the columns was poured. Next, the collar beam concrete was poured. The removal of forms was done the day after the pouring. For the second floor, the construction procedure was similar, except that no horizontal reinforcement was included and no anchoring bars were used to join the columns and adobe walls. To simulate the stories, wood beams were used laying on the longer collar beams and connected with wires in U shape embedded in the longitudinal collar beams.

SEISMIC SIMULATION TEST

The input signal for the seismic simulation test was derived from the L component of the record taken during May 31, 1970 earthquake (Ms 7.75) in Lima, Peru. The duration was compressed by a factor of ¾, keeping all the signal data. The idea was that the ratio between the signal excitation (4 Hz) and the specimen natural frequency (9.5 Hz), be similar to the ratio between the frequency of the real earthquake and the prototype natural frequency. Also, it is important to mention that both model and prototype are made out of the same material, and therefore, the elastic modulus (E, G), specific weight, damping ratio and resistant stresses are the same.

Figure 5: Details of the first story construction

Figure 6: Details of the second story construction

The movement intensity was varied in order to produce mild, moderate, severe and extreme simulated earthquakes. Table 1 indicates the steps followed with the corresponding table accelerations. The Peruvian Seismic Code establishes a peak ground acceleration of 0.4g for firm soil in seismic zone 3, which has the highest risk. A peak of 0.3g is established for firm soil in the seismic zone 2 (most of Peruvian highlands). A soil factor of 1.4 is required for bad soil conditions.

Table 1: Steps of the Seismic Simulation Test

Step	1	2	3	4	5
Ao (model)	0.20g	0.40g	0.60g	0.80g	1.00g
Ar (prototype)	0.15g	0.30g	0.45g	0.60g	0.75g
Movement	Mild	Moderate	Severe		Extreme

The instruments used in the test, shown in figure 7, were 9 accelerometers (Ai) and 13 displacement transducers LVDT (Di). Accelerometers A1 and A2 and LVDTs D1 and D2 were located on the first and second story of one wall parallel to the movement. Accelerometers A3 through A8 and LVDTs D3 through D6 were located on the wall and beams perpendicular to the movement. D7 and D8 recorded relative displacements between the concrete columns and adobe walls, D9 and D12 recorded the cracks at the mid height of each story, and D10 and D11 recorded vertical relative displacements between the foundation and concrete columns. The table platform movement is also measured by accelerometer A_0 and LVDT D_0.

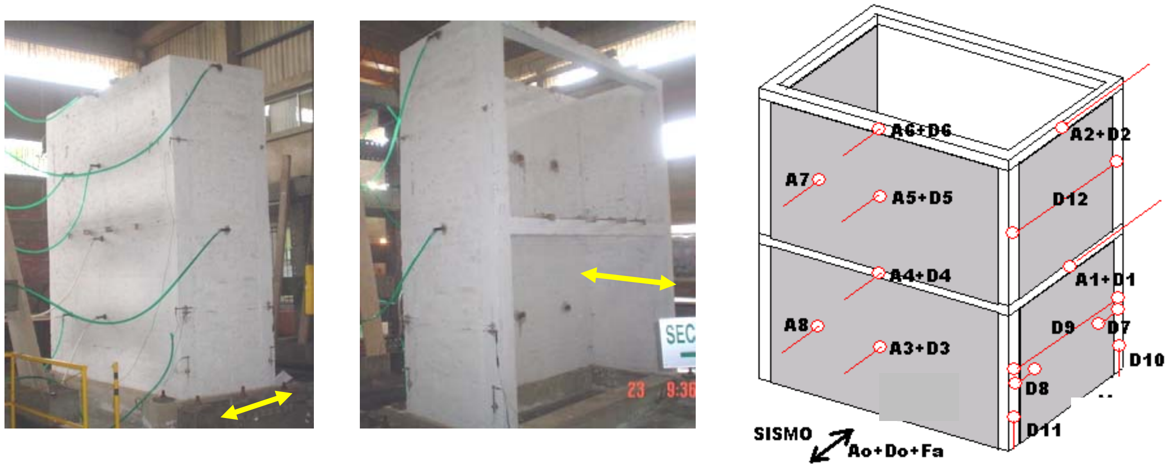

Figure 7: Global view of the specimen and instruments

After step 1, a horizontal crack appeared at the base of the second floor flexure wall. No other cracks were observed.

After step 2, the first crack become thicker, and vertical cracks started at the connection between column and flexure wall in the mid height of the second story (Figure 8). A stair-step diagonal shear crack appeared in the first story shear walls and in the lower half of the second story shear walls. At this stage, no cracks had appeared in the columns or collar beams. The failure probability of the second story flexure wall was quite high at this moment so the accelerometers A5, A7 and LVDT D5 were removed. Two wooden columns were installed at the model outside (with 30 mm gap with the adobe wall) to prevent outward collapse.

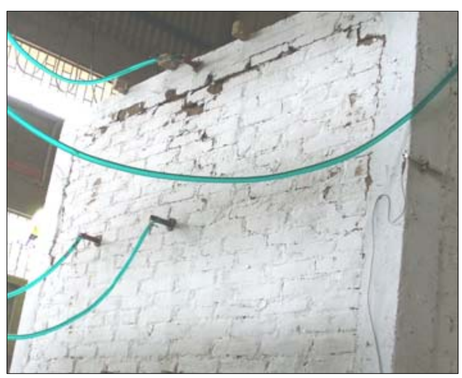

Figure 8: Second story flexure wall after step 2.

At the end of step 3, one unit of the upper layer of the second story flexure wall fell down, and it could be seen that this wall was near collapse (Figure 9). The stair-step shear cracks in the shear walls were amplified. Also, some vertical cracks appeared at the first story column-collar beam connection. It must be mentioned that no cracks appeared in the first story flexure wall.

During step 4, the central triangular part of the second story flexure wall collapsed (Figure 10). The shear cracks continued increasing their width, but the adobe units were not crushed. The first story flexure wall remained without cracks.

Figure 9: Specimen during step 3 (left) and after step 3 (right).

Figure 10: Specimen during step 4 (left) and after step 4 (center and right).

During step 5, the whole second story flexure wall collapsed (Figure 11). Some small cracks started to appear in the first story flexure wall. All the shear cracks widened as well as those cracks in the connection with the columns; however, the adobe units remained uncrushed.

Figure 11: Specimen during step 5 (left) and after step 5 (right).

TEST RESULTS

Table 2 presents the peak values reached by the instruments during the shaking table test. The displacement relative to the platform is called di = Di – D_0.

The base shear force at the model was obtained using the actuator force (Fa) and the base inertia force as indicated in Equation 1.

$$V = Fa - m A_0 \quad (1)$$

In which m = mass of the platform + mass of the foundation

The shear stress τ, was calculated dividing the base force V of the specimen by the shear area of the two walls parallel to the movement: 2 x 180 mm x 1360 mm. Also, the angular distortion was obtained dividing the displacement d1 by the height of the first story, 1.9 m.

Table 2: Peak values reached by the instruments during shaking table test.
Shear stress τ and angular deformation γ at the first story.

Instrument	Step 1		Step 2		Step 3		Step 4		Step 5	
Ao (g)	-0.19	0.21	-0.33	0.39	-0.47	0.61	-0.66	0.82	-0.88	1.05
A1 (g)	-0.38	0.40	-0.51	0.73	-0.75	0.70	-0.78	0.86	-1.08	1.17
A2 (g)	-0.63	0.57	-0.87	1.25	-1.50	1.59	-2.22	1.95	-3.15	2.37
A3 (g)	-0.25	0.34	-0.51	0.53	-0.88	0.81	-1.17	1.30	-1.81	1.78
A4 (g)	-0.43	0.52	-0.76	0.69	-0.93	0.95	-0.98	1.41	-1.77	2.16
A5 (g)	-1.17	0.98	-1.85	2.05	Accelerometer was removed					
A6 (g)	-0.68	0.65	-1.06	1.29	-1.84	1.35	-2.06	2.08	-3.00	2.67
Do (mm)	-14.82	11.06	-28.86	21.87	-43.55	32.74	-58.35	43.91	-73.93	55.51
d1 (mm)	-2.89	2.43	-6.04	5.34	-12.83	12.81	-21.76	20.13	-36.45	34.86
d2 (mm)	-4.46	3.88	-9.98	9.66	-22.29	24.50	-41.15	41.58	-58.21	70.20
d3 (mm)	-1.43	1.56	-9.04	26.20	LVDT was damaged					
d4 (mm)	-3.43	3.39	-6.89	9.02	-18.33	18.04	-32.41	----	-41.09	36.82
d5 (mm)	-7.69	9.57	-20.27	28.33	LVDT was removed					
d6 (mm)	-5.21	5.19	-11.60	13.99	-26.88	31.60	-52.03	49.34	-71.15	79.34
D7 (mm)	0.08		0.30		0.77		0.89		0.82	
D9 (mm)	0.10		0.47		1.07		1.73		1.82	
D10 (mm)	-0.14	0.23	-0.19	0.42	-0.19	0.89	-0.19	1.69	-0.42	2.22
D12 (mm)	0.05		0.55		2.33		5.17		9.59	
V (kN)	-28.72	37.19	-33.68	36.98	-59.39	49.50	-64.38	69.70	-71.93	94.45
τ (MPa)	-0.059	0.076	-0.069	0.076	0.121	0.101	-0.131	0.142	-0.147	0.193
γ (story 1)	-0.002	0.001	-0.003	0.003	-0.007	0.007	-0.011	0.011	-0.019	0.018

The envelope of the shear stress – angular distortion curves for the first story (Figure 12) was calculated using the peak values of maximum shear stress obtained at either the positive or negative loop of the V-d curve for each step. In figure 12, between steps 1 and 2, an almost flat relation can be observed. This fact agrees with the diagonal cracking of the shear walls. Afterwards, there was a continuous increase in the shear resistance that could be attributed to the horizontal reinforcement in the first story wall.

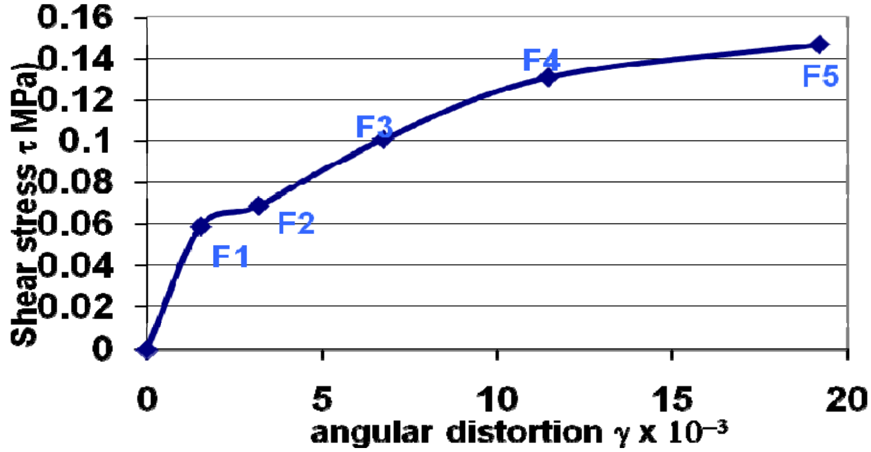

Figure 12: Shear stress vs. Angular distortion for first floor

NUMERIC RESULTS DISCUSSION

First, the shear resistance will be checked. San Bartolome proposal includes equation 2 to obtain the shear resistance VR of the adobe wall (in N), in terms of the wall shear area t L(t is the thickness and L is the wall length, both in mm) and the axial load P (in N):

$$VR = 0.05\ L\ t + 0.2\ P \qquad (2)$$

For the first story walls, t=180 mm, L = 1360 mm, and P=19200 N. Applying these values to equation 2 it yields a shear force VR = 16080 N. Then, the shear stress at diagonal cracking, τ_R, is found to be τ_R = 16080 / (180 x 1360) = 0.066 MPa. This result agrees pretty well with the experimental value obtained in the test at step 2, 0.069 MPa. Therefore, equation 2 gave a good prediction of the adobe wall shear capacity.

On the other hand, the shear stress value obtained in step 5 was 0.147 MPa, much higher than the shear stress previously obtained at cracking. This could be attributed to the yielding of the horizontal reinforcement. The stress of the actual reinforcement τ_s, may be obtained dividing the steel force at yielding (32 mm^2 x 420 MPa = 13440 N) by the area t s (180 x 525= 94500 mm^2), in which s denotes the spacing of the bars. The yield stress is then τ_s =13440/94500 = 0.142 MPa. The close agreement between the test value and the reinforcement yield has still to be verified by other tests, such as cyclic static load tests.

Finally, the angular distortion obtained without loss of resistance during step 5 reached 0.018. However, for the system to be reparable, the adequate maximum distortion could be set at 0.005, which corresponds to an intermediate situation between steps 2 and 3. This seems adequate because the flexure wall of the second floor was near collapse at the end of step 3.

CONCLUSIONS

The conclusions are limited to this only specimen, however, the seismic simulation test helped to point out some weak zones. Even for severe earthquakes good behaviour of the shear walls in the two stories and in the flexure wall of the first story was clearly observed. On the other hand, a fragile failure of the flexure wall of the second floor was observed, with collapse of the wall.

To prevent such undesirable behaviour, the inclusion of different types of reinforcement is needed, such as an extra column, a welded steel wire mesh connecting the wall with the upper beam, etc.

The use of horizontal continuous reinforcement in the first story adobe walls was effective to control the cracking in the shear walls. Also, the low resistance concrete confinements, and the minimum reinforcement used in these elements, had an adequate behaviour.

Given a ground acceleration of 0.3 g for firm soil in the highland regions of Peru (seismic zone 2), it looks promising that the reinforcement used in this research can be used for two-story adobe houses confined with concrete elements.

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