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INFLUENCE OF AXIAL FORCE APPLICATION IN THE BEHAVIOR OF TIMBER FRAMED MASONRY WALLS UNDER IN-PLANE STATIC CYCLIC LOADING

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ABSTRACT: Timber framed masonry structures are representative traditional construction techniques for many countries in the world. Besides being of cultural value, these buildings proved to be also earthquake resistant, from both earthquakes and experimental testing.
The mechanical behaviour of these structures is highly influenced by the interaction between masonry infill and the timber frame. When there are no bracings within the timber frame, compression perpendicular to grain of timber is a property which produces the slow increase in the stiffness of the wall, even after yielding. Also, the shear capacity of the masonry is the defining parameter for the maximum resistance of the system.
In order to evaluate the seismic behaviour, in the present paper, the influence of axial force is discussed on the shear capacity of a wall made of timber framed masonry (TFM). Thus, two experiments were compared on walls with similar dimensions, but with different material properties and different method to apply the vertical loading. The first experiment was conducted in Japan, Tokyo Institute of Technology, while the second was conducted in Romania, Technical University of Civil Engineering Bucharest.

KEYWORDS: timber framed masonry, seismic behaviour, axial load, materials

1 INTRODUCTION

In the recent years, more studies were conducted on TFM buildings due to their earthquake resistance, cultural value, eco-friendliness and cheap construction.
The mechanical behaviour of these structures is highly influenced by the interaction between masonry infill and the timber frame. In the present study, the timber frame with no bracings is investigated, where the compression perpendicular to grain of timber is a property which produces the slow increase in the stiffness of the wall, even after yielding. Also, the shear capacity of the masonry is the defining parameter for the maximum resistance of the system.
In order to better understand the seismic behaviour of TFM houses, the influence of axial force is discussed on the shear capacity of a wall made of timber framed masonry (TFM), together with the materials' strength. Thus, two experiments were compared on walls with similar dimensions, but with different material properties and different method to apply the vertical loading (not-constant vs. constant). The first experiment was conducted in Japan, Tokyo Institute of Technology, while the second was conducted in Romania, Technical University of Civil Engineering Bucharest.

2 JAPANESE TEST - TEST SETUP AND RESULTS [1]

2.1 TEST SETUP

The dimensions of the timber frame and masonry infill are shown in Figure 1, having four masonry panels with the same characteristics [1]. Figure 2 presents the cross-halved connections of the timber frame, reinforced with screw nails having a 6 mm (~0.24 in.) diameter and 90 mm (~3.55 in.) length, chosen based on the available materials in Japan and on the fact that the nails shear capacity was previously determined in [2]. The specimen showed a maximum shear capacity of 118 kN and an initial stiffness of 9034 kN/rad calculated as the secant of the shear force versus lateral displacement relationship passing through the yielding point.
The results of this test are thoroughly presented in [1] and will only be shown briefly hereby.

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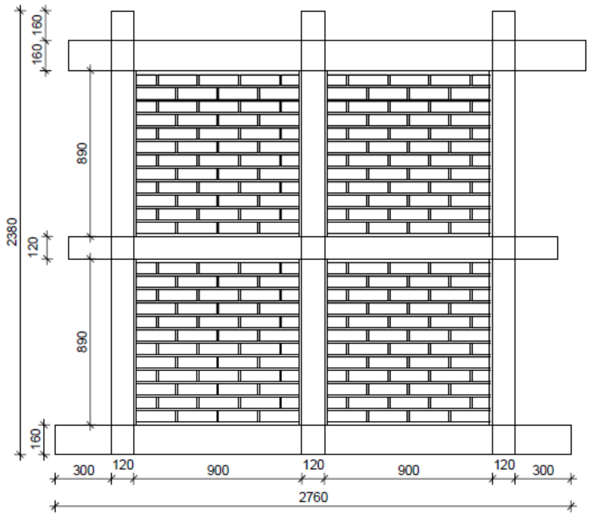

Figure 1: *Dimensions of the wall specimen in mm [1]*

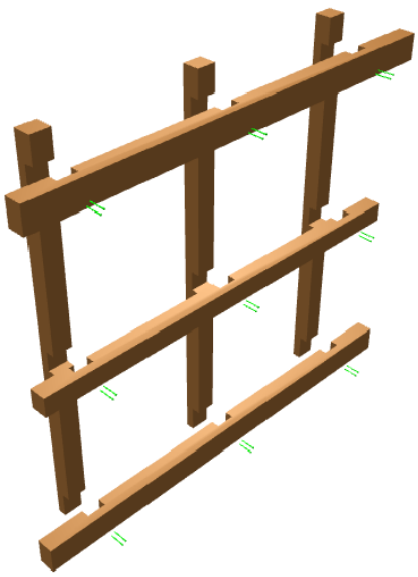

Figure 2: *Timber frame's connections – cross-halved with screw nails [1]*

A vertical force of 60 kN was initially introduced through steel tie rods and uniformly distributed on the top of the specimen using steel plates connected to the upper beam with screw nails. This value was calculated as the equivalent force acting on a first-floor wall in a four storey tall building.

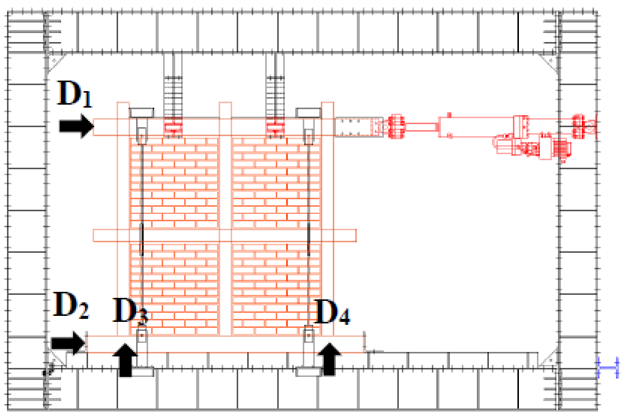

Figure 3*: Test setup (left) [1]*

The loading protocol is CUREE Caltech and is shown in Figure 4.

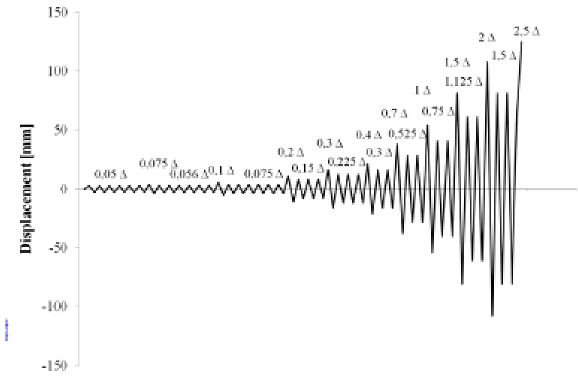

Figure 4*: Loading protocol [1]*

2.2 MATERIAL TESTS

Compression tests on mortar samples were carried out for each mortar made for both sub-assemble and wall test (i.e. for each test carried out that involved masonry) and also just to test the recipe. Table 1 shows the results. Same type of test was also performed on bricks, and the tests results are presented in Table 2.

Table 1: *Compressive strength (f_m) of mortar [1]*

No. of tested specimen for each type of test	f_m (MPa)
4	4.8
4	6.4
2	10.3
16	11.9
Average	8.35
Standard deviation	3.3

Table 2: *Compressive strength of bricks[1]*

No. of specimen	f_b (MPa)
1	34.08
2	57.2
3	60.6
4	64.1
5	62.6
6	67.2
Average	57.6
Standard deviation	12

Four-point bending and compression perpendicular to the grain tests were carried out on timber specimens belonging to the same batch as the timber elements used in the wall tests. Table 3 and Table 4, respectively.

Table 3: Maximum force obtained by bending test

No. of specimen	$F_{max,bend}$ (kN)
1	41.9
2	28.4
3	34.8
Average	35.0
Standard deviation	0.89

Table 4: Compressive strength perpendicular to the grain for timber [1]

Specimen (width 120 mm)	F_{cv} (MPa)
O-120-1	4.6
O-120-2	4.1
O-120-3	4.4
O-120-4	4.7
O-120-5	5.0
O-120-6	4.1
Average 120	4.5
Standard deviation	0.4

Compression strength was obtained from prism tests.

Table 5: Compressive strength of masonry prisms [1]

Specimen	Compression strength (MPa)
1	42.6
2	31.4
3	36.4
Average	36.8
Standard deviation	5.6

Bond-slip test between brick and mortar with and without pre-compression was conducted, and thus shear stress was obtained. The results are summarized in Table 6.

Table 6: Masonry prisms shear test results [1]

Specimen no.	Compression stress, σ [MPa]	Max. force recorded, Fmax [kN]	Max. shear stress, τmax [MPa]
1	0	15.98	0.53
2	0	10.91	0.36
3	0	30.28	1.01
4	0	13.82	0.46
5	0	37.43	1.24
6	0	25.62	0.85
7	0.3	24.96	0.83
8	0.3	31.53	1.05
9	0.3	24.93	0.83
10	0.7	58.94	1.97
11	0.7	37.34	1.25
12	0.87	51.01	1.7

2.3 TEST RESULTS

The specimen with a timber frame having masonry infills showed a maximum shear capacity of 118 kN and an initial stiffness of 9034 kN/rad calculated as the secant of the shear force versus lateral displacement relationship passing through the yielding point. Figure 8 shows the hysteretic behavior of the timber framed masonry panel (S2), represented in terms of lateral force and shear angle (rad).

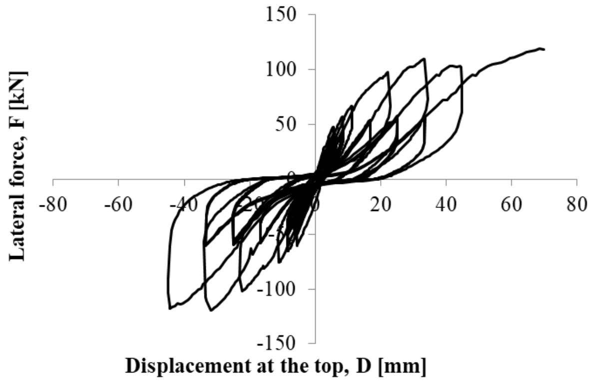

Figure 5: *Hysteretic curve of the wall test*

Various reasons lead to the early failure of the top timber beam such as the vertical loading system (the load was not perfectly uniformly distributed), the material defects (knots) or the cross-halving connection which reduces the strength of the element.

Damages appeared after 0,004 rad (~0.7% drift) due to compression perpendicular to the grain of timber elements induced from the masonry. Masonry panels separated from the timber frames after first cycle. The masonry started to crack in the bottom right panel above 0.003 rad (~0.5% drift), and in the bottom left panel, after 0.006 rad (~1% drift). The tests revealed significant uplift of the bottom connections (maximum was 30 mm). Although the masonry exhibited cracks that visibly extended from one end of the panel to the other around 0.012 rad (~2% drift), corresponding to 97 kN lateral force, the masonry infills continued to ensure the system's stiffness and to dissipate energy until the timber frame failed. 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 is the ideal failure mode for TFM and it was partly observed in S2, for large deformations (higher than 0.016 rad/ ~2.6% drift).

The vertical force introduced initially was 60 kN, but it increased directly proportional with the uplift and the displacement at the top (Figure 6). Although it is not clear the how much percentage of the axial force is distributed to the masonry panels and how much to the timber frame. It is known that the axial load has a significant influence in the shear capacity of the masonry, especially in TFM systems, where friction plays an important role due to the characteristic failure mode in shear sliding.

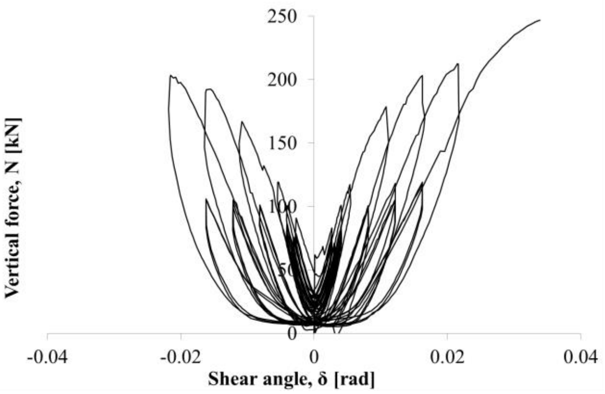

***Figure 6**: Variation of the vertical force with shear angle*

3 ROMANIAN TEST

3.1 TEST SETUP

A second test was performed in Romania, on a specimen that was slightly smaller than the one tested in Japan. The materials were different, specific to the ones available in Romanian market. The nails in the connections were screwed, specific for timber structures. The timber was quality B with a humidity around 10 %, softwood Pinacea type.

The test was conducted on a reaction frame with a pantograph system, and by applying the axial load on the columns of the specimen, the rotation of the upper beam was possible.

The specimen was connected to the reaction slab and the loading beam through steel bolts and steel restrains, which didn't allow the specimen any out of plane movement or sliding in the reaction frame.

Axial load applied was 55 kN, and the value was exactly as the weight of the loading beam. Although lower than the 60 kN value applied for the Japanese test, it was decided to keep it at this level due to the difficulty of manual control to keep the axial force constant. The vertical jack's capacity is 2000 kN and 60 kN is quite small for the total capacity, and it is difficult to keep it accurate.

Figure 7 shows the dimensions of the Romanian specimen.

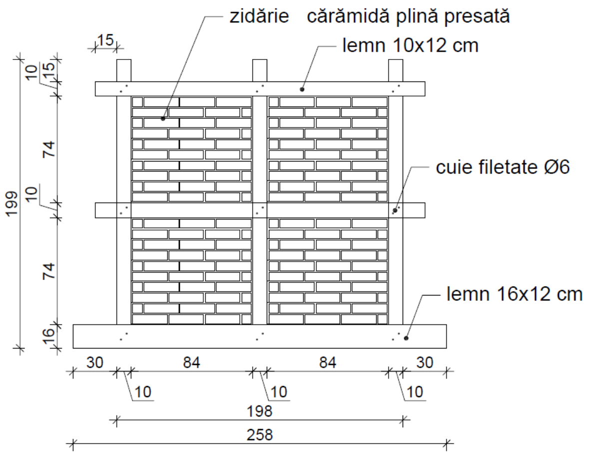

***Figure 7**: Romanian specimen dimensions [cm]*

The test setup is presented in Figure 8.

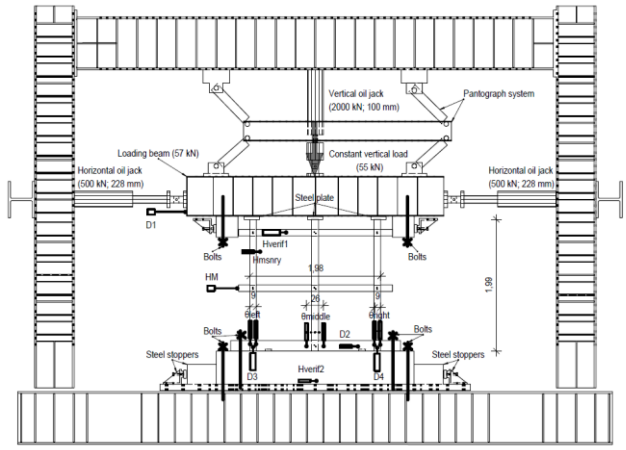

***Figure 8:** Romanian test setup*

3.2 MATERIAL TESTS

Compression tests on mortar samples were carried out. Table 7 shows the results. Same type of test was also performed on bricks, and the tests results are presented in Table 8.

***Table 7:** Compressive strength (f_m) of mortar*

No. of tested specimen for each type of test	f_m (MPa)
1	3.56
2	4.32
3	4.5
Average	4.12
Standard deviation	0.49

***Table 8:** Compressive strength of bricks*

No. of specimen	f_b (MPa)
1	37.32
2	50.18
3	36.96
Average	41.49
Standard deviation	7.5

Four-point bending and compression perpendicular to the grain tests were carried out on timber specimens belonging to the same batch as the timber elements used in the wall tests. Results are shown in Table 9 and Table 10, respectively.

***Table 9:** Maximum force obtained by bending test*

No. of specimen	$F_{max,bend}$ (kN)
1	18.1
2	18.8
3	11.8
Average	16.2
Standard deviation	3.85

Table 10: *Compressive strength perpendicular to the grain for timber*

Specimen (width 120 mm)	F_{cv} (MPa)
1	4.99
2	5.45
3	5.68
4	4.73
5	4.42
Average	5.6
Standard deviation	0.51

Compression strength was obtained from prism tests.

Table 11: *Compressive strength on masonry prisms*

Specimen	Compression strength (MPa)
1	13.00
2	7.37
3	13.04
Average	11.14
Standard deviation	3.26

Bond-slip test between brick and mortar with and without pre-compression was conducted, and thus shear stress was obtained. The results are summarized in Table 6.

Table 12: *Masonry prisms shear test results [1]*

Specimen no.	Compression stress, σ [MPa]	Max. force recorded, Fmax [kN]	Max. shear stress, τmax [MPa]
1	0.1	17.90	0.119
2		35.50	0.232
3		32.50	0.203
Average		28.63	0.185
Standard deviation		9.41	0.058

3.3 TEST RESULTS

The specimen showed a maximum strength of 49.5 kN (Figure 9), corresponding to 5.3 % drift, and a was tested until 6% drift when the jack's stroke limit was reached (Figure 10).

During the loading, the masonry panels cracked one by one, making a specific sound starting with the first crack and following their spread with lower sounds.

The yielding started around 0,008 rad (corresponding force was 24 kN), but the stiffness continued to increase, until 0,053 rad (corresponding force 49,5 kN) on the positive loading, while in the other direction the stiffness started to decrease significantly at 0,04 rad (corresponding force 48,5 kN), reaching 0,054 rad with a corresponding force of 31,5 kN. Although a decrease of 35% is observed, the specimen didn't show any fractures of the timber frame. The governing phenomena were diagonal tension crack and shear sliding in the masonry, after reaching the strut's capacity. Moreover, compression perpendicular to grain of timber was obvious and the crushing of the timber fibers around the nails could be heard.

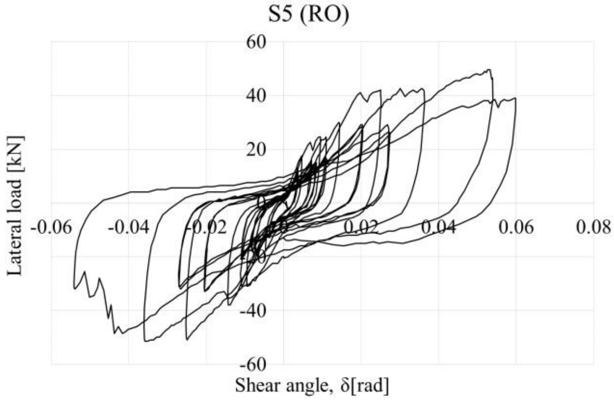

Figure 9: *Hysteresis curve for Romanian specimen*

Figure 10: *Specimen's last cycle at 6 % drift*

The specimen had a poor execution, specific for developing countries and gaps were obvious in the connections even before the test started (Figure 11). Moreover, due to the shrinkage process which was not done industrialized in the oven, some of the timber elements were distortioned from initial state (Figure 12).

Figure 11: *Gap in the connection due to poor execution*

Another type of execution error was considered the superficial cracks, present in many of the timber elements. Although superficial, they still represent defects of the timber, thus are considered weak parts.

Figure 12: *Execution errors: gap in the connections and distortion of the column*

Figure 13: *Gap in the connection due to poor execution and superficial crack in the bottom beam*

From early cycles in the beginning of the loading protocol, the separation between the masonry infill and the timber frame was visible (Figure 12).

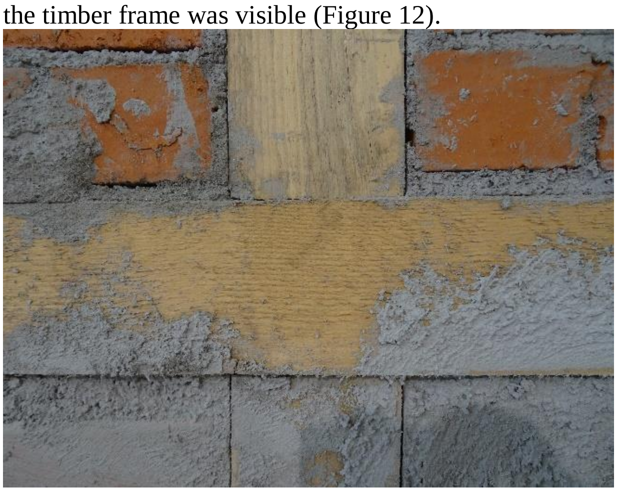

Figure 14: *Separation of the masonry infill from the timber frame*

The axial load versus the shear angle is shown in Figure 15. A light variation is observed for large displacements, but still in a neglecting values range.

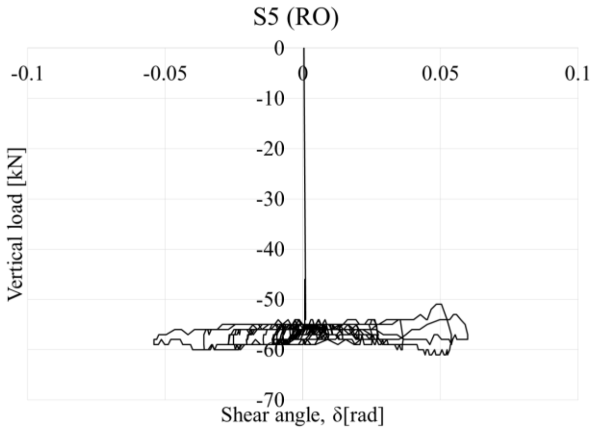

Figure 15: *Axial force value for Romanian specimen*

4 CONCLUSIONS

The envelope curves are compared in Figure 16. The stiffness difference is observed. The strength of the Japanese wall was 2.6 times higher than the one of the Romanian test.

The tests were not perfectly identical in terms of setup, so differences may occur initially from the test method. However, the difference being so large, other factors had to influence this result.

The most important factor was the axial load, which is known to contribute to the shear capacity of the masonry infill. Then, there was the poor execution quality in the Romanian specimen, compared to the Japanese specimen. For the first one, the connections were cross-halved on site, by chisel and hammer, while for the latter the cutting of the timber was done in the factory with very precise machines.

Another reason can be the difference in strength of the materials, which is 3 times lower for masonry prism in compression, for the Romanian test. The mortar was also half value, considering the average.

The timber bending strength was two times lower for the Romanian timber, while the brick compressive strength was 25 % lower.

All these factors influence the results of the test, but still the axial load has a

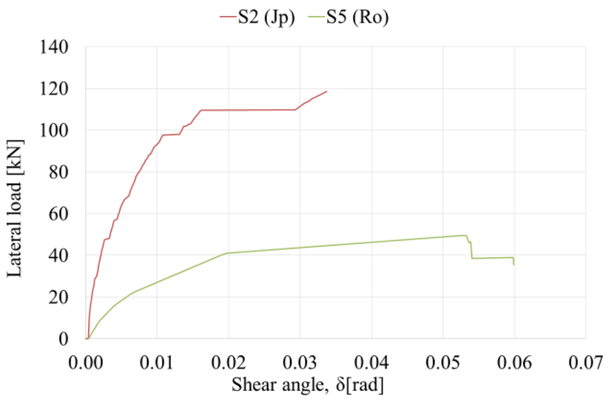

Figure 16: *Comparison of envelope curves for Romanian specimen and Japanese specimen*

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