CYCLIC RESPONSE OF TRADITIONAL AND CONFINED MASONRY HAITIAN WALLS

G.Villa Garcia[1], M.Blondet[2] and D. Quiun[2]
[1] Associate Professor, Department of Civil Engineering, Catholic University of Peru, gvillag@pucp.edu.pe
[2] Principal Professors, Department of Civil Engineering, Catholic University of Peru, mblondet@pucp.pe , dquiun@pucp.edu.pe

ABSTRACT
The 2010 Haiti earthquake caused more than 233 thousand deaths and around 1.2 million people lost their houses. Most of the collapsed or damaged structures were built informally with a traditional construction system consisting of reinforced concrete frames infilled with concrete block masonry.

This paper presents preliminary results of a research project developed at the Catholic University of Peru, whose objective is to provide a safe construction technology for Haiti based on confined masonry built with local materials. In order to reproduce Haitian masonry, concrete blocks were fabricated with low strength concrete. Two full-scale masonry walls (3 m x 3 m x 0.25 m) were built and tested under cyclic lateral load. The first wall (W1) represented traditional Haitian construction: a concrete frame was built first and infilled with concrete blocks. The second wall (W2) was built with confined masonry: the wall was built first, then the confining reinforced concrete elements. Small constructive changes were also introduced in the stirrups and the wetting of the blocks prior to laying.

Both walls were tested following the same pattern. Wall W1 developed large cracks which separated the masonry from the concrete frame. In a real earthquake, this wall would overturn out of plane. Wall W2 developed the typical shear diagonal cracks and the confining elements were able to maintain the wall integrity. Wall W2 was 15% stronger than wall W1 and its failure mode was much better. These results are encouraging and the work will continue by exploring other construction improvements.

KEYWORDS: confined masonry, concrete blocks, cyclic load test

INTRODUCTION
The January 12, 2010 Haiti earthquake has been devastating in terms of human lives and material loss. This 7.0 magnitude event, with epicentre 25 km WSW of Port-au-Prince, caused more than 233000 deaths, 300000 injured and around 1200000 homeless [1].

No seismic detailing, informality and poor construction techniques were responsible for most of the damage on poorest people housing. Traditional construction consisted of small dimension, weakly reinforced concrete frames infilled with low quality, hand made, concrete block masonry units (CMU). This system, used in most houses and multi story buildings in Haiti, had undesirable seismic behaviour as can be appreciated in Figure 1 [2].

Figure 1: Earthquake destruction in masonry houses in Haiti [2]: a) Out of plane failure; b) Lack of column at corner

A research project developed at the Catholic University of Peru attempts to provide a safe and low cost construction technology for Haiti, based on confined masonry built with local materials. It is well known that confined masonry is an economic construction system that has had excellent seismic performance [3] and is extensively used in countries like Mexico, Peru [4], and Chile, for buildings from one to five stories.

This paper presents the results of the first stage which included reproducing traditional Haitian construction materials and system, determining its structural characteristics, and comparing the seismic behaviour of a full scale wall subjected to lateral cyclic load with a similar wall built with confined masonry.

CONCRETE MASONRY UNIT PREPARATION

The concrete masonry unit used in the project had the same dimensions as the typical Haitian CMU, 150 x 200 x 400 mm, with three square openings as shown in Figure 2. However, the openings did not pass through the other side, which finished in a continuous 25 mm thick layer. A metallic form was designed and fabricated with these specifications, and 800 blocks were hand made for the whole project.

.

Figure 2: CMU preparation: a) Metallic form; b) Solid surface; c) Drying

Materials used for the concrete mix of the units were Portland cement, coarse sand with F.M. 2.86 and tap water in a volume ratio of 1:9:2. This concrete mix gave 8.2 MPa concrete cylinder strength at 7 days. The compression strength of CMU was tested at 90 days giving a gross characteristic strength of 5.79 MPa. This value exceeds slightly the minimum established by the Peruvian Masonry Code [5], which is 5.0 MPa to be considered a load bearing block unit.

PROPERTIES OF CONCRETE MASONRY

Natural suction of CMU gave 69 g/200cm^2/min, a value which is too high, considering that Peruvian Masonry Code establishes a range at the time of placement of 10 to 20 g/200cm^2/min. Wetting superficially the unit, suction was reduced to 45 g/200cm^2/min. Thus, two sets of masonry specimens were tested, one without previous treatment (PS) and the other treated (PT) by wetting superficially the units immediately before laying.

Small masonry specimens were constructed to determine the masonry properties. All prisms consisted of three units stacked vertically and joined with 15 mm layer of cement-sand mortar in volume proportion 1:8. Slenderness ratio of all prisms was 4.2. Compression strength of mortar cubes was 6.2 MPa at 28 days. The prisms were instrumented with two LVDT's placed vertically on each side to determine Young's Modulus. Figure 3 shows testing set up and Table 1 presents the results of tests at more than 28 days.

Table 1: Axial Compression of Prisms

Results / Specimen	Test ID	Compression Strength – f'm			Young's Modulus E		
		Individual (MPa)	Average (MPa)	Standard Deviation	Individual (MPa)	Average (MPa)	Standard Deviation
No wetting	PS-1 PS-2 PS-3	2.50 3.81 3.59	3.30	0.7	3778 3556 4133	3822	291
Wetting	PT-1 PT-2 PT-3	3.02 2.87 2.86	2.92	0.1	3354 3785 2136	3092	855

Figure 3: s) Axial compression of prism; b) Diagonal compression of wallet

To determine the shear strength of masonry, almost square 820 mm side wallets were prepared and tested in diagonal compression. As can be appreciated in Figure 3, each wallet had two LVDT's perpendicularly placed at the centre of the loaded span, to determine shear modulus G. Table 2 presents results of both sets of wallets: without treatment (MS) and with previous watering (MT).

Table 2: Diagonal compression of wallets

Results / Specimen	Test Number	Shear Strength – v'_m			Shear Modulus G		
		Individual (MPa)	Average (MPa)	Standard Deviation	Individual (MPa)	Average (MPa)	Standard Deviation
Not wetted	MS-1 MS-2 MS-3	0.48 0.40 0.41	0.43	0.04	1462 985 1180	1209	240
Wetted	MT-1 MT-2 MT-3	0.34 0.47 0.29	0.37	0.09	613 749 1108	823	256

CONSTRUCTION OF WALLS

Two masonry walls were built varying the construction sequence of RC columns and masonry wall. Each wall was built over a RC foundation beam of dimensions 300 x 400 x 3900 mm. Columns cross section was 150 x 250 mm. Reinforcement consisted of 4 Φ 9.5 mm (3/8") bars for both walls. Stirrups of 6 mm with 90° hook were spaced uniformly every 250mm for Wall 1 (W1) and for Wall 2 the hook was 135° and they were spaced 1@50mm, 4@100mm and the rest @250mm. Crown beam cross section was 150 x 200 mm. It was reinforced with 4 Φ 9.5 mm (3/8") bars and stirrups of 6 mm spaced uniformly every 250mm for both walls.

Average concrete strength was 27.8 MPa for the foundation beams, 17.1 MPa for the columns and 15.2 MPa for the crown beams.

The main difference between both walls was the construction process. W1 simulating Haitian construction was built with CMU laid without previous treatment. The concrete columns and the top beam were built first as a normal RC frame, and later, CMU were infilled to the frame. W2 was built with CMU wetted superficially prior to laying. The masonry wall was built first, leaving the ends toothed. After the masonry wall was finished, the end columns and crown beam concrete was poured producing a confined wall. Figure 4 shows the overall dimensions of the walls and the difference in the construction sequence of each wall.

LATERAL CYCLIC LOAD TESTS

The testing sequence was the same for both walls. Displacement controlled lateral cyclic load was applied in several steps. Each step was defined by its maximum horizontal displacement and consisted of a number of load cycles, each applied at a speed of 1 cycle every 4 minutes. Table 3 presents the testing sequence.

Table 3: Testing sequence for walls

Step	1	2	3	4	5	6	7	8	9	10	11	12	13
D1 (mm)	0.5	1.5	2	3	4	5	7.5	10	12.5	15	20	40	80
Cycles	2	3	3	3	3	3	3	3	3	1	1	1	1

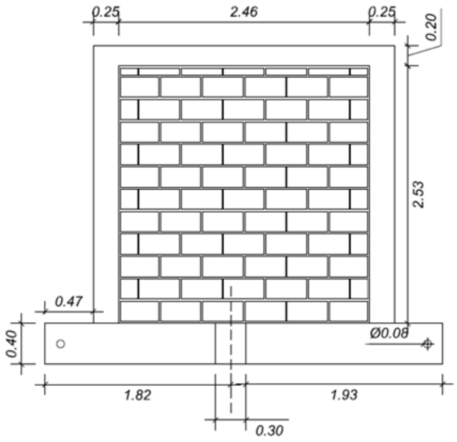

Figure 4: Wall construction: a) Overall dimensions; b) Infilled masonry in RC frame; c) Confined masonry

Instrumentation consisted of 10 LVDT's as shown in Figure 5: D1 was used for test displacement control, D2 and D3 recorded the diagonal elongation at the centre of the wall, D4 and D5 recorded the vertical separation between column and foundation beam, D6 and D7 measured total diagonal elongation of the wall, D8 and D9 monitored horizontal separation between wall and columns and D10 monitored horizontal movement between the wall and the crown beam.

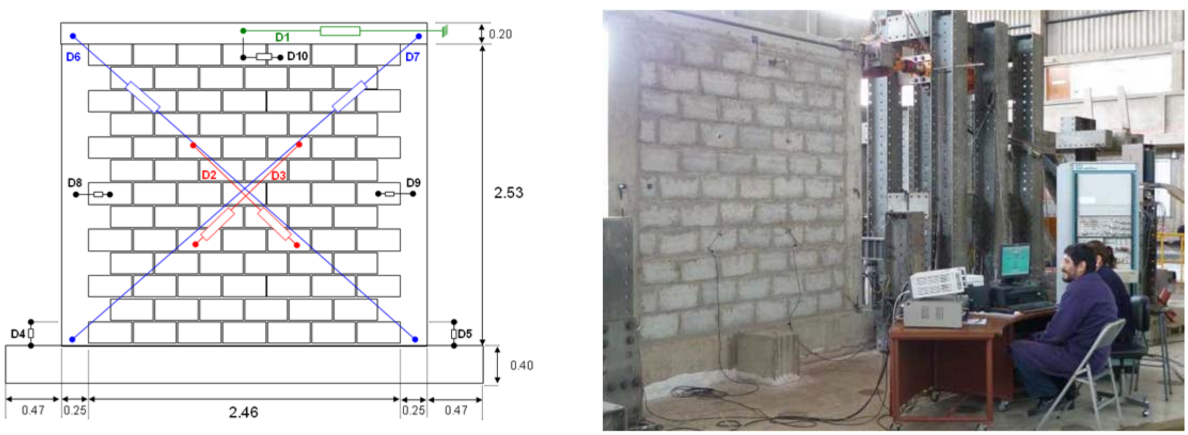

Figure 5: a) Wall instrumentation; b) Testing set up.

Failure pattern was different in both walls. Wall W1 developed fine vertical separation cracks (0.15mm) between the masonry wall and RC columns in step 1. This separation process advanced to complete vertical separation from columns (step 2) and horizontal separation from both the foundation beam (step 3) and the top beam (step 4). At this point, average separation crack width was 0.5 mm. At step 6, separation cracks were 3 mm wide and during step 7, cracks passed through the wall. It was only in step 8 that diagonal shear cracks started to appear in W1.

Wall W2 had no visible cracks in step 1, and starting step 2, it developed tension flexure horizontal cracks in the columns which extended to the wall joints. Figure 6 shows crack detail of both walls at step 3, where the different behaviour is observed. At step 4 many diagonal cracks appeared, which developed in stairways through the joints. Neither vertical nor horizontal wall separation cracks appeared in W2 during the test.

Figure 6: Step 3 for: a) W1; b) W2.

The Peruvian Seismic Code [6] establishes that the maximum drift for a masonry structure should be limited to 0.005 (1/200), which corresponds to step 9 of this test sequence. However, further steps were carried on to observe the final failure mode of each wall. Thus, Figure 7 shows the walls at the end of step 9 and Figure 8 presents the walls after step 13.

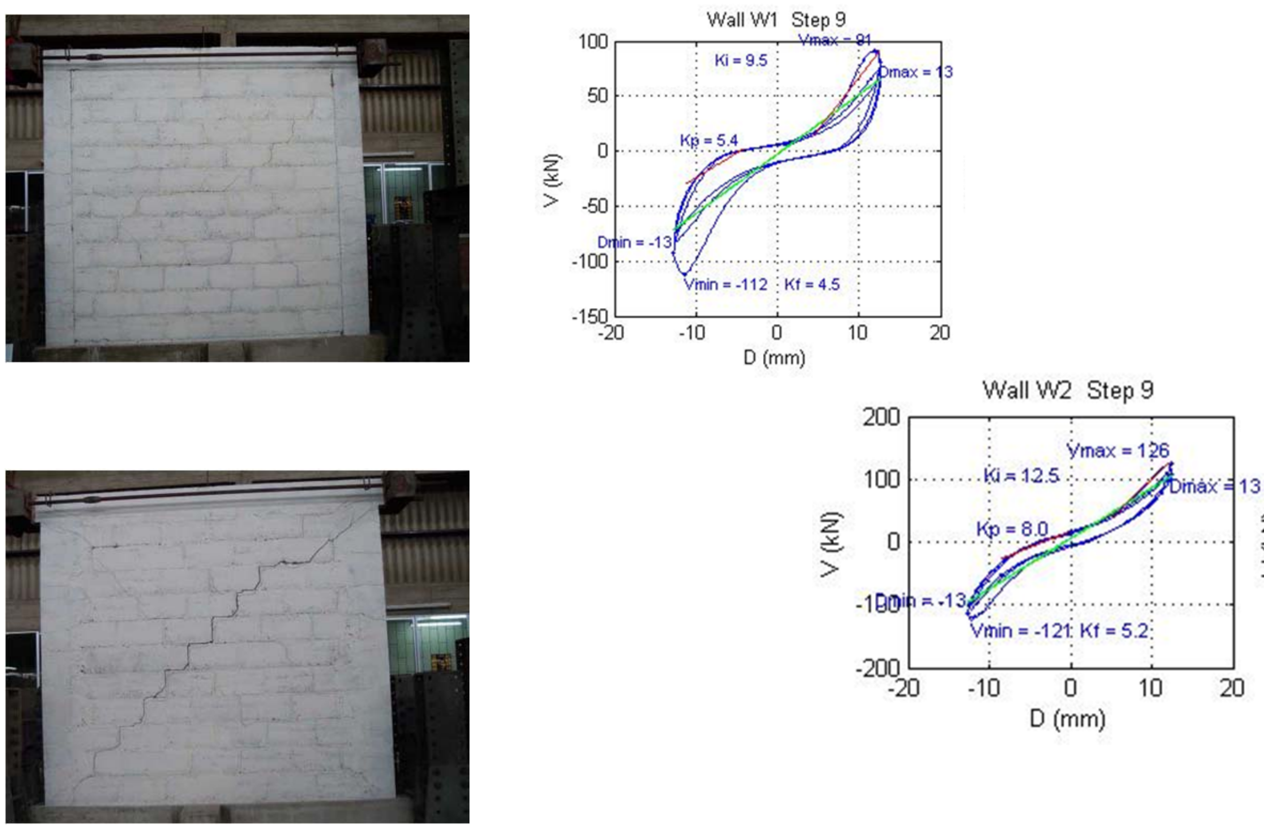

Figure 7: Step 9 for: a) W1 and hysteretic curves; b) W2 and hysteretic curves

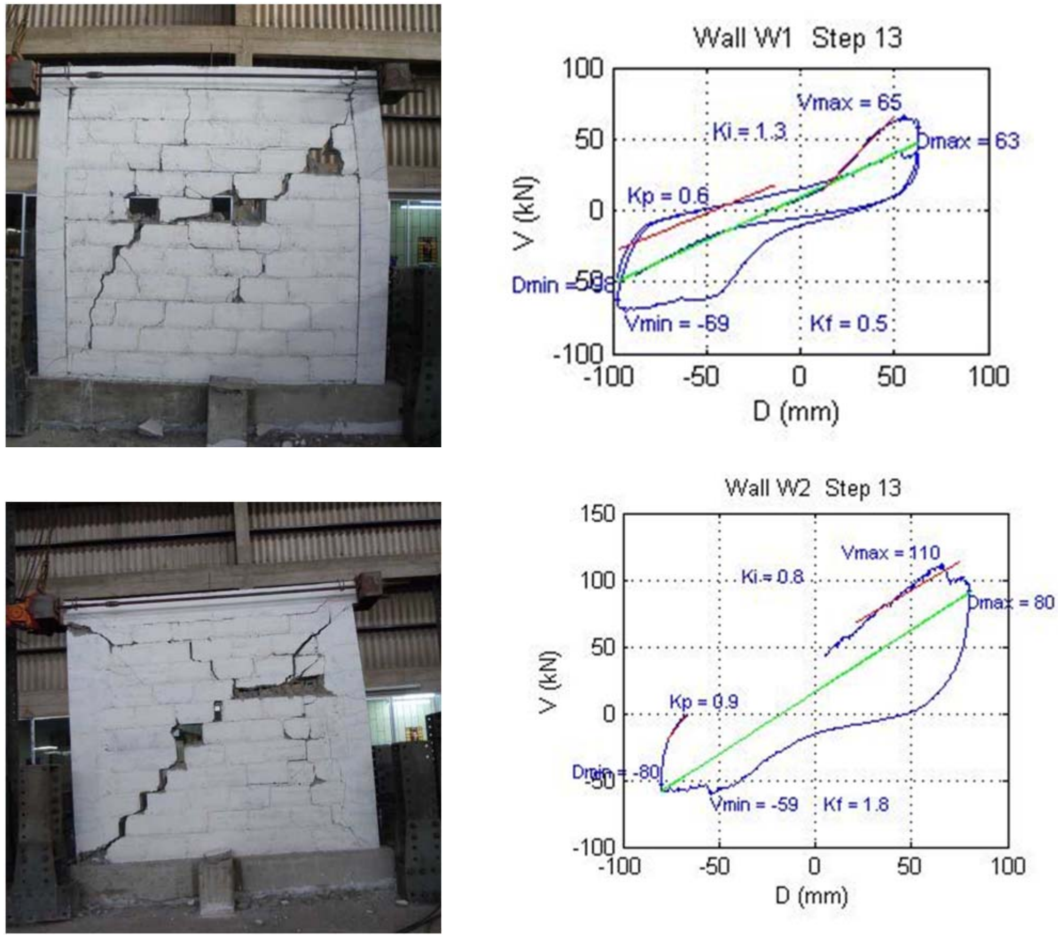

Figure 8: Step 13 for: a) W1 and hysteretic curve; b) W2 and hysteretic curve

Table 4 compares the peak values and Figure 9 shows the complete hysteretic loops of lateral force vs. lateral displacement for both walls.

Table 4: Comparison of peak values between W1 and W2

Parameter	Unit	W1	W2
First crack load	kN	34.7	91.36
Maximum horizontal load	kN	118.9	141.6
Initial stiffness	kN/mm	90.6	88.6
% of initial stiffness at step 9	%	10.5	14.1
Maximum displacement between wall and crown beam	Mm	12.04	1.37

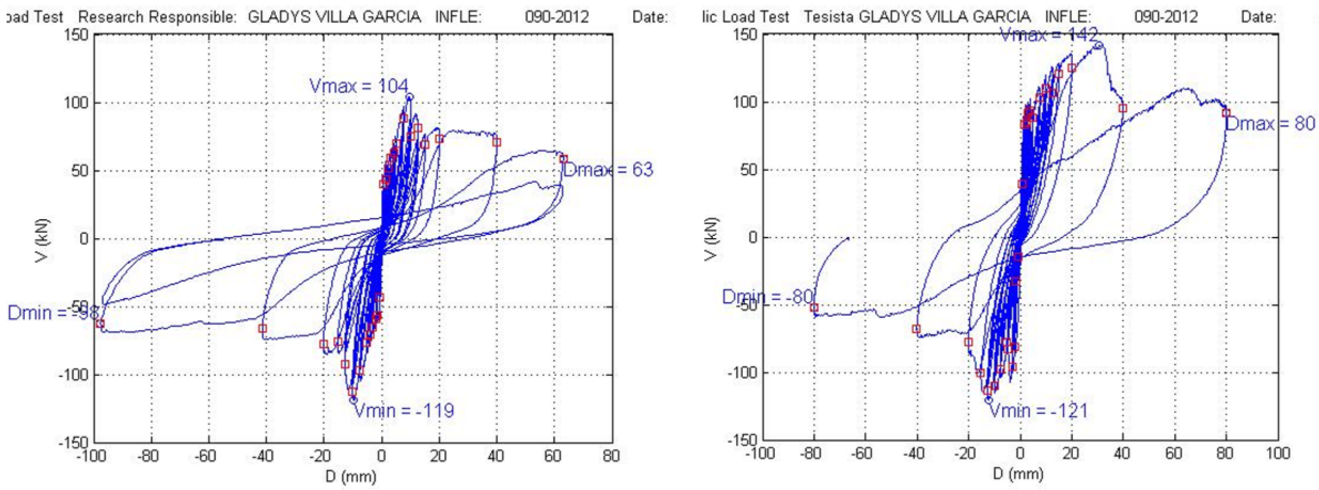

Figure 9: Hysteretic loops for: a) W1; b) W2

As expected, initial stiffness of both walls is quite similar, because the difference between them is only in the constructive process. Average stiffness has been calculated from the first half cycle and the last half cycle of each step. Figure 10 compares average stiffness and stiffness loss in both walls. At step 3 traditional wall W1 has only 34% of initial stiffness, while confined wall W2 still has 52%.

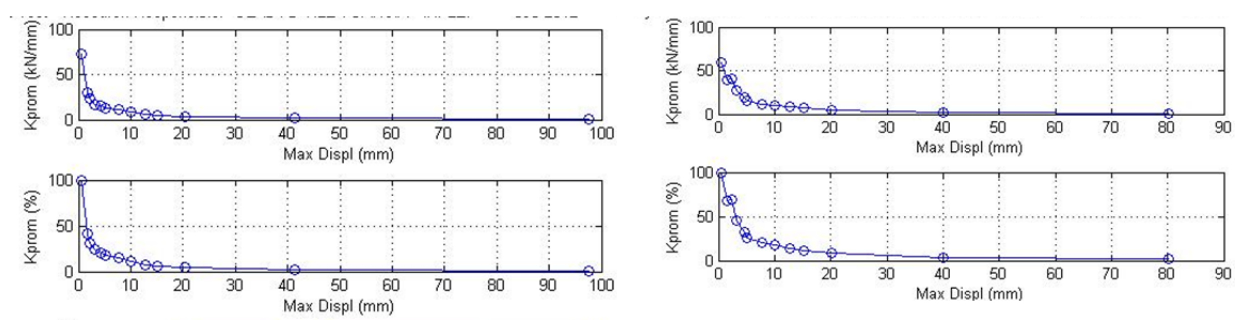

Figure 10: Average stiffness loss in: a) W1; b) W2

The force – displacement envelope curve is presented in Figure 11. For each wall it has been obtained with the maximum values of top displacement D1 and lateral load V for each step. According to the Peruvian Seismic Code [6], the limit of lateral displacement for masonry is 12.5mm (correspond to a drift of 1/200=0.005) in order to be economically repaired after a severe earthquake. It may be observed that the traditional wall W1 has a very short elastic range and a strong strength degradation before it reaches the code limit drift of 0.005, as compared to confined masonry wall W2.

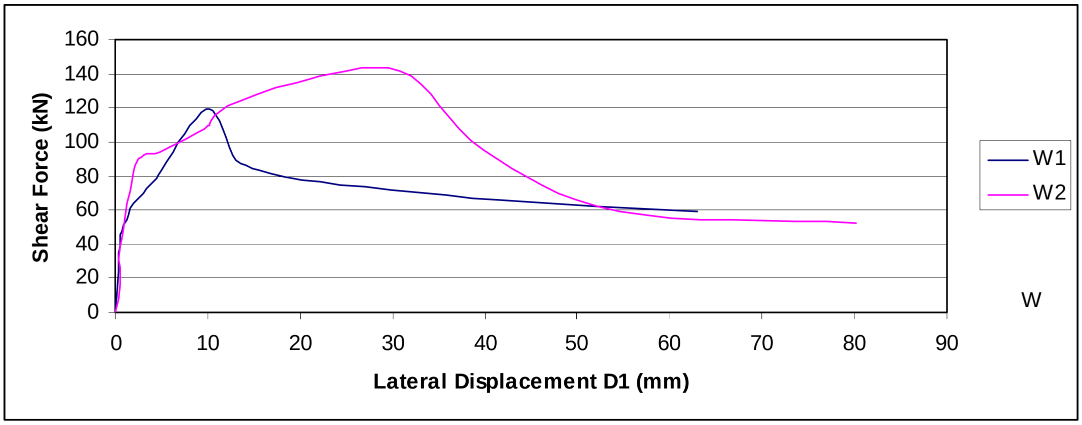

Figure 11: Envelope lateral load-displacement curve for W1 and W2.

CONCLUSIONS

It has been possible to reproduce the inadequate seismic behaviour of Haitian CMU and masonry construction quality with low cement content concrete for units and structural elements.

Infilled wall W1 under the cyclic load test has separated almost immediately from the concrete frame, thus leaving the wall free to collapse by out-of-plane forces. On the other hand, confined masonry wall W2 has maintained the integrity between the wall and the columns, which enables the system to develop shear crack patterns.

Wall W2 has resisted 15% more horizontal load than wall W1, which is not so relevant. The most important contribution of wall W2 lies in the fact that without much extra costs, wall capacity to support perpendicular forces once it is cracked has been notably increased. This is a first and very important recommendation for Haitian reconstruction which involves mostly hand labour and masons training.

Fragility of CMU accounts for final crushing at the centre of the wall. This suggests that something has to be done to limit the crushing of these fragile units, such as the filling of the voids of the units or a complete change, such as the use of solid units.

The overall results indicate that confined masonry is much better than the traditional RC frame system, and could be an alternative to increase safety at minimum cost. However, more research has to be done to provide an effective and cheap solution.

ACKNOWLEDGEMENTS

The authors recognize the help of the Structures Laboratory staff in the development of the experimental work.

REFERENCES

1. EERI (2010) "The M_w 7.0 Haiti Earthquake of January 12, 2010: Report #1", EERI Special Earthquake Report – April 2010, in http://www.eeri.org/site/images/eeri_newsletter/2010_pdf/Haiti_Rpt_1.pdf
2. EERI (2010) "The M_w 7.0 Haiti Earthquake of January 12, 2010: Report #2", EERI Special Earthquake Report – May 2010, in http://www.eeri.org/site/images/eeri_newsletter/2010_pdf/Haiti_Rpt_2.pdf
3. San Bartolome A., Quiun D, and Silva W. (2011) "Design and Construction of Seismic Resistant Masonry Structures", Fondo Editorial PUCP, Lima, Perú ("Diseño y Construcción de Estructuras Sismorresistentes de albañilería" in Spanish).
4. San Bartolomé A. and Quiun D. (2007) "Design Proposal of Confined Masonry Buildings", 10th North American Masonry Conference, St. Louis, Missouri.
5. SENCICO (2006) "Norma Técnica E 070 Albañilería", Lima, Perù (in Spanish)
6. SENCICO (2006) "Norma E.030 Diseño Sismorrresistente", Lima, Perú (in Spanish).