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EXPERIMENTAL STUDY ON RETROFIT TECHNOLOGIES FOR RC FRAMES WITH INFILLED BRICK MASONRY WALLS IN DEVELOPING COUNTRIES

Matsutaro SEKI[1], Viorel POPA[2], Eugen LOZINCA[3], Andreea DUTU[4], Andrei PAPURCU[5]

ABSTRACT

The developing countries in the earthquake prone regions in the world are still suffering a lot of casualties as well as building damage. These damages might be caused by inadequate structural design by engineers and/or poor quality control of construction works. In order to contribute to disaster mitigation for existing reinforced concrete (RC) buildings in developing countries, economic and easy to apply retrofit methods should be developed based on the local materials and the construction technologies.
For this reason, a project was conducted in Technical University of Civil Engineering of Bucharest, Romania (UTCB) aiming to find a solution that can be easily applied in the developing countries and with as low cost as possible. Two retrofit solutions were considered: one with timber framed masonry panels input inside the RC frame, connected just with mortar to the RC frame and the other using two layers of wire mesh applied on a brick masonry infill.
Five specimens were tested: one bare RC frame, one RC frame with timber cushion and brick masonry infill, one RC frame with brick masonry infill, one RC frame with timber framed masonry infill and one RC frame with brick masonry infill strengthened by two layers of wire mesh and mortar (ferrocement), respectively. The test specimens are a half-scaled in order to fit in the reaction frame. The concrete compressive strength is about 14 MPa, and masonry prism strength of is about 11 MPa. These values are quite low aiming to simulate the low material quality in developing countries. The cyclic loading test was performed and the ultimate failure mode, maximum strength, maximum horizontal drift and other behavior characteristics were obtained and are discussed in this paper.

Keywords: developing countries; RC infill wall frame; retrofit solutions; timber framed masonry panel; ferrocement

1. INTRODUCTION

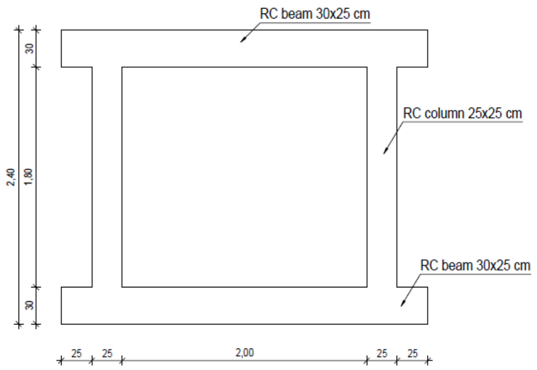
Weak reinforced concrete frames in developing countries are highly vulnerable to severe earthquakes. The concrete quality is poor, with average compressive strength of 8-10 MPa, and the reinforcement amount and details are not complying with the minimum requirements for structural members in seismic structures. Application of effective retrofit methods for concrete frames (for example, infilled steel braced frames) is rather difficult. Steel anchors necessary to connect the infilled panels to the surrounding frame can be hardly installed in low concrete quality. In this respect, an experimental research program was conducted at the Technical University of Civil Engineering of Bucharest to find cost effective retrofitting solutions that can be easily applied for weak concrete frames in developing countries. Two retrofitting methods based on infilled masonry were considered. In the first solution, infilled timber framed masonry, with two different layouts, connected just with lime-cement mortar to

[1]Visiting research fellow, Building Research Institute, Tsukuba, Japan, sekimatsutaro@yahoo.co.jp
[2]Associate Professor, Technical University of Civil Engineering Bucharest, Romania, viorel.popa@utcb.ro
[3]Lecturer, Technical University of Civil Engineering Bucharest, Romania, elozinca@utcb.com
[4]Lecturer, Technical University of Civil Engineering Bucharest, Romania, andreea.dutu23@gmail.com
[5]Assistant Professor, Technical University of Civil Engineering Bucharest, Romania, andrei.papurcu@utcb.ro

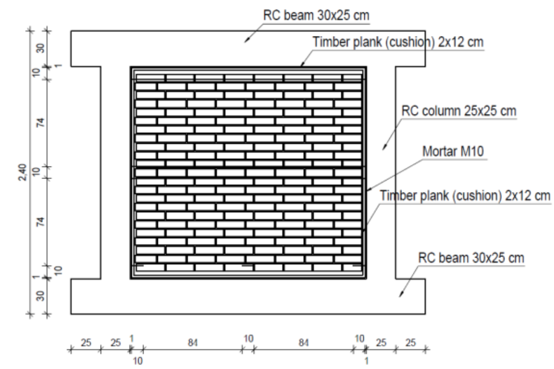
the surrounding frame was used. In the second solution, masonry panels jacketed with wire mesh reinforced mortar were used as infilled panels.

2. DESCRIPTION OF TEST SPECIMENS

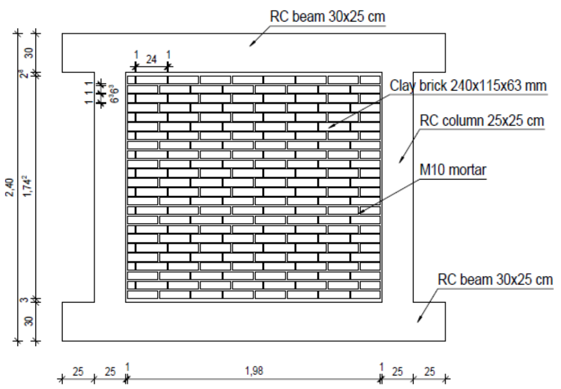
The testing program included five specimens. Specimens details are presented in Figure 1.



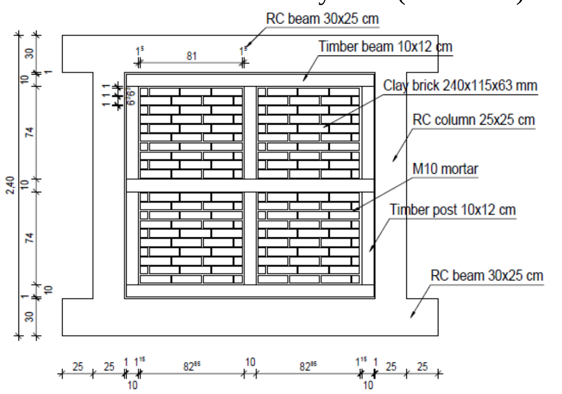
a. Reinforced concrete (RC) frame (S1-F)



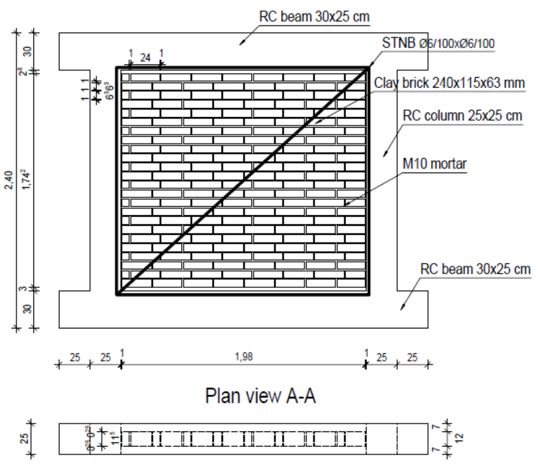
b. Bare reinforced concrete (RC) frame with timber cushion and masonry infill (S2-FTCM)



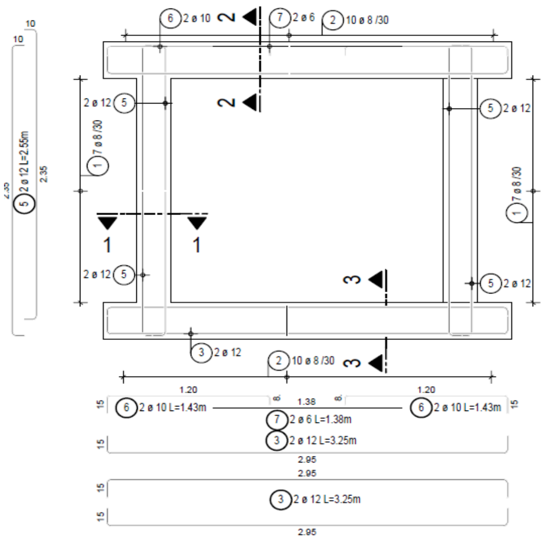
c. Reinforced concrete (RC) frame with brick masonry infill (S3-FM)



d. Reinforced concrete (RC) frame with timber framed masonry (brick) infill (S4-FTM)



e. Reinforced concrete (RC) frame with brick masonry infill and wire mesh (two layers) applied on all specimen (S5-FMFc)



f. Reinforcement of the RC frame

Figure 1. Test specimens' dimensions and layout and reinforcement of RC frame

2.1 Reinforced concrete (RC) frame (S1-F)

The first specimen was a weak bare concrete frame. The rectangular columns (25x25cm) were reinforced with 4ϕ12 mm longitudinal rebars and ϕ8 mm stirrups spaced at 30 cm. The reinforcement details for all concrete frames are presented in Figure 1-f. The reinforcement ratio was 0.6% in case of the columns and 0.3% in case of the beams. These reinforcement details were chosen to reproduce the current practice for concrete frames in developing countries.

2.2 Reinforced concrete (RC) frame with timber cushion and masonry (brick) infill (S2-FTCM)

The second specimen was retrofitting using a timber framed masonry panel. The masonry infill was made with burned clay bricks, available on the Romanian market with the dimensions of 240x115x63 mm. This infill detail was considered to partially transfer the masonry strut compression force to the surrounding concrete frame. The timber planks were considered to act as a cushion, based on the compression perpendicular to grain property of the timber. Between the timber plank and the concrete frame, a mortar layer of approximately 1 cm thickness was introduced. The same layer was introduced between the timber planks and the masonry, on all edges, except the upper part (Figure 2). The timber was Romanian softwood, fir.

Figure 2. RC frame with timber cushion and masonry infill

Figure 3. RC frame with masonry infill

2.3 Reinforced concrete (RC) frame with masonry (brick) infill (S3-FM)

In case of specimen S3-FM, a plain masonry infill panel was considered. The masonry infill was made with the same clay bricks as in the case of S2-FTCM. The layout of the specimen is shown in Figure 3. There were concerns that, given the weak concrete frame, the diagonal compression force in the masonry panel can severely damage the concrete frame at small lateral displacement. Similar masonry panels retrofitted using various techniques were previously tested by the authors (Seki, 2016, Popa 2010).

2.4 Reinforced concrete (RC) frame with timber framed masonry (brick) infill (S4-FTM)

In case of specimen S4-FTM, a timber framed masonry panel was used. The infill panel was made using a timber frame, with cross-halved timber connections (Figure 4), and masonry, with cement based mortar. Common construction materials, readily available, were used. The dimensions of the timber framed masonry panels were chosen based on previous experimental study (Dutu et. al, 2015) in Tokyo Institute of Technology (Figure 5).

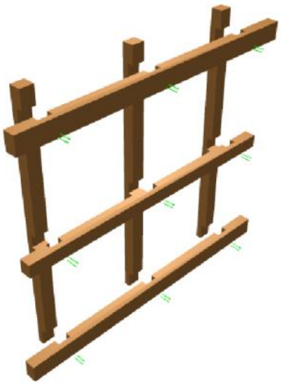


Figure 4. The timber frame with cross-halved connections and screws (Dutu et. al, 2015)

Figure 5. Previous experimental test in Tokyo Institute of Technology (Dutu et. al, 2015)

Some of the connections had undesirable construction gaps, as shown in Figure 6. This reflects the reality of the current construction practice in developing countries. The connection between the timber frame and concrete frame was made using lime-cement mortar, manually introduced in the gap (Figure 7). Properties of the mortar are given in the subsequent paragraph. The timber was Romanian softwood, fir.

Figure 6. Connection execution error – existing big gap

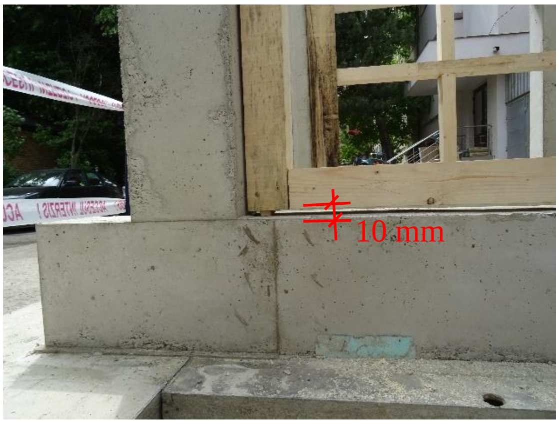


Figure 7. Gap between the RC frame and timber frame, filled with finishing mortar

2.5 Reinforced concrete (RC) frame with brick masonry infill and wire mesh (two layers) applied on all specimen (S5-FMFC)

Specimen S5-FMFC was a concrete frame retrofitted using a masonry panel jacketed with steel wire mesh reinforced mortar. Wire mesh with ϕ0.9mm wires spaced at 13 mm on both directions was used. The wire mesh was fixed on the concrete elements with nails (Figure 8-a and b) and on the masonry wall with small clamps (Figure 8-c and d). Wire meshes were 25 cm overlapped (Figure 8-f). Initially, two 1,0 m wide meshes were applied on the masonry panel overlapped on the horizontal direction (Figure 8-g). Then a wire mesh was installed on each RC beam overlapped with the masonry meshes on 25 cm (Figure 8-g). In the third step, two 1m wide meshes were applied on the columns, overlapped on the horizontal direction (same as on the masonry panel) and overlapped on the masonry panel on 25 cm. The mesh was bent over the column and between the column and the masonry with a wooden batten (Figure 8-h). The fourth step was to apply a second layer of mesh on the masonry panel, but this time, the overlapping (also on 25 cm) was made on vertical direction, and three pieces of wire mesh were used (two being 1 m wide each and another being 0.5 m wide) (Figure 8-i). Between the mesh and the infill or RC elements, a gap of maximum 1 cm was left, so the wire mesh could be well embedded in the finishing mortar.

a – nails used to position the wire mesh on the concrete (mesh was at 1 cm distance from the concrete surface)

b – mesh applied on the concrete surface and also on the masonry

c – the clamps used to position the wire mesh on the masonry panel, embedded in the mortar joints

d – the clamps applied to position the wire mesh on the masonry panel, embedded in the mortar joints

e – the tie wire used to connect the wire meshes on both sides of the masonry panel

f – overlapping of the wire mesh

g – overlapping wire mesh from RC beam on the masonry

h – bending the wire mesh to follow the shape of the column with a wooden batten

i – all wire meshes are applied

j – the mortar finishing is applied by hand throwing

k –starter layers applied for reference thickness of the mortar layer

l – finished specimen

Figure 8. Execution of the specimen 5 (FMFC)

3. MATERIALS TESTS

3.1 Bricks

All the materials used were bought from Romania. Solid burnt clay bricks (240 x 115 x 63 mm) were used. Results of the compression tests on bricks are given in Table 1.

Table 1. Compression test on bricks

Prism specimen	**Area (mm^2)**	**Testing speed (mm/min)**	**Max strength (MPa)**	**Max strength average (MPa)**
1			37.32	
2	27600	5	50.18	**41.49**
3			36.96	

3.2 Mortar

General masonry lime-cement mortar was used. This is readily mixed dry mortar delivered in paper bags. Results of the material tests for mortar show an average compression strength of 6.7 MPa. Some difference in mortar characteristics, for masonry joints and for finishing or timber-concrete, was noticed, such as for the latter it was around 4.7 MPa, while the first ranged from 5 MPa to 9 MPa.

3.3 Masonry prisms

Compression tests were also conducted on masonry prisms. The results are presented in Table 2.

Table 2 - Compression test on masonry prism specimens

Prism specimen	**Area (mm^2)**	**Max strength (MPa)**	**Max strength average (MPa)**
1		9.49	
2		11.27	**9.78**
3	27600	8.59	
1		12.72	**13.51**
2		14.31	

3.4 Concrete cylinders

The results of concrete cylinder (diameter 100 mm, height 200 mm) tests showed an average compression strength of 14 MPa.

3.4 Reinforcement for concrete

The results of rebars tensile tests are presented in Table 3.

Table 3. Tensile test on reinforcement bars (rebars)

No.	Diameter [mm]	Type	Yield. force (kN)	Average yield strength (MPa)	Max force (kN)	Average strength (MPa)
1			10.5		12.00	
2	6		11	379.76	12.50	432.14
3		OB 37	10.4		11.80	
1			18.5		21.50	
2	8		18	364.67	21.50	429.33
3			18.2		21.40	
1			35.5		43.50	
2	10		36	453.59	43.50	552.74
3		PC 52	36		44.00	
1			49		59.50	
2	12		48	428.61	59.30	525.96
3			48.3		59.50	

3.5 Compression perpendicular to grain on timber

Compression tests were conducted on timber plank pieces and prisms. Test results are shown in Table 4.

Table 4. Compression tests on timber prisms (1-3) and on planks (4-5) were made perpendicular to the grain

Specimen	Max Strength [MPa]	σ_y [N/mm^2]	ε_y	E_1[MPa]	E_2 [MPa]	Max. force [kN]
1	3.80	3.62	0.035	113	2.28	37.39
2	4.27	4	0.026	207	3.27	41.07
3	5.06	3.63	0.023	206	14.08	49.88
Average	4.38	3.75	0.028	175.24	6.54	42.78
4	5.20	2.92	0.044	105	14.74	69.03
5	4.84	2.98	0.053	101	11.31	64.47
Average	5.02	2.95	0.05	102.67	13.03	66.75

4. TEST SETUP

The loading protocol used was according to Figure 9. Quasi-static cyclic static lateral loading under constant axial load was applied using the reaction frame installed at the Structural Testing Laboratory of the Seismic Risk Assessment Center (CERS) in UTCB. This testing equipment (Figure 10) was donated by the Japanese International Cooperation Agency during the seismic risk reduction project in Romania (2002-2007) The applied axial load ratio was 0.4 (considering the average concrete compressive strength). For each specimen, three concrete samples cylinders were tested in compression in the test day.

The vertical load was applied by vertical jack. The horizontal load was measured with two load cells.

The measurements were made using displacement transducers, with 2 and 5 cm strokes. Their position is represented in Figure 10, where: H - horizontal displacement of the steel loading beam (mm), HS - horizontal displacement (mm), HF - horizontal displacement (mm), VS - vertical settlement on the left side (mm); VD - vertical settlement on the right side (mm); LS - sliding at the top (between specimen and steel beam) (mm); LJ - sliding at the bottom (between steel slab and specimen) (mm);

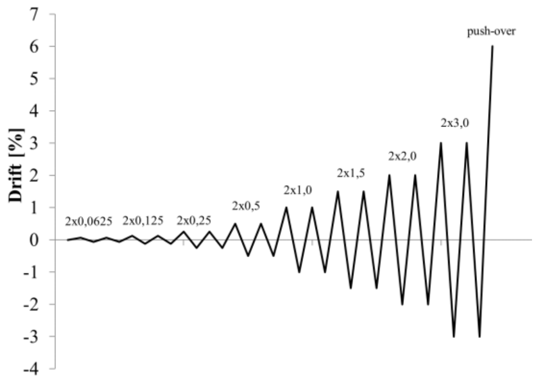


Figure 9. Loading protocol

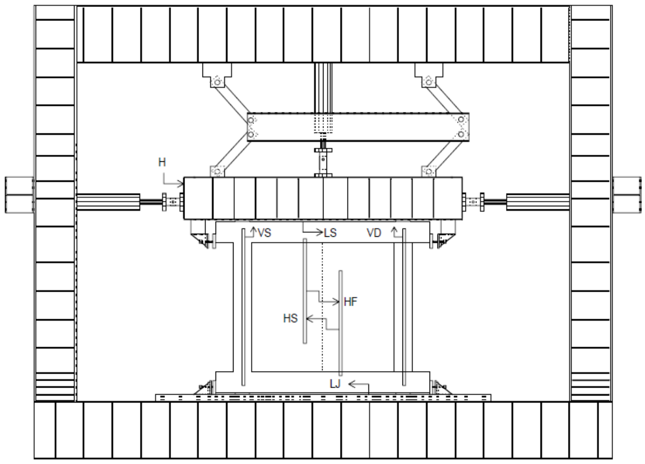


Figure 10. Test setup on the reaction frame

The drift was calculated as average of the two horizontal displacements, HS and HF.
The angle drift was calculated as:

$$Angle\ drift = \frac{Drift}{2100} \cdot 100\ [\%] \tag{1}$$

5. TEST RESULTS

5.1 Bare reinforced concrete (RC) frame (S1/F)

The maximum recorded lateral force was 81 kN, corresponding to 1 % lateral drift. The failure occurred at 2 % drift, in the negative cycle (Figure 11). The damages concentrated in the columns. Although some cracks occurred in beams, no significant damages were noticed in these elements (Figure 11).

5.2 Reinforced concrete (RC) frame with timber cushion and masonry (brick) infill (S2/FTCM)

The maximum recorded lateral force was 122 kN, corresponding to 4.2 % drift, on the positive cycle, and 168 kN corresponding to 3% drift on the negative cycle. The lateral strength was maintained up to 4% drift in the positive cycle (Figure 12). The damage was concentrated in the columns, top and bottom. A diagonal tension crack followed by shear sliding were noticed in the masonry. The cracks went through both mortar joints and bricks. The masonry panel together with the timber frame detached from the RC frame from early cycles. The hysteretic curve shows the significant contribution of the timber framed masonry in increasing the lateral strength and ductility of the concrete frame. A hardening phenomenon caused by the compression perpendicular to grain property of the timber was observed.

5.3. Reinforced concrete (RC) frame with brick masonry infill (S3/FM)

S3-FM specimen failed at 3% drift, with the loss of the axial force carrying capacity caused by the sear failure at the bottom of the columns. Transfer of the axial force to the masonry infill was observed (Figure 13). The recorded maximum lateral force was 190 kN, at -1 % drift. The masonry showed two types of failure mechanisms: diagonal tension crack and shear sliding. Initially one diagonal strut developed in the masonry. Subsequently, progressive damage of the masonry panel favored the development of several struts with a relative uniform compression field. The RC frame’s collapse occurs due to the shear failure of the columns at the bottom.

5.4 Reinforced concrete (RC) frame with timber framed masonry (brick) infill (S4/FTM)

The specimen maintained the lateral strength up to 3% drift after the shear failure of the columns at the bottom and at the top. The maximum lateral force was 107 kN, at 3 % drift (Figure 14). In the beginning of the test, the masonry panel rotated, due to the gaps in the timber frame’s connections. After the rotation in the timber joints reached the necessary level to close the gaps, or to exhibit compression perpendicular to grain in the timber elements, the masonry panels started to fail in shear sliding mode. As it can be seen in the hysteric curve, after entering the plastic range, the specimen had a stable envelope until the RC frame failed. This is due to the shear sliding in the joints of the masonry panels and the timber which gives flexibility in the same time with confinement for the panels. Initially, strut and tie is obvious in the panels, but gradually, with the shear sliding, several struts appear (i.e.: one in the upper part and one in the lower part) or the direction of the strut changes. In the end of the test, after the columns lost the axial load, it was naturally transferred to the neighboring timber framed masonry panel, and this was observed by the compression perpendicular to grain in the lower timber beam. Even from the beginning, the lower masonry panels (left and right) showed an out-of-plane sliding of 2, and respectively 4 mm. Until the end of the test, the out-of-plane movement reached 4 and 10 mm, respectively. In the beginning, the shear sliding occurs in the infill, in the lower panels, and gradually, the cracks appear in a stepwise way, especially in the upper panels.

5.5 Reinforced concrete (RC) frame with brick masonry infill and wire mesh (two layers) applied on all specimen (S5/FMFC)

Specimen S5-FMFc failed at 1.5% drift (Figure 15), due to shear failure at the upper part of the columns. The maximum recorded lateral force was 321 kN, at -0.25 % drift. Two types of masonry failure modes were observed: diagonal compression and shear sliding. The shear sliding was significantly observed on two horizontal joints, which created three panels, therefore three struts which pushed into the columns and distributed the damages along its height, rather than concentrate them in the bottom. The wire mesh was tensioned, thus pulled and twisted (in some areas) outside the plane, causing the damage of the mortar.

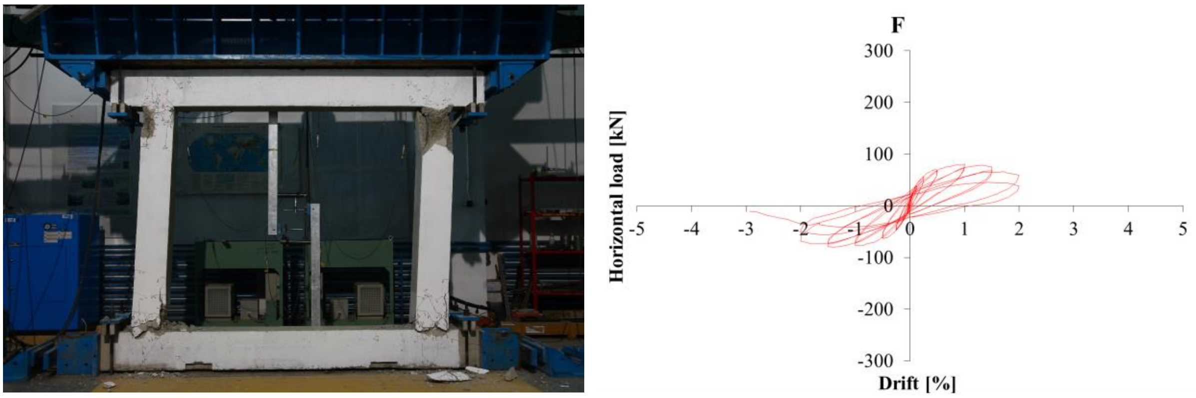


Figure 11. Frame’s (S1-F) failure at 2 % drift (left) and hysteric curve

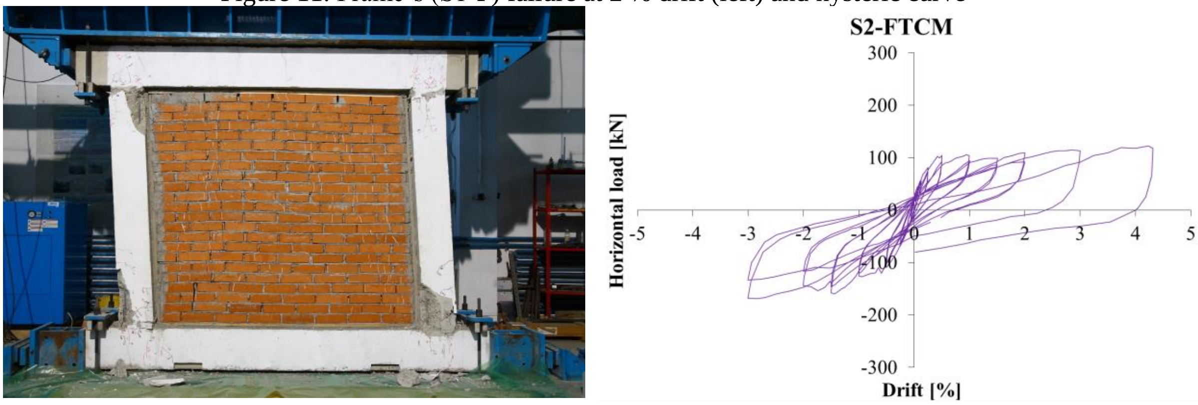


Figure 12. Frame’s (S2-FTCM) failure at 4% drift (positive cycle) (left) and hysteric curve (right)

Figure 13. Frame's (S3-FM) failure at 3% drift (left) and hysteric curve (right)

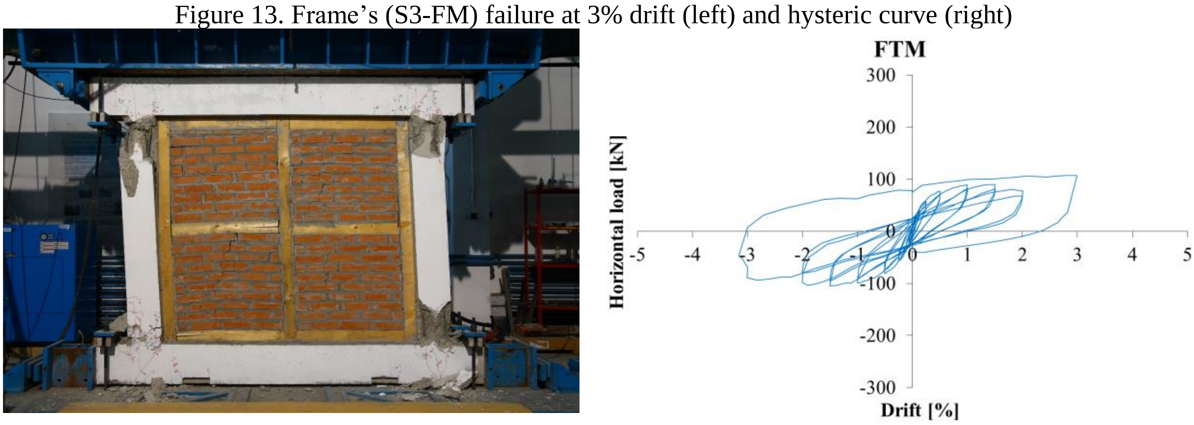


Figure 14. Frame (S4-FTM) at 3% drift (left) and hysteric curve (right)

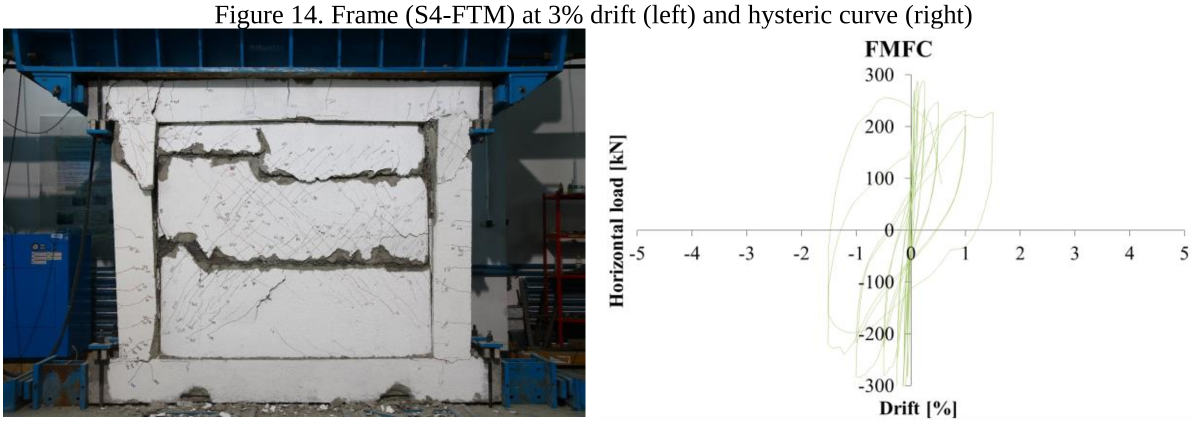


Figure 15. Frame's (S5-FMFC) failure at 1.5 % drift (left) and hysteric curve (right)

6. CONCLUSIONS

Four retrofitted RC frames using several types of infills (timber framed masonry, masonry, masonry and timber cushion, wire meshed masonry) were tested. The results were compared with those obtained on a reference bare concrete frame specimen.

From the strength point of view, the specimen with wire mesh and masonry infill (S5-FMFc) had the best behavior. Lateral strength of this specimen was 4 times larger than the strength of the reference specimen (S1-F). From ductility point of view, the specimen with thin timber cushion and masonry infill (S2-FCTM) showed highest values (4.3 % drift). This specimen had a displacement capacity 2 times larger than the reference specimen.

For S1-F, S2-FTCM and S4-FTM, the plastic hinges appeared at the top and bottom of the RC columns, in more or less same cycle (about 1% drift). In case of S3-FM and S5-FMFc, the cracks were distributed along the height of the columns and, for the later, also on the upper beam.

In case of S2-FTCM specimen, the lateral strength is increased at the beginning of the test, after 0.5 % drift the timber framed masonry panel starts to work, and a hardening behavior can be observed. This

may be due to the compression perpendicular to the grain property of the timber.
The masonry infilled frame, S3-FM, has almost double lateral strength in comparison with S1-F with similar lateral displacement capacity. Figure 16 shows that the envelope curves are highly similar, but with a constant strength difference of almost 100 kN.
The S4-FTM retrofitting technique significantly the ductility up to 3 % drift. A slight increase of the lateral strength in comparison with the reference specimen was recorded as well. The timber frame produced the hardening behavior which can be observed in Figure 16.
In terms of construction technology, all specimens were easy to build (as retrofit solution), not requiring specific skills.

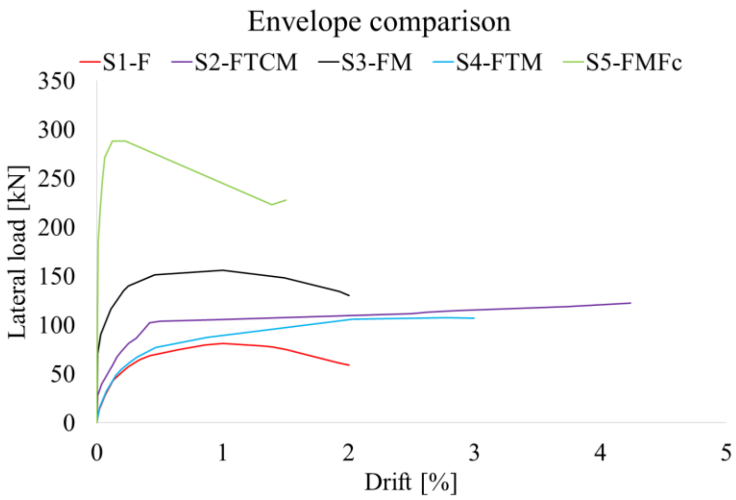


Figure 16. Comparison of all specimens' envelope curves

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