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Seismic evaluation of Romanian traditional buildings with timber frame and mud masonry infills by in-plane static cyclic tests

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ABSTRACT

The Romanian territory has an important seismic potential in Europe, with the Vrancea source. The most destructive seismic events that occurred in Romania in the 20th century (November 10, 1940 and March 4, 1977) have revealed a high level of seismic vulnerability of the built environment. An important part of this built environment is represented by historical buildings, including the traditional ones (timber frame and infills from various materials as brick, stone, adobe, etc.). The investigations after seismic events from November 10, 1940 and March 4, 1977, revealed that the traditional buildings did not suffer any or important damages, thus showed their particular seismic behavior.

Both the experience after similar seismic events from other countries as Turkey, Haiti, China, Myanmar, etc. and research studies from countries like Portugal, France and Japan, revealed an unexpected good behavior of such of buildings. Therefore, in this paper the results of the static cyclic tests on walls are presented. The test specimens were built according to the findings of the field investigations done on traditional buildings from Romania which are briefly presented hereby. The typology on which this paper focuses is the one found as predominant in the investigated areas (around the Vrancea seismic source).

1. Introduction

The present paper has as goal the seismic evaluation of traditional buildings with timber frame and mud masonry infill, which have proven over the time to be an earthquake resistant structure and with a remarkable architectural potential. Many countries in the world have structures with timber skeleton and masonry infill or other kind of infills, representing valuable heritage.

In some countries, timber framed walls were built most for aesthetical and architectural purposes (i.e. Germany, France, Czech Republic, etc.), although they may have structural role at least under gravitational loads. In others countries, they also have an earthquake resistance contribution (i.e. Portugal, Italy, Turkey, etc.) [1]. Timber framed masonry (TFM) system is also being presently used as reconstruction solution of areas that were destroyed by major earthquakes (i.e. Portugal, Pakistan) [1].

In most of the countries where these types of buildings are found, except Portugal and Italy, they were built without being based on any design regulation, but there are some situations (i.e. Turkey), where even if they date since 15th century, it was observed how people adapted their houses to local seismicity and made the structure as earthquake resistant as possible. Their behavior under earthquakes could be seen after some strong events as Kocaeli 1999, Kashmir 2005 or Haiti 2010. In the Izmir seismic event it was noticed that even if their damage state was advanced, at least they still stood up, while other types of structures fell [2]. In some situations, buildings with timbered masonry showed few damages (minor cracks, plaster falls, etc.), while poorly executed reinforced concrete structures near them collapsed or showed extensive damage [3].

Experimental studies were previously carried out for different configurations [4–9]. The common result was confirmation of the excellent behavior of in plane masonry infilled timber frames under cyclic loading, which is characterized by a significant ductility. Detailed and simplified numerical models were also developed for timber frames with masonry infills [10–12].

In Romania, in the last years, the studies on earthquakes produced an increase in the awareness of the population and authorities. For example, the most seismic exposed cities from Romania are Bucharest and Iasi. In Bucharest, according to seismic code P100-1/1992 the a_g (maxim expected seismic ground acceleration) was 0.20 g and today,

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Fig. 12. Upper structure of the house (Fig. 9 was made after this house).

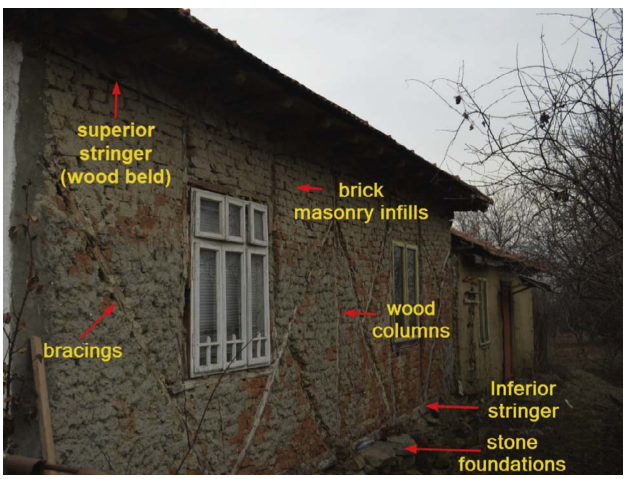

Fig. 13. Description of the type 1 structure – main specific structural elements.

Fig. 14. House with a partial basement.

attic.

3. Experimental programme

3.1. Test configurations and experimental setup

Based on the field investigations, it was decided to reproduce and build wall specimens, with the investigated types for static cyclic in-plane loading tests. The present paper focuses on the first typology, as being the predominant one in the field investigation. Due to the variation of the details of the houses, even when part of the same typology, it was decided to take one representative house and follow its details. It was observed that for type one, the house usually had a height of 2.7 m (from foundation until underneath the roof). In the same time, the best height for the testing frame setup was for 2.4 m. The maximum in-plane dimension of specimens it is 300 × 240 cm, and the scale was slightly reduced (1:0.88) so they can fit into the reaction frame. The test specimens have the dimensions as in Figs. 25 and 26.

Timber skeleton for S1 and S2 specimens is composed by vertical (columns), horizontal (stringers) and bracing timber elements (Pinacea) with 12 × 12 cm cross section. Both wall specimens have masonry infill (mud bricks with rough dimensions of 24 × 11.5 × 6.3 cm and earth mortar) and the parameter is the position of the diagonal timber brace.

The bottom connections are mortise-tenon type with nails and the upper ones are cross – halving type with nails (Fig. 27). The bricks are traditional, made from the local clay in Viperesti area. The walls to be tested are executed by a sub-contracted construction company, which had to follow as much as possible the real situation, according to the field investigation findings. In this respect, most of the used materials were collected from the Viperesti area (earth and bricks). The construction was done by first erecting the timber frame and fix it to the position, and afterwards laying the earth bricks, connected with the earth mortar.

Tests were conducted in a static cyclic regime on a self-balanced reaction frame provided with two horizontal hydraulic jacks with the maximum capacity of 1000 kN (228 mm stroke) and one vertical hydraulic jack with the maximum capacity of 2000 kN (100 mm stroke). Test setup is shown in Fig. 28 and Fig. 29.

The measurement system consisted of wire displacement transducers recording the lateral displacement of the specimen at the top (D1) and middle (HM) of the wall and inductive displacement transducers to measure the vertical displacement in two points (D3 and D4, on the backside of the wall), at the bottom of the edge columns (Fig. 28). Another two inductive displacement transducers were used to measure the horizontal slippage of the bottom timber beam against the reaction slab (D2), and also the upper beam against the loading beam. Rotation of the connections were measured by inductive displacement transducers, in pairs of two for each connection (left and middle bottom).

Axial load was applied at the top of the wall, with the value of 26 kN, representing the roof's weight and snow load. Although efforts were done to keep it constant, due to the big capacity of the jack (2000 kN) it had a 20% variation during the loading, and was controlled manually. Lateral load was applied manually and although the strength of the walls was very small compared to the capacity of the testing system, the accuracy was acceptable to obtain good results. The testing system has pantograph, but enough rotation was allowed by the way the axial load was applied: in between the loading beam and the upper timber beam of the specimen small steel plates were applied, through which the axial load was transmitted to the columns only. The specimen was fixed at the lower part to the reaction slab by steel bolts, and same system was used to connect it to the loading beam.

Average compressive strength for earth (mud) was obtained as 2.6 MPa. Compression tests were also conducted for masonry prisms made of mud bricks with layout as in Fig. 30. The average compressive strength was 1.86 MPa while the Young's modulus, determined as the secant stiffness corresponding to 1/3 of the maximum strength value, was 0.6 GPa. The timber is Romanian fir (Pinaceae), with an average specific weight of 385 kg/m^3 and an average moisture content of 15%.

3.2. Loading protocol

The CUREE Caltech standard protocol for wood frames was used with a loading history consisting of initiation cycles, primary cycles, and trailing cycles Fig. 31. Initiation cycles were executed at the beginning of the loading history to check loading equipment, measurement devices, and the force-deformation response at small amplitudes

a. The foundations are made only from stone (river rocks)

b. The diagonals are stopped below the upper connection between beam and column, and the last row of bricks is laid in an inclined position

Fig. 15. Details of traditional house with timber skeleton and brick masonry infill (Type 1) from Teiș village, Dâmbovița county.

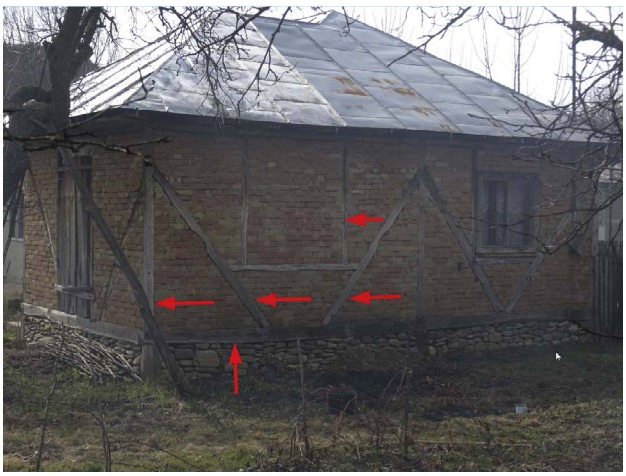

Fig. 16. Structural timber elements in "paianta" houses: columns and braces which do not intersect at the bottom.

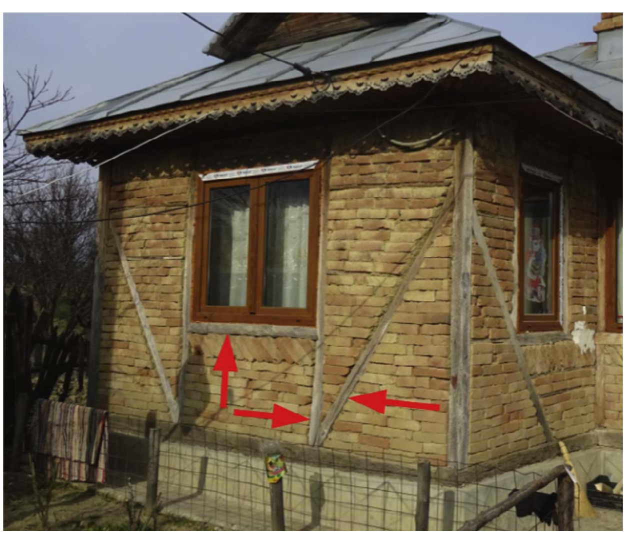

Fig. 17. Structural timber elements in "paianta" houses: columns and braces which do intersect at the bottom.

[19]. The choice of the protocol is made in reference to previous experiments to be able to compare the test results [20]. The reference displacement, Δ, was chosen based on previous experimental test observations as an 80-mm displacement at the top of the wall [21]. The input deformation, the shear angle, δ, was used to cancel the rocking by subtracting from the drift (displacement at the top of the wall) the deformation of the wall produced by the uplift in the bottom connections. This allows observation of the pure shear behavior of the wall. δ was calculated with the following equation:

$$\delta = \frac{D_1 - D_2}{Height} - \frac{D_3 - D_4}{Width} \tag{1}$$

where D1, D2, D3, D4 = measured displacements. Fig. 28 presents the test setup on the reaction frame and the displacement controlled loading protocol.

4. Experimental results

The experimental test results were assessed in terms of strength, stiffness, energy dissipation, damping ratio and ductility, with reference to ASTM 2126-09 [28] and EN 12512. Bi-linearization was applied using equivalent energy elastic-plastic (EEEP) method given in [22], and both methods a (for curves with two well defined linear parts) and b (for curves without two well defined linear parts) described in [23]. Both sides of the hysteretic loops were considered for analysis of mechanical properties. Results are shown in Table 1, for S1, and Table 2, respectively, for S2. The specific force, displacement and stiffness values that characterized each bilinear capacity curve are also reported according to the criteria based on the ductility ratio defined by [23].

Where: F_u is the force at the failure limit; d_u is the displacement at the failure limit; K_0 is the initial stiffness of the envelope curve; K_{pl} is the post-elastic stiffness of the envelope curve; F_0 is the residual force; F_y is the force at yielding limit; d_y is the displacement at the yielding limit; α is the elastic stiffness of the bi-linear capacity curve; $\mu = d_u/d_y$ is the ductility ratio. The parameters were defined as in [24]. Fig. 32 shows the hysteresis loop and envelope curves for both specimens in terms of horizontal force and top displacement (including uplift of connections). Figs. 33 and 34 show the bi-linearization curves for both S1 and S2.

Yielding condition is defined based on the bi-linearization methods that was used, and thus giving the variation in elastic branch stiffness values, and also in the ductility values. Results show that for S1, method b from EN12512 [22] is giving closer results to the envelope curve, this being due to the not clear two linear parts. But for S2 it overestimates the ultimate force, and thus method "a" seems closer to the envelope curve.

4.1. Specimen S1

Figs. 35 and 36 show the wall specimen in the undamaged state and, respectively, after the last cycle of test. The S1 specimen didn't fail until 0.053 rad (~5.3% drift) when the stroke of the jack reached the limit. It did not show any significant damage except local compression perpendicular to grain in the upper cross-halved connections (Fig. 37) and pull-out of the braces which on reverse cycles went back into original

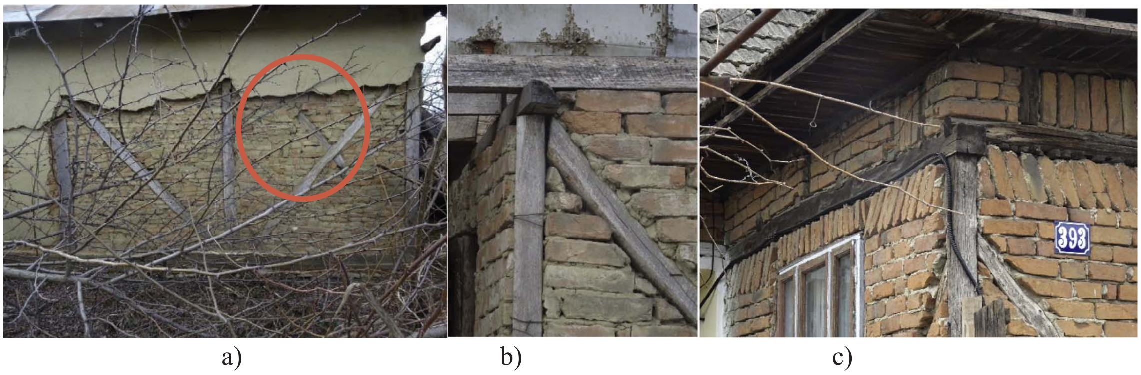

Fig. 18. (a) Stone foundations (river rocks); (b) bracings positioned at the corner's house; (c) The diagonals are stopped below the upper connection between beam and column, and the last row of bricks is laid in an inclined position.

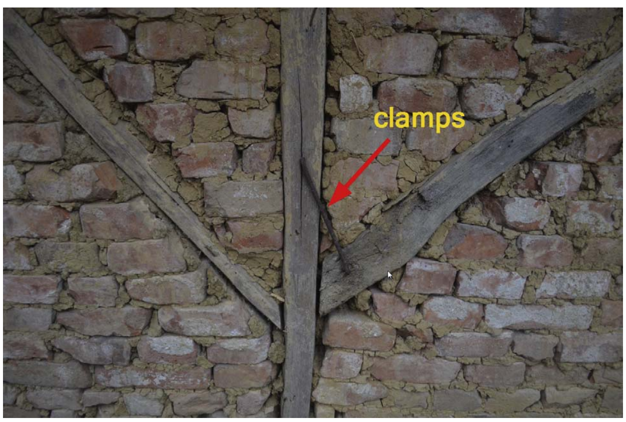

Fig. 19. Clamps used for timber elements connections.

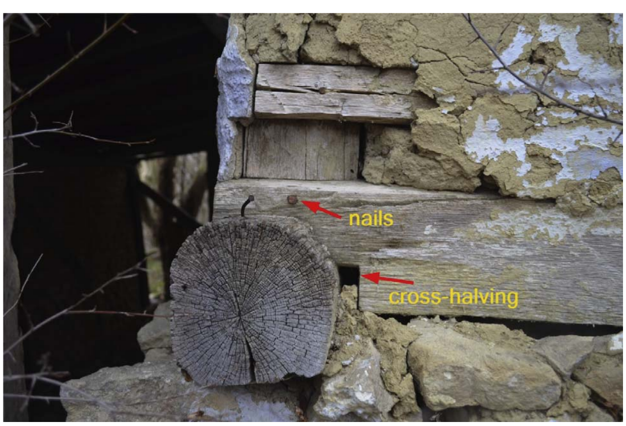

Fig. 20. Nails and cross-halving for timber elements connection.

Fig. 21. Mortise tenon connection.

Fig. 22. Detail of pile foundation connected to the bottom beam with steel clamps.

positions (Fig. 39). The uplift in the bottom connections were not significant (Fig. 38). Some initial defects from the execution did not influence the behavior (Fig. 40).

Out of plane behavior of the infill was significant (Figs. 41 and 42), starting in the areas below the diagonal brace. Mud mortar was initially cracked due to the shrinkage from the drying, as in the real situation. This is a common property of the mud, and due to this, every year, the owners usually make small repairs with new mud, applying a new layer and thus infilling the cracked walls (due to shrinkage). With the detachment of the infill from the frame, due to the large deformation at the top, the infill can easily fall).

Damages appeared after 0.01 rad (~1.1% drift), such as cracks in the infills at the bottom, attributable to large displacement at the top. Masonry panels separated from the timber frames after the first cycle of

Fig. 23. Detail of floor in an abandoned house.

Fig. 24. Detail of roof structure in an abandoned house.

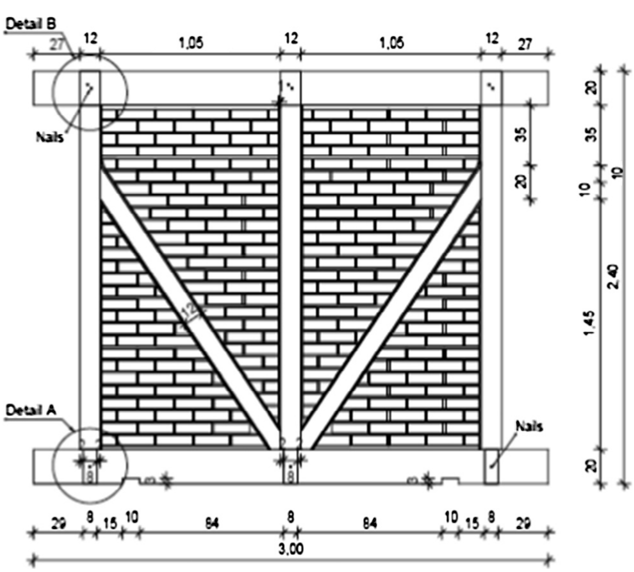

Fig. 25. Specimen **S1** (from type 1)-timber and brick masonry infill structure with the diagonal bracing lower than the upper timber joint [cm].

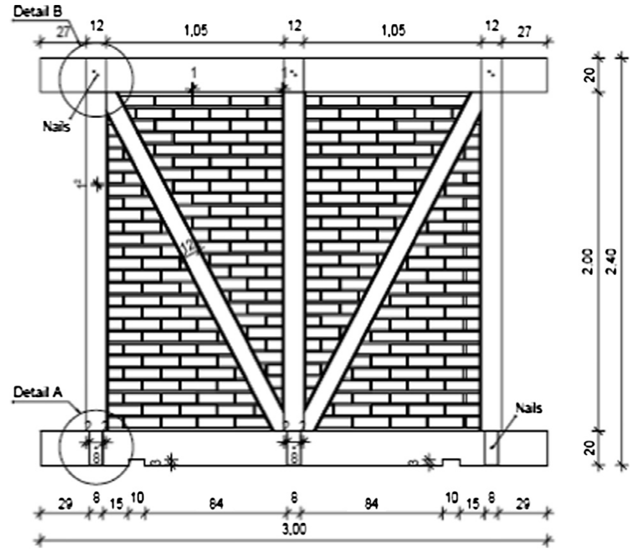

Fig. 26. Specimen **S2** (from type 1)-timber and brick masonry infill structure with the diagonal bracing connected to the upper timber joint (normal) [cm].

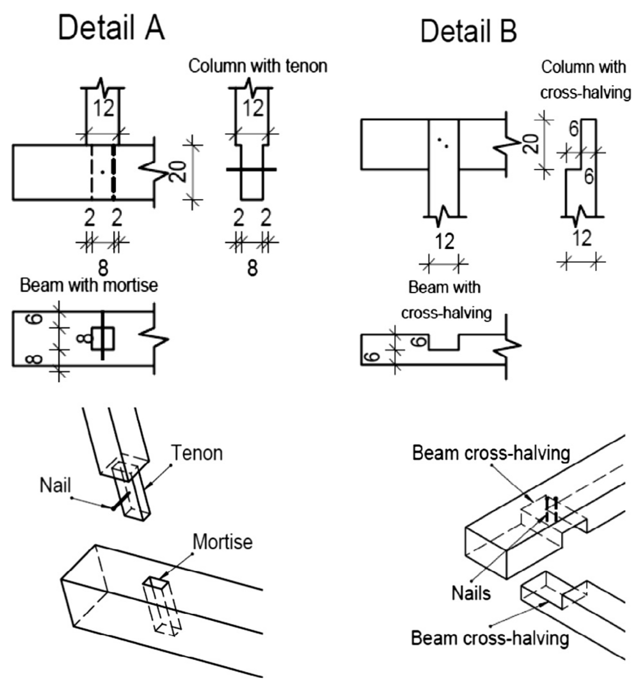

Fig. 27. Mortise-tenon (Detail A) and cross halving connections (Detail B) [cm].

the loading protocol. The tests reveal around 13 mm uplift of the bottom connections. The masonry infill cracked, starting from the bottom, and increasing towards the upper part of the wall. It has little contribution in the stiffness of the wall, since almost from the beginning, the infill was cracked (shrinkage) and easily detaching from the timber frame. However, the confining effect of the frame, keeps the infill in place and through shear sliding, it dissipates energy, which is an important contribution in the seismic behavior.

The strength of the mortar is highly important because the use of a weak mortar can direct the energy to dissipate in the mortar joints through cracking and, consequently, sliding. This type of damage was clearly observed in S1 (Fig. 36).

4.2. Specimen S2

Figs. 43 and 44 show the wall specimen undamaged and, respectively, in the last cycle of the test. The S2 specimen didn't fail until 0.048 rad (~5.6% drift) when the stroke of the jack reached the limit. It showed significant uplift in the connections (Figs. 45 and 46) and pull-

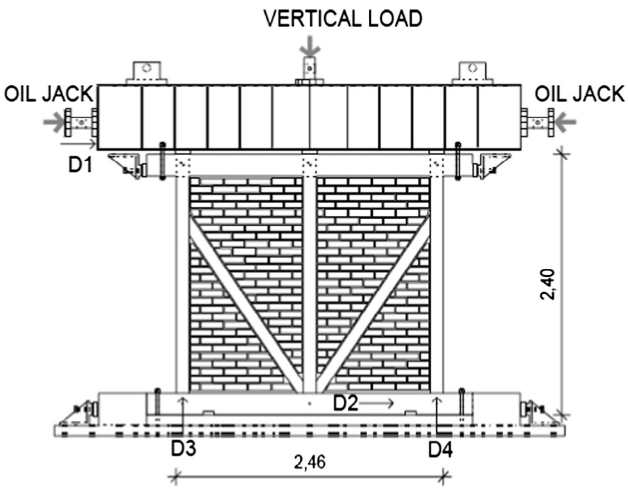

Fig. 28. Setup of wall specimen (measurement system) [m].

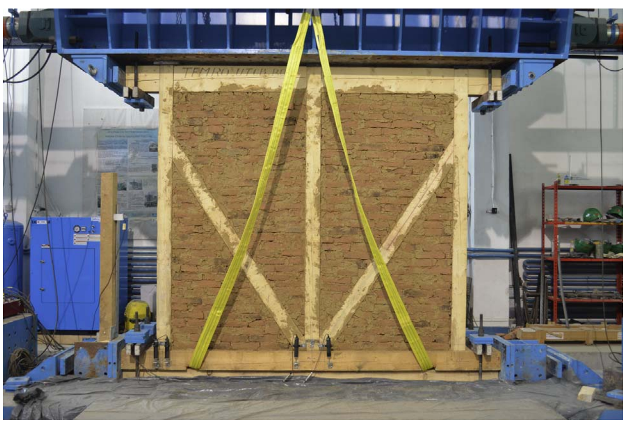

Fig. 29. Position of the wall in the reaction frame.

Fig. 30. Layout of specimen and compression test on masonry prisms.

out of the braces which on reverse cycles went back into original positions (Fig. 46). The reverse input of the tenon into the mortise in the beams also influenced the hysteresis, giving some stiffness immediately after unloading (Fig. 32). Out of plane behavior of the infills was significant (Fig. 47), similar to S1 specimen.

Damage appeared after 0.01 rad (~1.1% drift), such as minor cracks in the infills, attributable to large deformation of the wall and also detachment of the upper beam with local compression

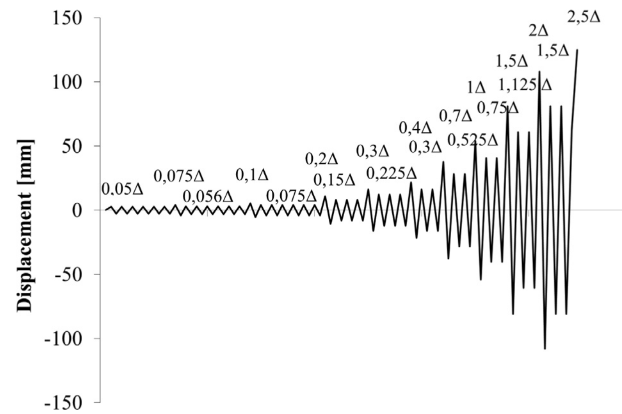

Fig. 31. CUREE Caltech loading protocol [19].

Table 1
Seismic characterization parameters corresponding to S1 specimen (lower diagonal).

Specimen S1	Positive loading			Negative loading		
Bi-lineariz. method	EN-a	EN-b	ASTM	EN-a	EN-b	ASTM
F_u [kN]	28.50	29.53	21.00	24.50	23.43	16.50
d_u [mm]	119.60			108.00		
K_0 [kN/mm]	2.23	0.65	2.14	0.96	0.42	0.94
K_{pl} [kN/mm]	0.24	0.11	0.00	0.26	0.07	0.00
F_0 [kN]	17.66	10.72	21.00	6.36	16.67	16.50
F_y [kN]	18.50	20.00	21.00	8.00	19.00	16.50
d_y [mm]	8.30	31.00	9.80	8.30	45.00	17.60
α [kN/mm]	2.23	1.30	2.14	0.96	0.90	0.94
β [kN/mm]	0.24	0.11	0.00	0.26	0.07	0.00
μ	14.41	3.86	12.20	13.01	2.40	6.14
Strength degradation		< 20%				
Ductility Class	H	L	H	H	L	H

Table 2
Seismic characterization parameters corresponding to S2 specimen (normal diagonal).

Specimen S2	Positive loading			Negative loading		
Bi-lineariz. Method	EN-a	EN-b	ASTM	EN-a	EN-b	ASTM
F_u [kN]	30.00	44.24	24.00	27.60	32.81	21.20
d_u [mm]	124.10	101.70				
K_0 [kN/mm]	2.95	1.33	1.60	6.29	0.90	1.10
K_{pl} [kN/mm]	0.24	0.22	0.00	0.34	0.15	0.00
F_0 [kN]	28.65	17.66	26.00	21.95	16.67	21.00
F_y [kN]	28.00	20.00	26.00	22.00	20.00	21.00
d_y [mm]	9.50	15.00	8.70	3.50	21.00	3.40
α [kN/mm]	2.95	1.30	2.90	6.29	0.90	6.25
β [kN/mm]	0.24	0.22	0.00	0.34	0.15	0.00
μ	13.06	8.27	14.26	29.06	4.84	29.91
Strength degradation		< 20%				
Ductility Class	H	H	H	H	M	H

perpendicular to grain of timber, where the diagonal brace pushes the upper beam. Masonry panels separated from the timber frames after the first cycle. The tests reveal up to 25 mm uplift of the bottom connections. The masonry infill cracked, starting from the bottom, and increasing towards the upper part of the wall. As in the case of S1, effect of the infill, was to dissipate energy by shear sliding in the joints, and also to oppose the buckling of the diagonal and being thus subjected to compression. The diagonal braces have the main contribution in carrying the lateral load.

4.3. S1 vs. S2

Comparison between the two specimens' hysteretic curves (Fig. 48) reveals a higher stiffness of S2, which is quite expected since the

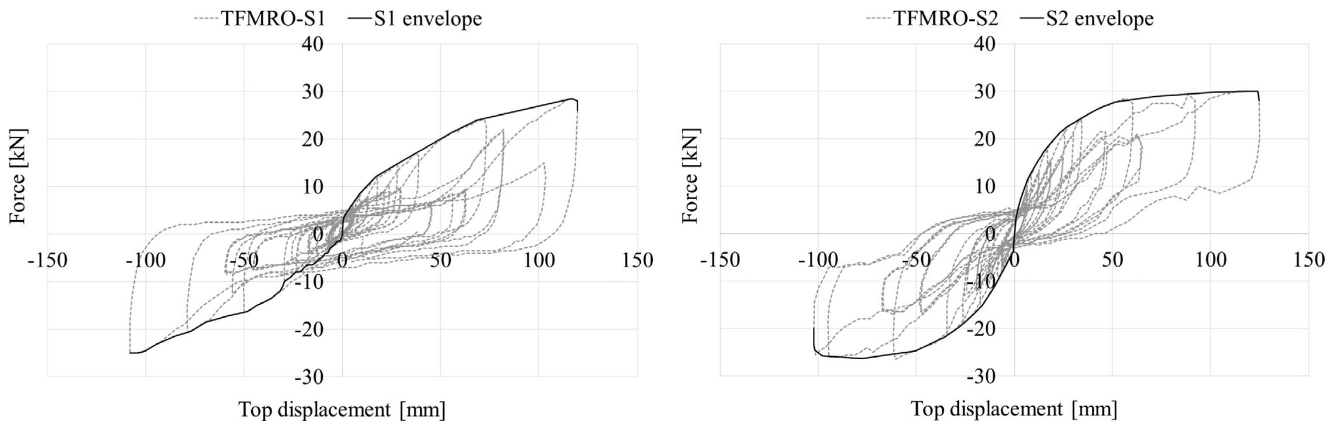

Fig. 32. Hysteresis loop and envelope of the first cycles for S1 wall with lower diagonal (left) and S2 wall with normal diagonal (right).

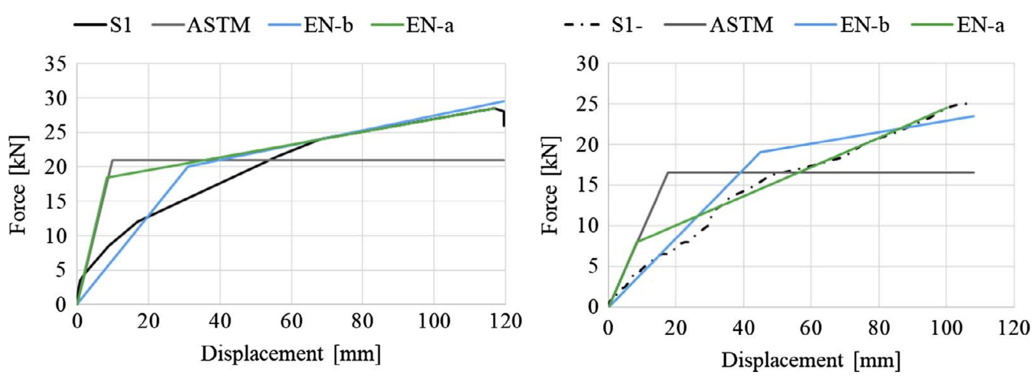

Fig. 33. Bi-linearization of the envelope curve for S1 on positive loading (left), and negative loading (right).

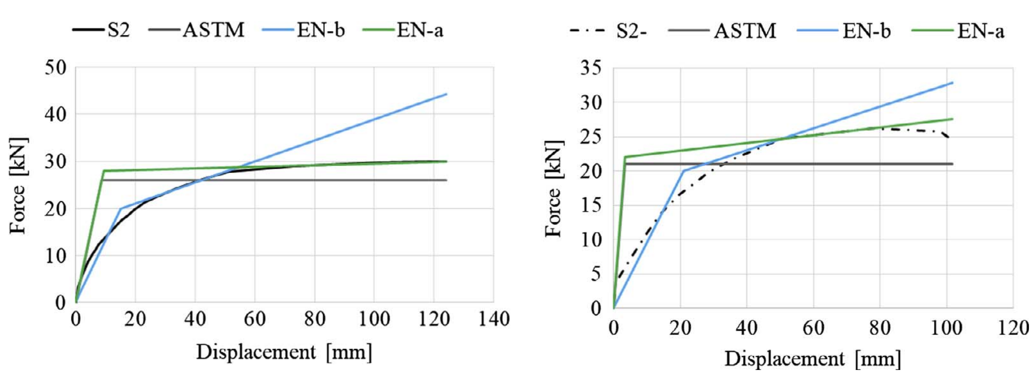

Fig. 34. Bi-linearization of the envelope curve for S2 on positive loading (left), and negative loading (right).

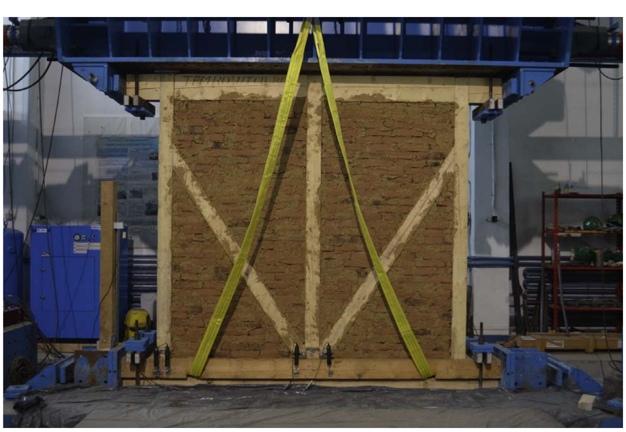

Fig. 35. S1 before the start of the test.

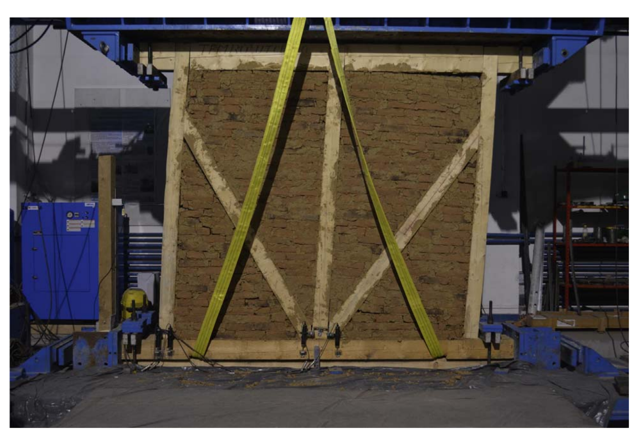

Fig. 36. S1 in the last cycle of the test, at 5.3% shear angle [rad].

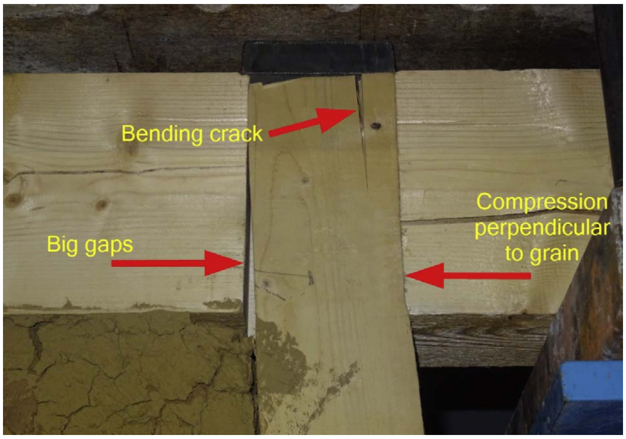

Fig. 37. Upper marginal timber connection compression perpendicular to grain.

Fig. 38. Bottom marginal connection with small uplift.

Fig. 39. Bottom middle connection with significant pull-out of one diagonal brace.

diagonal timber brace starts to work from early cycles, unlike the diagonal element in S1, which starts to contribute in the lateral load resistance later. It seems that until the last cycle, at 0.05 rad shear angle, they reach a similar value of strength (around 30 kN).

Considering the damages that the both specimens exhibited, S1 showed a less damaged state than S2, for which the uplift in the bottom connections was significant (Fig. 49).

In terms of energy dissipation, both S1 and S2 show similar values, 3.22 kN · m (S1), respectively 2.68 kN·m (S2). The energy dissipation,

Fig. 40. Upper marginal connection with imperfect executed connection (initial defect).

Fig. 41. Separation of the infill from the timber frame.

ΔW, was calculated as the area of the hysteretic curve for each specimen. Fig. 50, left shows the energy dissipation of S1 and S2 in every cycle.

The equivalent damping ratio h_e of the system was also calculated using the formula:

$$h_e = \frac{1}{4\pi}\cdot\frac{\Delta W}{W} \tag{2}$$

where ΔW = total dissipated energy by the specimen under cyclic loading; and W = potential energy at the maximum deformation. The values are presented in Fig. 50, right. Thus, the damping ratio for S1 is higher than the one for S2. The sudden drop in the damping ratio is explained by the fact that the S1 test was done in two different steps, first until cycle 24, and second from cycle 24. Between the two steps, the specimen's position in the reaction frame was moved, in order to take advantage of the full stroke of the jack, which was already very limiting. Both energy dissipation and damping ratio were compared starting from the 11th cycle, because the first cycles represent just

Fig. 42. Out of plane of the infill.

Fig. 43. S2 before the start of the test.

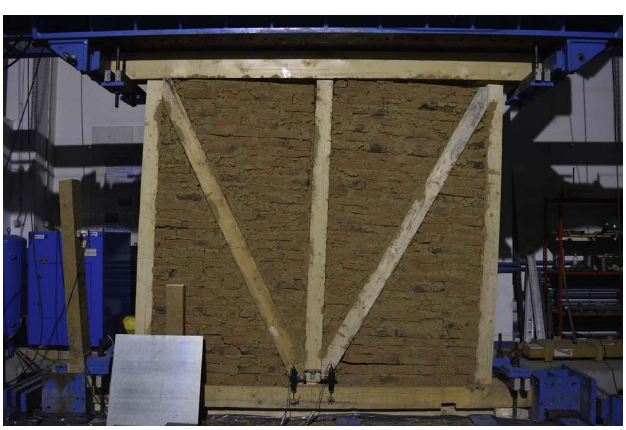

Fig. 44. S2 in the last cycle of the test, at 4.8% shear angle [rad].

verification cycles for the testing machine and were not considered relevant for analysis.

Secant stiffness is compared for both S1 and S2 along the positive

Fig. 45. Upper marginal timber connection significant uplift.

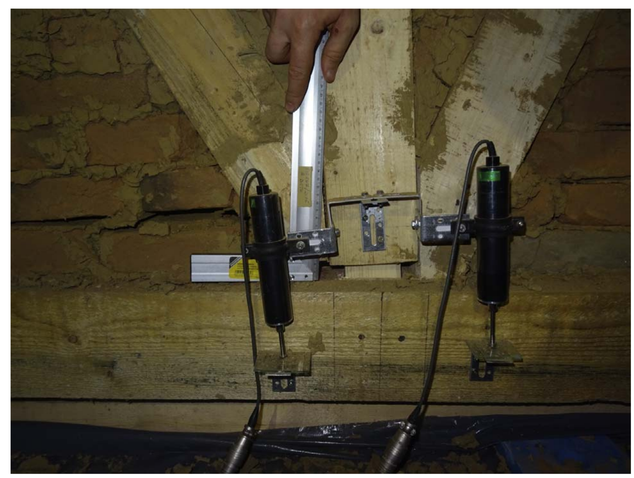

Fig. 46. Bottom middle connection with significant pull-out of one diagonal brace, uplift of central column and infill crack.

and negative loading envelope curve. Fig. 51 shows that the S1 has lower stiffness compared to the S2. This means that the position of the diagonal braces is important for the system, contributing when they are connected to the upper joint. On the other hand, for large displacement at the top of the wall, the uplift in the joints is not so high for S1, compared to S2. And the same situation applies for compression perpendicular to the grain.

4.4. Comparison with other test results on traditional and innovative light timber frame systems

For a better understanding of how the walls from traditional Romanian houses behave in earthquake, they were further compared with other experiments on: 1. Shear walls with studs and OSB/plywood panels sheathing [25]; 2. Walls composed by linear boards assembled with carpentry joints [24] (a. blockbau wall, b. Layered wall with dovetail insert); 3. Walls composed by rigid glued CLT panel assembled with mechanical connections (a. un-jointed CLT wall, b. Jointed CLT wall); 4. Walls composed by deformable panel assembled with metal fasteners (a. Stapled wall, b. Light frame timber wall, c. Heavy frame timber wall, d. Mixed wood-concrete frame); 5. Pombalino walls [21] and 6. Timber frames with masonry infills (TFM) [20]. Results of the comparison is shown in Table 3.

Observing the seismic parameters for TFM [20], it should be noted that the vertical force was not constant, and this had a significant influence in the shear capacity of the infill masonry panels, this resulting

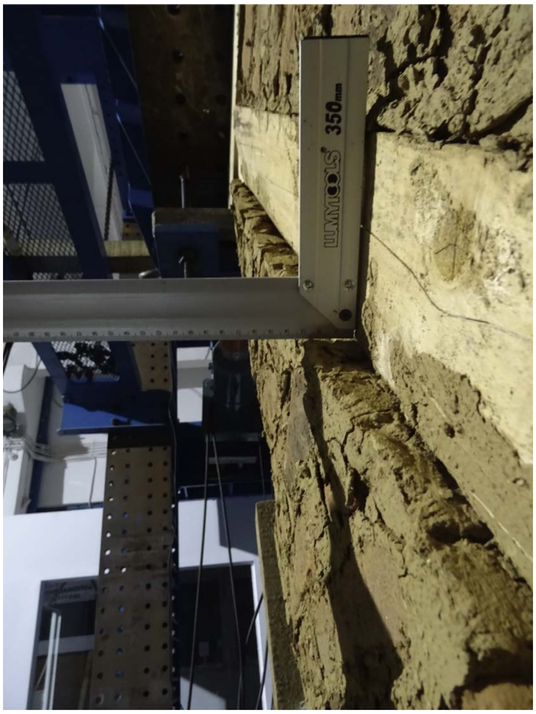

Fig. 47. Out of plane behavior of the infill.

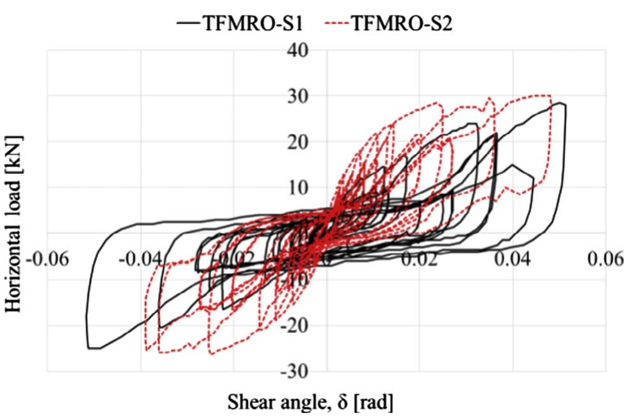

Fig. 48. Comparison of hysterical curves.

in a high initial stiffness. The ductility factor is high for the traditional Romanian walls, when compared with the other studies, but the initial stiffness is relatively low. The seismic parameters for them are in the specific range for traditional houses, as they are similar to BlockBau walls [24].

5. Conclusions

As it was observed in other countries, after important earthquakes, most of the infilled timber frames are resilient enough so they protect the lives of their inhabitants. Within this study, a field investigation was conducted in Romania and showed that in the rural areas (nearby the seismic source Vrancea), among the traditional houses, most of them have timber frame with brick infill structure. Other types of infilled timber frames were also identified, but main focus was on type 1: “paianta” because their larger number, but also due to other’s countries’ earthquake experience in which they showed a good deformation capacity and very low rate of collapse.

The “paianta” house has some details such as the diagonals being stopped lower than the connection between the beam and the column. Almost all houses having this structure investigated had this particular detail. Experimental tests on timber frames with mud brick masonry infill (“paianta”) showed the behavior of a wall specimen having as a varying parameter the position of the diagonal. Some observations are highlighted as follows:

- Both specimens showed high deformation capacity (over 5% drift), with a ductility factor over 13 (according to EN 12512, method a), which is quite significant even for timber frames;
- S1 has lower stiffness, because the braces is lower than the upper joint, and thus timber connections do not have much contribution, as in the normal diagonal case (S2);
- The uplift is more significant in the S2 (normal diagonal) and also local compression perpendicular to grain, but this latter property allows constant hardening in the hysteretic loop;
- The maximum strength is the same for both of them;
- Settlement of the walls was observed for both, due to the cracking of the mud mortar and easy falling which makes them vulnerable to out-of plane direction.

Three bi-linearization methods were used to identify the seismic parameters, and the most suitable resulted the method a, according to EN 12512 [22]. When compared to other experimental results [20,21,24,25], it is clear that the stiffness is lower than the innovative systems, and the ductility is significant. The characteristics of S1 and S2 of the present study are similar to the “BlockBau” walls, and thus within the specific parameters of traditional structures which do not often have any metal fasteners, other than simple nails.

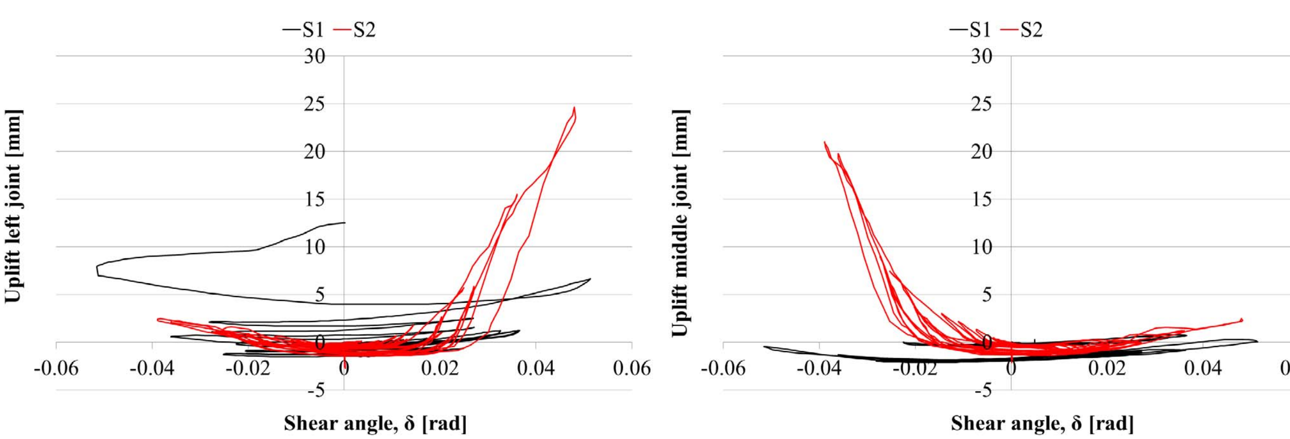

Fig. 49. Comparison of the uplift of the bottom left joint (left) and bottom middle joint (right).

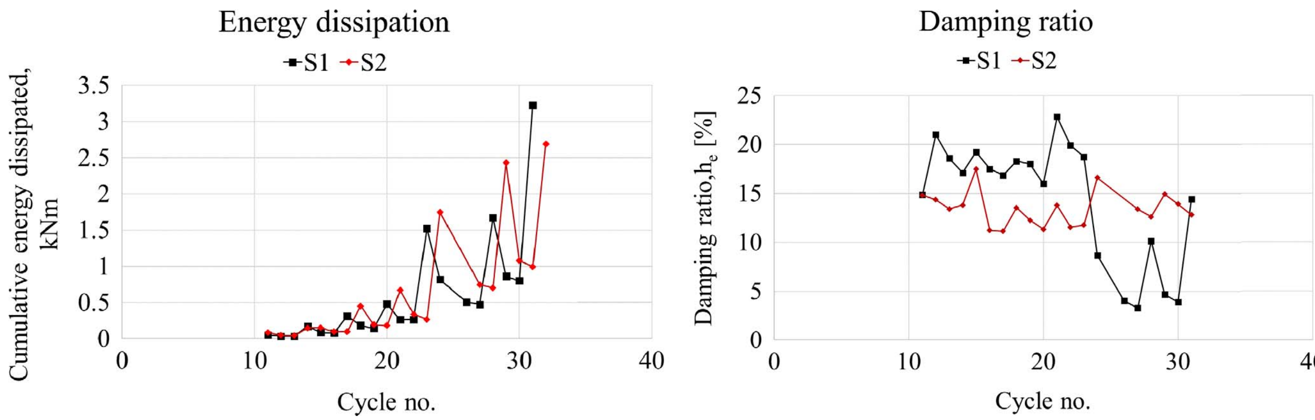

Fig. 50. Comparison of the cumulative energy dissipated per cycle number (left) and damping ratio (right).

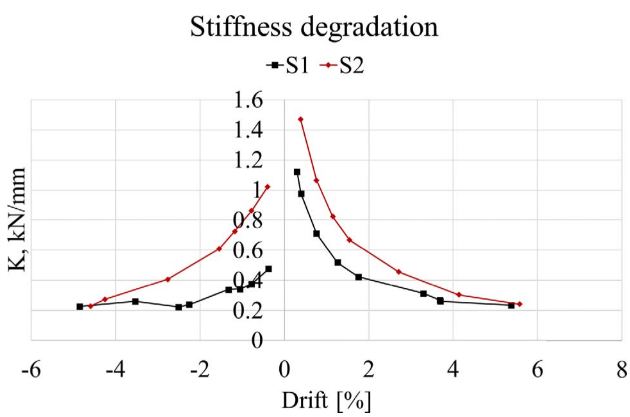

Fig. 51. Comparison of the stiffness degradation on positive and negative loading.

Some suggestions can be made on the improvement of the strength capacity of the timber frames with masonry infills:

- The connections can be improved with metal plates (T or L shaped) [26];
- Out-of-plane can be prevented with the insertion of barbed wire in the masonry joints [27];
- The infill masonry should not be strengthened to increase the stiffness too much, as it may become stronger than the timber frame, and this can cause rigid-body deformation and induce collapse of the frame; However, as shown from the comparison of the seismic parameters with TFM results (Table 3) [20], the infill can be made of burned clay bricks and thus stiffness can be added to the infill, which may have a good contribution in the stiffness of the wall; if the mortar should be used other than mud mortar, a lime mortar with low strength capacity (up to 5 MPa) should be considered in order to keep the property of dissipating energy through cracks in the mortar joints and shear sliding.

This experimental analysis is conducted for the first time on the traditional Romanian buildings, thus it represents an important result on the subject of seismic behavior of small residential houses and the beginning of the improvement and promotion of traditional Romanian architecture. If carefully chosen and applied the correct or improved construction details, it can be a cheap and easy to build solution for residential houses, using local materials, which also keeps the cultural identity of the Romanian traditional village.

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Table 3
Comparison of seismic characterization parameters for different experiments [20,21,24,25]

Wall type/specimen/Bi-linearization method/dimension	Vert. load [kN]	F_u [kN]	d_u [mm]	α [kN/mm]	β [kN/mm]	F_y [kN]	d_y [mm]	$\mu = d_u/d_y$
Timber frame with mud masonry infill/S1/EN-a/b = 2.4 m; h = 2.4 m	10 kN/m	28.50	119.60	2.23	0.24	18.50	8.30	14.41
Timber frame with mud masonry infill/S2/EN-a/b = 2.4 m; h = 2.4 m	10 kN/m	30.00	124.10	2.95	0.24	28.00	9.50	13.06
1.a Shear wall with studs and plywood panels [25]/Envelope curve/b = 2.44 m; h = 2.44 m	0	31.90	79.20	2.18	0.26	25.50	28.00	2.80
1.b Shear wall with studs and OSB panels [25]/Envelope curve/b = 2.44 m; h = 2.44 m	0	33.30	83.10	2.01	0.25	25.37	26.00	3.19
2.a Blockbau wall [24]/EN-a/b = 2.95 m; h = 2.95 m	10 kN/m	29.30	80.00	5.21	0.25	9.77	1.88	42.64
2.b Layered wall with dovetail insert [24]/EN-a/b = 2.95 m; h = 2.95 m	18.5 kN/m	67.00	80.00	3.50	0.36	42.58	12.17	6.58
3.a Un-jointed CLT wall [24]/EN-a/b = 2.95 m; h = 2.95 m	18.5 kN/m	78.72	38.40	6.77	0.73	56.71	8.37	4.56
3.b Jointed CLT wall [24]/EN-a/b = 2.95 m; h = 2.95 m	18.5 kN/m	108.62	51.90	7.25	0.93	69.31	9.56	5.42
4.a Stapled wall [24]/EN-a/b = 2.95 m; h = 2.95 m	18.5 kN/m	176.99	80.00	8.54	1.35	81.94	9.59	8.35
4.b Light frame timber wall [24]/EN-a/b = 4.88 m; h = 2.44 m	8.0 kN/m	46.00	35.00	6.00	0.43	33.23	5.54	6.32
4.c Heavy frame timber wall [24]/EN-a/b = 2.95 m; h = 2.95 m	18.5 kN/m	84.88	80.00	4.00	0.49	52.00	13.00	6.15
4.d Hybrid wood-concrete wall [24]/EN-a/b = 3.40 m; h = 3.24 m	20 kN/m	145.70	178.00	4.60	1.10	54.89	10.20	17.37
5. Pombalino wall [25]/EN-a/b = 2.16 m; h = 2.04 m	30 kN/m	46.43	55.02	4.49	0.84	34.00	7.58	7.26
6. TFM wall [14]/EN-a/b = 2.16 m; h = 2.22 m	30 kN/m	109.2	97.11	5.81	2.00	109.2	18.78	5.17

Laboratory in UTCB and Laboratory of the Reinforced Concrete Department, for the materials' testing.

Appendix A. Supplementary material

Supplementary data associated with this article can be found, in the online version, at http://dx.doi.org/10.1016/j.engstruct.2018.02.062.

References

[1] Dutu A, Ferreira JG, Guerreiro L, Branco F, Goncalves A. imbered masonry for earthquake resistance in Europe. Materiales de Construccion. 2012;62(308):615–28.

[2] Gülkan P, Langenbach R. The earthquake resistance of traditional timber and masonry dwellings in Turkey. 13th World Conf. On Earthquake Engineering, Vancouver, BC, Canada. 2004.

[3] Langenbach R. Online video—lecture at the world bank cultural heritage thematic group; 2011. < http://www.conservationtech.com/ > [Dec. 2013].

[4] Dutu A, Ferreira JG, Dragomir CS. Timber framed masonry buildings, an earthquake resistance influenced architecture. 2ndInt. Conf. on Structures and Architecture. Guimaraes, Portugal: Minho Univ.; 2013.

[5] Vieux-Champagne F, Sieffert Y, Grange S, Polastri A, Ceccoti A, Daudeville L. Experimental analysis of seismic resistance of timber-framed structures with stones and earth infill. Eng. Struct. 2014;69:102–15.

[6] Poletti E, Vasconcelos G. Seismic behaviour of traditional timber frame walls: Experimental results on unreinforced walls. Bull. Earthquake Eng. 2014;13(3):885–916.

[7] Ali Q, et al. In-plane behavior of full scale Dhajji walls (wooden braced with stone infill) under quasi static loading. Earthquake Spectra 2012;28(3):835–58.

[8] Ruggieri N, Zinno R. Behavior of the Borbone constructive system under cyclic loading. 1st Int. Conf. on Historic Earthquake-Resistant Timber Frames in the Mediterranean Area (Heart). Cosenza, Italy: Univ.of Calabria; 2013.

[9] Vieux-Champagne F, Sieffert Y, Grange S, Belinga Nko'o C, Bertrand E, Duccini JC, et al. Experimental analysis of a shake table test of a timber-framed structures with stone and earth infill. Earthq. Spectra 2017;33(3):1075–100.

[10] Ruggieri N, Sandhaas C, Ceccotti A. Seismic Vulnerability of Borbone Masonry Reinforced with Timber Frames. Cruz H, Saporiti Machado J, Campos Costa A, Xavier Candeias P, Ruggieri N, Manuel Catarino J, editors. Historical Earthquake-Resistant Timber Framing in the Mediterranean Area. Lecture Notes in Civil Engineering, vol. 1. Cham: Springer; 2016.

[11] Kouris LA, Kappos A. A practice-oriented model for pushover analysis of a class of timber-framed masonry buildings. Eng. Struct. 2014;75:489–506.

[12] Vieux-Champagne F, Sieffert Y, Grange S, Polastri A, Ceccoti A, Daudeville L. Experimental analysis of seismic resistance of timber-framed structures with stones and earth infill. Eng Struct 2014;69:102–15.

[13] Bostenaru Dan M. Vernacular and modernist housing in germany and romania an analysis of vulnerability to earthquakes, reports from the "World Housing Encyclopedia" EERI and IAEE project, Cuvillier, Göttingen; 2010.

[14] Dutu A, Yamazaki Y, Sakata H. Shear spring model proposed for seismic evaluation of a timber framed masonry infilled wall. Eng. Struct. 2018. [in press].

[15] Dima DI, Duţu A. Traditional buildings with timber frame and various infills in Romania. In: Proceedings of world conference of timber engineering, WCTE 2016, August 22–25, Vienna; 2016.

[16] Duţu A, Dima ID, Bulimar EG. Materials and techniques for traditional Romanian residential houses. 3rd International Conference on PROTECTION OF HISTORICAL CONSTRUCTIONS (PROHITECH), Lisbon, Portugal. 2017. 12–15 July.

[17] Riggio M, Parisi MA, Tardini C, Tsakanika E, D'Ayala D, Ruggieri N, et al. Existing timber structures: proposal for an assessment template. In: Jasienko Jerzy, Nowak Tomasz, editors. Structural Health Assessment of Timber Structures, Worclaw, Poland. 2015.

[18] Graham Saunders, Romain de Vries, Thierry Joffroy, CRAterre Editions. Assessing local building cultures for resilience & development – a practical guide for community-based assessment; 2015.

[19] Krawinkler H. Loading histories for cyclic tests in support of performance assessment of structural components. 3rd Int. Conf. on Advances in Experimental Structural Engineering. Berkeley, CA: PEER, Univ. of Berkeley; 2009.

[20] Dutu A, Sakata H, Yamazaki Y, Shindo T. In-Plane Behavior of Timber Frames with Masonry Infills under Static Cyclic Loading. J. Struct. Eng. 2015. http://dx.doi.org/10.1061/(ASCE)ST.1943-541X.0001405, 04015140.

[21] Duţu A, Ferreira JG, Goncalves A. The behaviour of timber framed masonry panels in quasi-static cyclic testing. 9th Int. Conf. on Urban Earthquake Engineering/4th Asia Conf. on Earthquake Engineering Tokyo: Tokyo Institute of Technology; 2012.

[22] ASTM 2126-09. Standard test methods for cyclic (reversed) load test for shear resistance of vertical elements of the lateral force resisting systems for buildings; 2009.

[23] CEN European Committee for Standardization. Timber structures—test methods—cyclic testing of joints made with mechanical fasteners. EN 12512, CEN, Brussels, Belgium; 2001.

[24] Ceccotti A, Massari M, Pozza L. Procedures for seismic characterization of traditional and modern wooden building types. International Journal for Quality research 2016;10(1):47–70.

[25] Karacabeyli E, Ceccotti A. Nailed Wood-frame Shear Walls for Seismic Loads: Test Results and Design Considerations. San Francisco, USA: Structural Engineers World Congress; 1998.

[26] Duţu A, Sakata H, Yamazaki Y. Comparison between different types of connections and their influence on timber frames with masonry infill structures' seismic behavior. In: 16th world conference on earthquake, 16WCEE 2017, Santiago Chile, January 9th to 13th 2017; 2017.

[27] Langenbach R, Kelley S, Sparks P, Rowell K, Hammer M, Julien OJ. Preserving Haiti's gingerbread houses. 2010 earthquake mission rep., World Monuments Fund, New York; 2010.

[28] ASTM E2126–09. Standard Test Methods for Cyclic (reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings.