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WIRE-MESH AND MORTAR CONFINED MASONRY AS SEISMIC RESISTANT SYSTEM FOR HOUSES UP TO TWO STORIES

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Abstract

This research aims to evaluate the effectiveness of a wire mesh and mortar confined masonry system in resisting earthquake loading for the specific case of one and two story houses. A full-scale prototype was tested at the Housing Research Centre Structural Laboratory of the National Polytechnic School in Quito, Ecuador. The prototype used for the investigation was two story height (h=4.50 m) and approximately in plane 3.7 m by 2.50 m. It included a door and windows, as in a real house. To form the structural system, a 3 cm thick mortar layer ($f´c$=10 MPa) reinforced with the thinnest wire mesh available in the market (φ3.5 mm @ 150x150 mm), was placed only in the inner face of the walls. In addition, the masonry used in the experiment was built following an artisan process to reassemble typical informal construction conditions. The reinforcement was placed only in the inner face, which represents the extreme case condition to be tested. The prototype proved to be highly effective resisting a lateral force equivalent to the full self-weight, that is V=100%W with R=1, and yet the capacity of the system was not reached. Considering that 70% of Ecuadorian construction of 1-3 story buildings has been made without the assistance of a structural engineer and that Ecuador is located in a high seismic risk area, this proposed system provides an effective solution to improve the seismic performance of both new and existing informal construction for housing programs up to two stories.

Keywords: externally reinforced masonry; shear walls; social housing program

1. Introduction

Ecuador is located in the Pacific Ring of Fire, a region where a large number of earthquakes occur every year. However, according to official statistics [1], 73% of buildings do not satisfy basic earthquake security requirements and are categorized as informal construction. Around 98% of residential buildings range from one to three stories and are composed of reinforced concrete frames with partitions made out of hollow concrete blocks. This is a common practice that covers around 54% of the total buildings in Ecuador. Therefore, this investigation aims to assess the earthquake performance of a special masonry reinforced system conceived under the premises of construction convenience, moderate costs and real material properties produced in Ecuador. The application of the system is valid for up to two story residential buildings and can also be implemented in structural retrofitting To validate the system, a prototype was constructed in the Housing Research Centre Structural Laboratory of the National Polytechnic School.

2. Proposed system

Ordinary and special reinforced concrete moment frames are the exclusive structural systems used in Ecuador for low-rise residential buildings. For instance, 94% of them use reinforced concrete as the structural material [2]. However, during recent major events, such as the 2010 Maule earthquake, buildings composed of reinforced concrete bearing walls have shown a satisfactory performance under large lateral seismic forces. . Since the use of concrete hollow blocks for partitions is a common practice in the Ecuadorian construction industry, this research sought to combine the advantages of the bearing wall systems without disrupting the Ecuadorian construction idiosyncrasy.

The system consists of reinforcing traditional hollow concrete masonry with an inner, outer or both faces of a 3 cm thick layer of mortar with specified compressive strength of $f'c$=10 MPa and the thinnest wire mesh available in the market (φ3.5 mm @150x150 mm) in replacement of reinforced concrete columns and beams. The mortar layer is intended to resemble the masonry plaster but with a somehow different composition, by adding coarse aggregate of 10 mm maximum size. The connection between the masonry and the wire mesh was done through 4 mm-diameter steel bars that were set in the mortar paste between block rows. On the other hand, the slab was connected to the masonry by adding 8 mm-diameter steel bars of 1m length. Fig.1 & 2 show the configuration of the system.

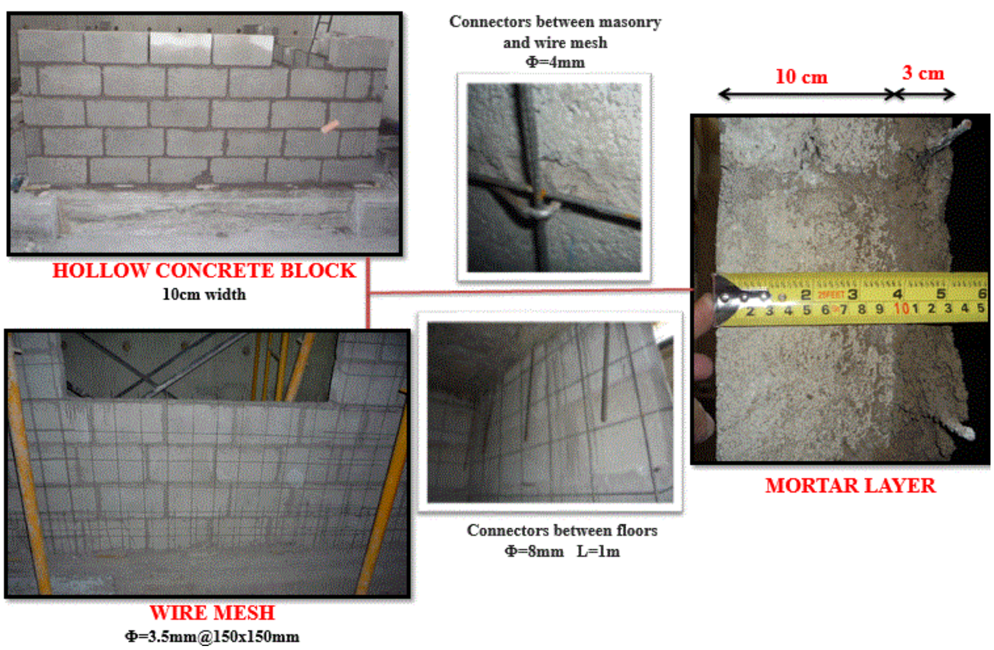


Fig. 1 – Proposed system configuration

Fig. 2 – Connectors installation process between masonry and wire mesh

3. Prototype

An investigation was held to test the proposed system [3] between April and October, 2010, at the Housing Research Centre Structural Laboratory of the National Polytechnic School. The geometry and actual prototype are shown in Fig.3. and Fig.4, respectively.

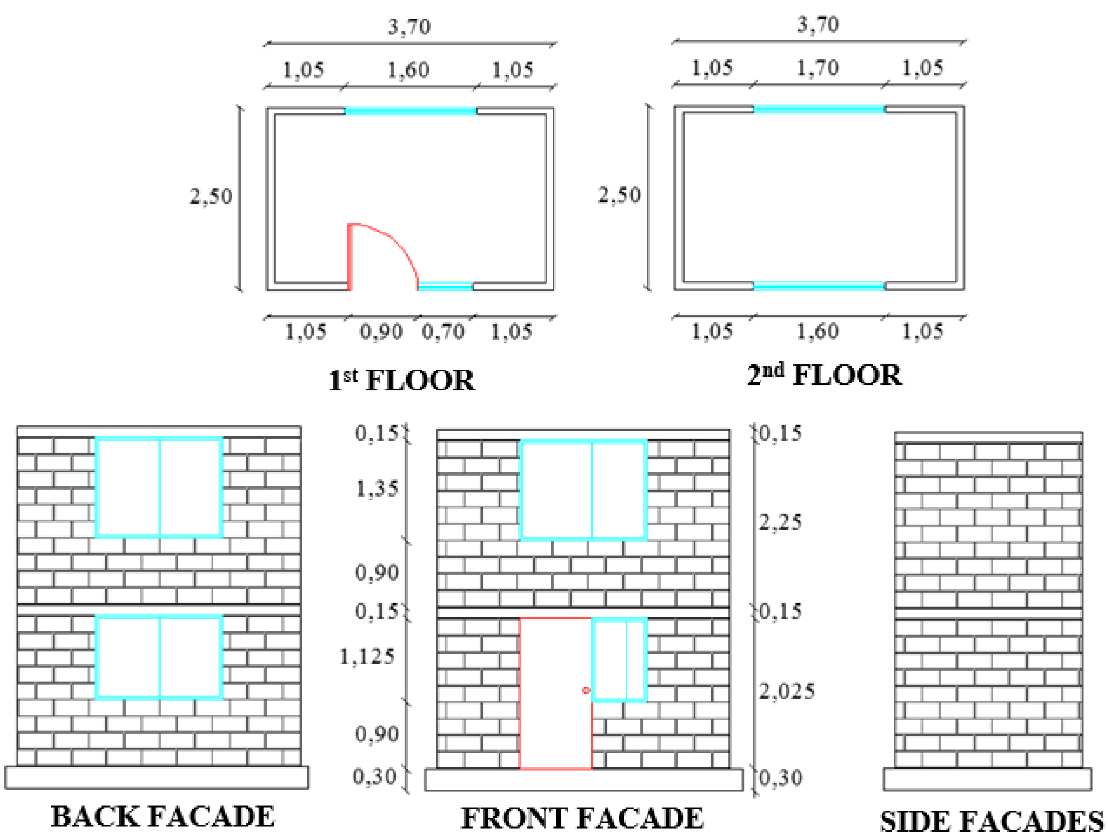


Fig. 3 – Geometry of the prototype (in meters)

Fig. 4 – Prototype

The construction of the housing prototype involved the following steps:

- Step1: Foundation beams. 30x30 cm foundation beams were constructed along the perimeter of the prototype;

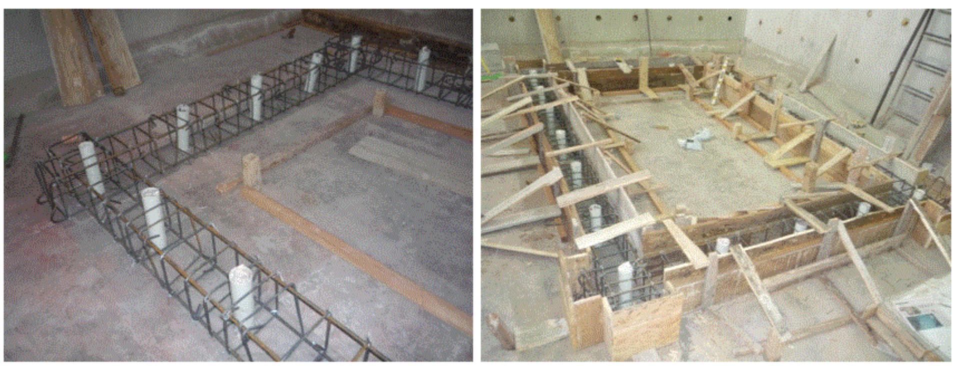


Fig. 5 – Foundation beams

To avoid sliding of the prototype during lateral loading, 20 anchor bolts were placed along the perimeter of the foundation beam. The position of the bolts match existing holes staggered on the surface of the laboratory floor. Each anchor was pretensioned to 100 kN approximately.

- Step 2: 1st floor walls. The masonry of the 1st floor was completed taking care of leaving the connectors (Fig. 2) each three block rows.

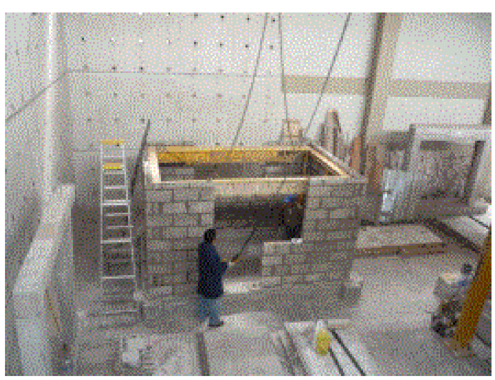


Fig. 6 – 1st floor walls

- Step 3: 1st slab. The 1st slab was build using typical Ecuadorian practice: waffle slab. The connection between the walls and the slab was done through 8 mm-diameter steel bars of 1m length. The beams located in the central region of Fig.7 were built specifically for the experiment in order to transfer the lateral load from the actuators to the walls. They are not necessary in a real project.

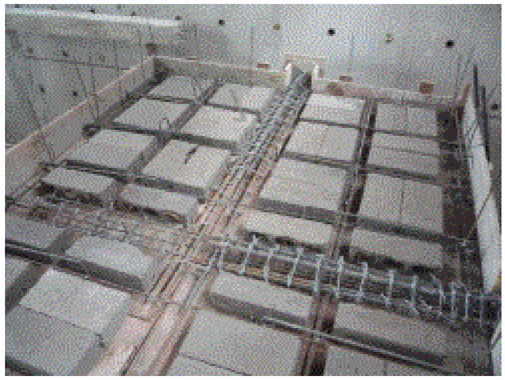


Fig. 7 – 1st slab

- Step 4: 2nd floor masonry walls and top slab. The process illustrated in the previous steps was repeated for the second floor.

Fig. 8 – 2nd floor walls

- Step 4: installation of wire mesh. By using the connectors left in the walls, the wire mesh was anchored to the masonry.

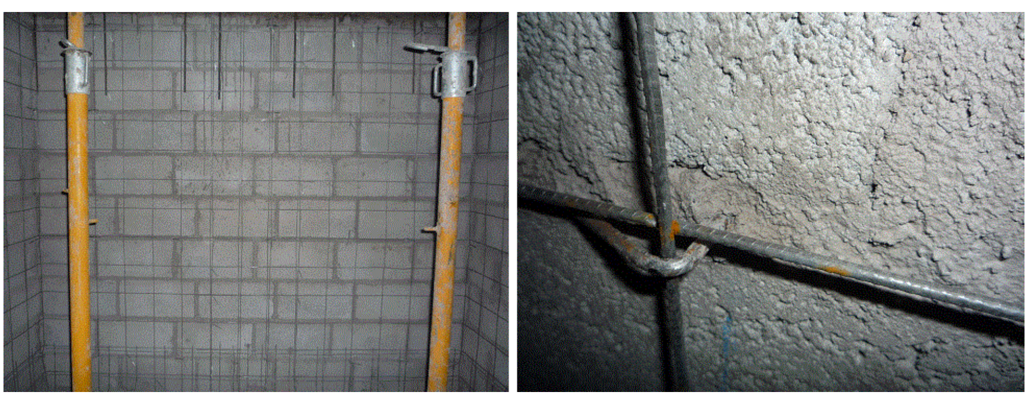


Fig. 9 – Wire mesh installation

- Step 5: mortar fill. A 3-cm layer was placed on the interior face of the walls.

4. Mechanical properties of the materials

A series of tests were performed to determine the mechanical properties of the materials. The walls were classified, according to the Ecuadorian standard enforced at that time, as type D: “exterior divisor walls with or without lining”, requiring a minimum compressive strength of 2.5 MPa [4].

Fig. 10 – Block compression test

Table 1 – Block mechanical properties

	S1	**S2**	**S3**
f´m (MPa)	1.00	1.40	0.95

The concrete used in the foundation beams, slabs and wall lining was tested according to ASTM procedure [5].

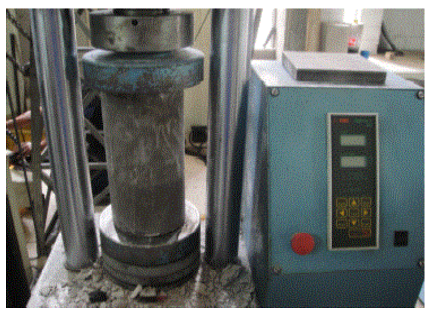


Fig. 11 – Compression test of concrete

Table 2 –Concrete properties of foundation beams, slabs and wall

	Foundation Beams	**1st slab**	**2nd slab**	**1st floor wall**	**2nd floor wall**
f´c (MPa)	21.45	15.29	17.52	10.88	10.73

The mortar used to paste each block row was also tested according to the ASTM procedure [6].

Fig. 12 – Compression test of mortar

Table 3 – Masonry mortar

	S1	S2	S3
f´c (MPa)	9.61	7.77	9.20

4. Design of the experiment

4.1 Loads

4.1.1. Vertical Load

The total dead weight of the prototype was calculated to be 136 kN.

4.1.2. Horizontal Load

At the time when this investigation was held the valid Ecuadorian code was CPE INEN 005-1 [7]; however, at the beginning of 2015 a new Ecuadorian code was released and enforced [8]. The horizontal load is calculated with both references and contrasted for comparison. According to CPE INEN 005-1 [7], the lateral base shear is calculated with equations (1), (2) and (3).

$$V = \frac{Z \cdot I \cdot C}{R \cdot \phi_p \cdot \phi_e} \cdot W \tag{1}$$

$$C = \frac{1{,}25 \cdot S^s}{T} \tag{2}$$

$$T = Ct \cdot hn^{3/4} \tag{3}$$

Where,

Z= zone factor. Represents the maximum expected rock acceleration for the design earthquake. In Quito, *Z*=0.4;

I= importance factor. The structure is classified as “other structures”, so I=1.0;

S= depends on the soil. It was selected “soft soils and deep strata”, so *S*=1.50;

Ct= 0.06 (“other structures”);

hn= 4.90 m;

R= reduction factor. *R*=5 for “confined walls”. Instead, a factor of *R*=3 was used. The use of *R*=3 implies a system with very limited ductility. This was later adopted in NEC-SE-DS [8].

After doing the previous calculation, equation (3) yields a period of *T*=0.1976 s and equation (2) is equal to 11.62. This value cannot be larger than 2.8, so *C*=2.8. The shear base is then computed as *V*= 0.37W= 50.32 kN.

According to the current code, NEC-SE-DS [8], the calculations are slightly different:

$$V = \frac{I \cdot S_a}{R \cdot \varphi_p \cdot \varphi_e} \tag{4}$$

$$Sa = z \cdot \eta \cdot F_a \tag{5}$$

Where,

η= 2.48, for Sierra provinces;

Fa= 1.20, for seismic zone V and Soil Type C;

The other factors are the same as in the previous code. Therefore, *V*= 0.40W= 54.40 kN. Considering *R*=1 (no ductility reduction), *V*= 1.12W= 152 kN for the previous code and *V*= 1.19W= 161.84 kN for the current one.

The lateral load was applied by two hydraulic actuators connected to the middle and top slab of the prototype in the two orthogonal directions, as shown in Fig. 13.

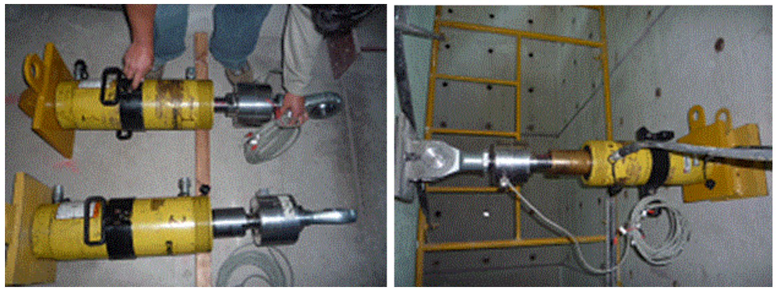


Fig. 13 – Hydraulic actuators used to apply lateral load

The lateral displacement was measured using a series of LVDTs, located as shown in the Fig. 14.

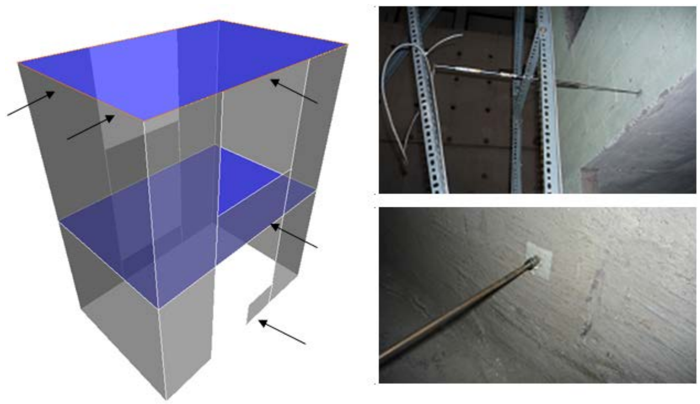


Fig. 14 – Location of the LVDTs in the long direction

4.2. Shear capacity of the structural system

Eq. (6) was used to calculate the shear capacity of the system.

$$Vn = A_{cv}(0.53\sqrt{f'c} + \rho_s \cdot F_y) \tag{6}$$

Where,

Acv: shear resistance area, cm^2;

f´c= 10 MPa (see Table 2);

ρs= As/(bd)= 0.002133;

Fy= 550 MPa;

φ=0.85

In order to consider the influence of the door and windows openings in the long direction, a simple finite element model was constructed and a unitary load was applied at both levels. The reduction factor was then estimated based on the ratio between the displacement of the model with openings and the model without openings, as depicted in Fig. 13. Table 4 shows the results of the reduction factors.

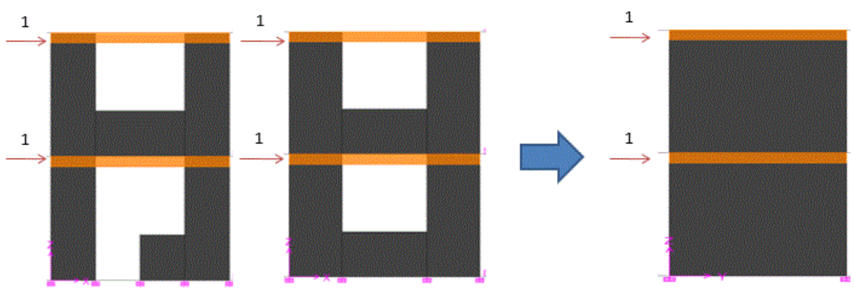


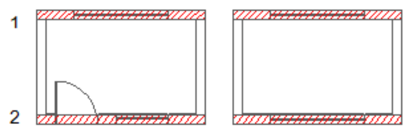
Fig. 15 – Determination of the reduction factor due to openings

Table 4 – Reduction factor due to door and windows openings

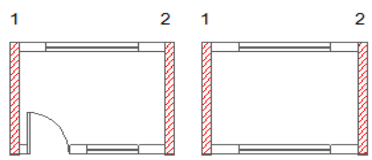
		Δ With openings (mm)	*Δ* Without openings (mm)	factor	*media*
Front Wall	1st floor	0.29116	0.15134	0.52	***0.53***
	2nd floor	0.70099	0.37422	0.53	
Back Wall	1st floor	0.23938	0.15134	0.63	***0.63***
	2nd floor	0.60210	0.37422	0.62	

Using the reduction factors from Table 4 the shear capacity of the structural system was calculated with equation (6). The results are shown in Table 5.

Table 5 – Shear resistance of the structural system



X-X	***b*** **(cm)**	***h*** **(cm)**	***Acv*** **(cm²)**	***Vn*** **(kN)**	***φVn*** **(kN)**
1	3	350	1050	94.46	80.31
2	3	350	1050	112.29	95.45
					175.76



Y-Y	***b*** **(cm)**	***h*** **(cm)**	***Acv*** **(cm²)**	***Vn*** **(kN)**	***φVn*** **(kN)**
1	3	230	690	117.08	99.64
2	3	230	690	117.08	99.64
					199.28

The overturning moment capacity of the system is equivalent to a lateral force of 132.82 kN and 153.15 kN applied in the short and long direction, respectively. These values are similar to a seismic load considering a factor R=1, therefore the upper boundary loading condition was guaranteed by the global stability of the system.. Since the shear capacity of the prototype ranged between 175 and 200 kN, a failure mechanism different than shear was expected to occur. Based on this information, the lateral load was applied in cycles of 30, 60, 90, 120, 140 and 160 kN, as depicted in Fig. 16 the short direction walls.

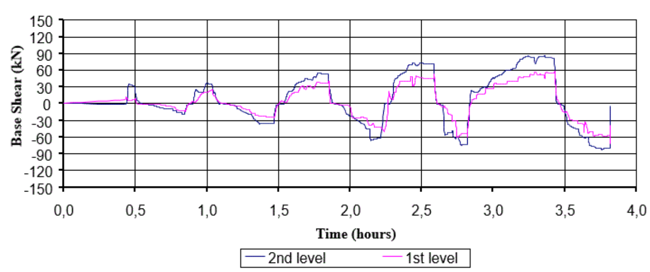


Fig. 16– Cyclic loading in the short direction walls

The long direction walls were loaded in several cycles within a 5 day period in order to perform a detailed evaluation of the progressive stiffness degradation. The first day a complete 30kN cycle was imposed. The second day 30 kN and 60 kN cycles were imposed, until a total lateral load of 75 kN. The third day 60 kN and 90 kN cycles were given to the prototype. The fourth day a 110 kN was completed. The last day a maximum of 170 kN was imposed. The test was stopped because of the evident degradation of the prototype.

5. Results

Fig. 17 and 19 show the hysteretic behavior of the walls in both the short and long direction. The short direction walls have no openings whereas the long direction walls have door and windows openings.

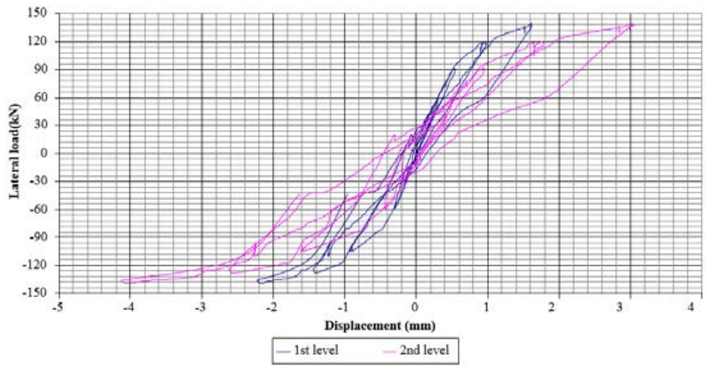


Fig. 17– Hysteretic behavior of the short direction walls

Fig. 18– Right: block cracking in the long direction due to loading in the short direction (100 kN cycle); left: walls in the short direction have no cracking (140 kN cycle).

In the short direction, the test was stopped at 140 kN because the anchorage to the foundation beams (wire mesh) broke, as predicted by the limited overturning capacity of the system (132.82 kN). Even though the walls were expected to have a capacity reserve (until 200 kN), the hysteretic diagram show an slightly stiffness deterioration of the system, especially in the second floor. The calculated lateral seismic load was between 152 – 162 kN with *R*=1, so the walls resisted, without visible cracking, between 86-90% of the base shear, which is equivalent to 1.2 times the total weight of the prototype The walls behaved almost completely elastic.

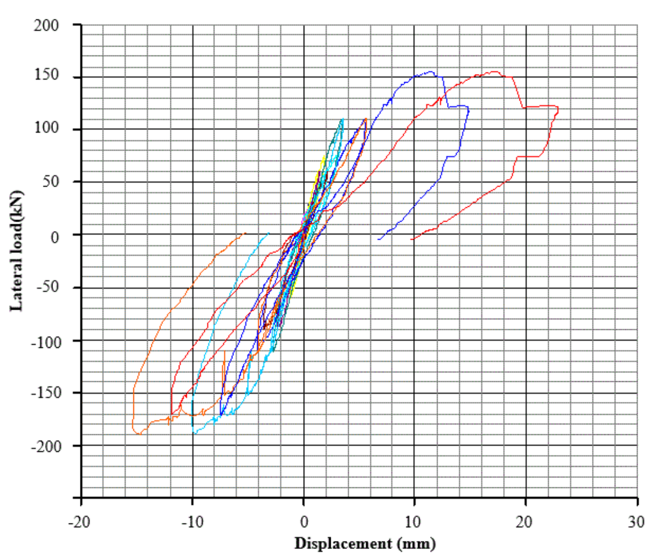


Fig. 19– Hysteretic behavior of the long direction walls

The lateral seismic load with *R*=1 was calculated between 152-162 kN. The maximum overturning capacity was calculated to be 153.15 kN. The shear capacity of the system, considering the reduction due to door and windows openings, was calculated as 175.76 kN. Fig 20, 21 and 22 show the cracking of the walls for each loading stage.

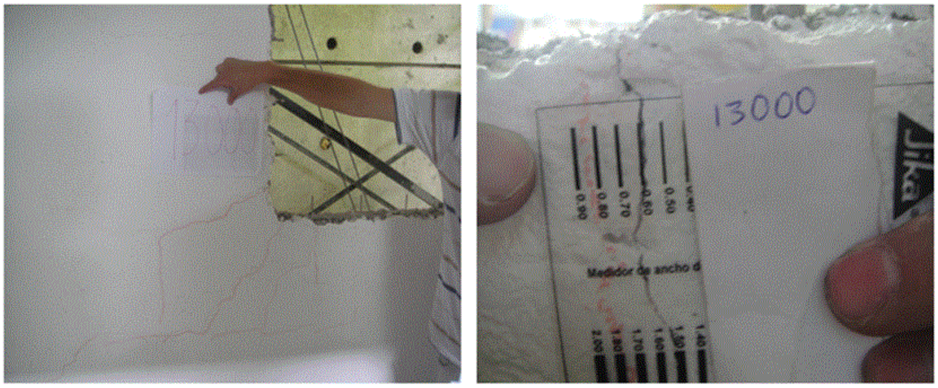


Fig. 20– Wall damage at 130 kN. Crack widths of around 0.60mm.

Fig. 21– Short direction blocks in the long direction test at 150 kN. Tensile cracking due to exceedance of the overturning capacity is evident in the connection with the foundation beams.

Fig. 22– Cracking at the last cycle (185 kN). Left: wire mesh rupture at the window opening proximity; right: cracks width of around 1.60mm.

The predicted behavior limits for the test were reviewed. It is worth to mention that the system resisted a lateral load equivalent to its total weight. The prediction of the shear wall capacity, 175 kN, is congruent with the observed behavior.

5. Conclusions

A structural system consisting of reinforced traditional hollow concrete masonry blocks with a thin layer of low strength concrete and wire mesh was tested under seismic lateral loading. The prototype resembled typical informal construction conditions by using low quality materials, yet a common practice in Ecuadorian informal construction. The worst scenario was tested by reinforcing only the interior face of the masonry; however, in a real project both faces should be reinforced in order to confine the walls. In such case, the lateral resistance will increase by a factor of two.

Even in the abovementioned conditions the system resisted the full calculated lateral load with *R*=1, that is, more than the total weight of the prototype as lateral load (112-120%). The short direction walls (without openings) remained uncracked until the end of the test, which was stopped because the connection of the wall to the foundation beams started failing, due to the wall *H/L* ratio. The long direction walls (with openings) started to show cracks of 0.6 mm width when reaching the full weight as lateral load (130 kN), but continued supporting lateral load without stiffness degradation. The system resisted a maximum load of 185 kN. At this point, the wire mesh in the proximities of the windows broke and there was a considerable stiffness degradation, as show in the hysteretic diagram.

The methodology used to obtain the reduction factors to consider the door and windows openings in the long direction walls proved to be accurate, since the shear capacity (175 kN) was almost the limit of the test. Door and windows corners should be reinforced with additional bars to avoid cracking. There was not apparent failure of the connection between the masonry and the reinforced mortar layer, so the connectors performed well. The system is believed to be useful not only in the construction of new one to two stories houses but also for retrofitting existing ones. The new Ecuadorian Code, NEC-15 [8], includes the use of externally reinforced masonry using the system proposed in this investigation with a reduction factor of *R*=1.5.

The tested system has been proposed to Ecuadorian authorities as an alternative to retrofit low-rise structures affected by the recent earthquake occurred in Ecuador (M7.8, Abril 16th), as well as for construction of new buildings up to two stories.

6. Acknowledgements

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