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Seismic Analysis of Timber Frames with Infills in Romania

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Abstract: Like in many places around the world, before using reinforced concrete, in Romania, most buildings were made of timber frames and different types of infill. As the technology evolved, this type of structure has not been used so much anymore and construction method is rarely remembered correctly. But recently, traditional houses' architecture started to draw attention for owners, and moreover, the fact that it involves natural materials makes it even more attractive.
At this moment, in Romania, for this type of structures there is no specific design method specified in the national Code P100-1/2013, and also no evaluation procedures for this existent type of building. This fact motivated the present study and after a field study near Vrancea seismic source area, three categories of structures based on timber frames and different types of infills (masonry, wattle, daub and earth) were found and focused on.
This paper presents the results of a simple modeling using a finite element program ETABS, calibrated to an existing experimental test, previously conducted in Japan, on a timber framed masonry wall. The same wall was evaluated according to the existing regulations in Romania, to check the accuracy of the evaluation results. The comparison can be made because the general mechanical principles were of the wall configuration studied in Japan, are also valid for the Romanian traditional houses.

Keywords: Timber Frames, Masonry, Seismic analysis

1. INTRODUCTION

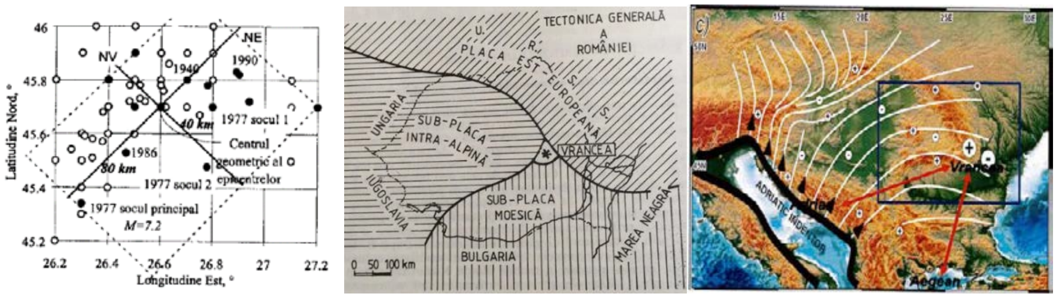
In Romania, in the last years the studies on earthquakes produced an increase in the awareness of the population and authorities. For example, the most seismic exposed cities from Romania are Bucharest and Iasi. In Bucharest, according to seismic code P100-92 [1] the ag (maxim expected seismic ground acceleration) was 0.20g and today, according to code P100- 1/2013 ag is 0.30g (50% increasing) and for Iasi city, it was also 0.20g and increased to 0.25g (25% increase).

After the two major earthquakes that occurred in Romania on 10 November 1940 and 4 March 1977, there is not much information about traditional buildings with timber frame and masonry infill or other infills which suffered complete collapse or major damages. Thus people generally assume that traditional residential houses behaved well during seismic events.

Today, more owners want to build such traditional houses with infilled timber frame structure, because they are easy to build, relatively cheap, ecologic, aesthetic and, the most important, as the recent studies have shown, they have a satisfactory seismic resistance and especially a high ductility, aspect also revealed by the past seismic events. In this moment, in Romania, for this type of structures there is no specific design method specified in the national Code P100- 1/2013, and also no evaluation procedures for this existent type of building.

2. TRADITIONAL HOUSES WITH TIMBER SKELETON AND VARIOUS INFILL FROM ROMANIA

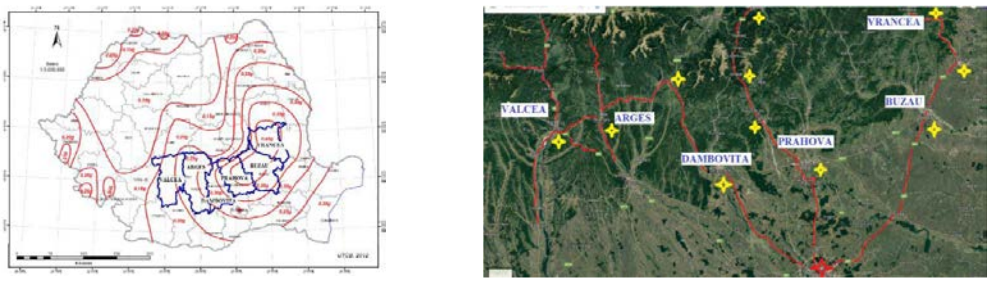
Romanian is one of the most earthquake prone country in Europe. The Romanian seismic source is a point source in Vrancea region, which was geographically defined as 40x80 km^2 [2] (Figure 1a) and is located at the confluence of three main tectonic plates as: East-European Plate, Intra-Alpine (subduction) Plate and Moesic Sub-Plate (Figure 1b) [3]. Also, Vrancea source is located at the influence of active deformation areas Adriatic, Aegean and Vrancea, through ALCADI Panonic system (Figure 1c) [3]. The Vrancea source can generate major earthquakes with very large focal energies released, around 2-3 seismic events/ century, that could have magnitudes on Richter – Gutenberg scale of M_{G-R}=7÷7.5.



a) b) c)

Figure 1. a) The epicenters distribution of Vrancea source **[2]**; b) Tectonic dynamic of Romanian territory **[3]**; a) Deformations transfer through ALCADI Panonic system **[4]**.

As worldwide, especially in seismic countries (i.e. Greece, Portugal, Turkey, Italy, China, Myanmar etc.), the traditional houses with timber frames and various infills can be found also in Romania. The previous studies shown that generally the traditional houses with timber skeleton and various infill were built mainly in seismic regions nearby the material's sources (wood, stone, clay), such as the mountain regions (where there are forestry and quarries) or hill regions. Thus, in order to study the seismic behavior of the traditional houses in Romania, some regions were selected, located near the Vrancea source (Figure 2a) and nearby mountain and hill regions in Buzau county, Vrancea county, Dambovita county, Prahova county, Arges county and Valcea county (Figure 2b).



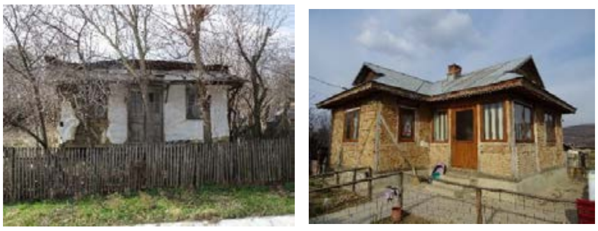
a) b)

Figure 2. a) The seismic location of the investigated regions; b) The geographic location [5]

A number of 129 traditional houses were investigated and the following statistics were obtained: 80% of them are with timber skeleton and brick masonry infill structure (Figure 3a); 15% are with timber skeleton and strips applied at 45^o and clay plaster (Figure 3b); 5% with timber skeleton and wattle & daub; additionally, 73% of them are between 60-90 years old; 14% of them are older than 100 years (Figure 4a) and 13% are younger than 60 years old (Figure 4b). Even though they aren't traditional ones, houses with timber skeleton and AAC (autoclaved aerated concrete) masonry infill was found.

a) b) c)

Figure 3. a) Traditional house with timber skeleton and brick masonry infill from Dumitreştii de Sus village, Vrancea county b) Traditional house with timber skeleton and strips applied at 45o and clay claster from Mustăţeşti, Argeş county; c) Traditional house with timber skeleton and wattle & daub from Băbeni, Vrancea county



a) b)

Figure 4. a) Traditional house with timber skeleton and wattle & daub over 100 years, from Băbeni, Buzău county; b) Traditional house with timber skeleton and brick masonry infill less then 60 years (~20 years), from Buzău county

A particular issue was observed during field investigations, all traditional houses are only with one story, except one (Figure 5a), unusual for Vrancea county, because it is built in “Fachwerk” style, just like Peleş Castel from Sinaia (Figure 5b).

a) b)

Figure 5. a) Traditional house with timber skeleton and masonry infill (only at 1st story and attic), from Chiojdeni, Vrancea county; b) Peleş Castel from Sinaia, Prahova county

Other important technical aspects are: the foundations are made only from stone (river rocks Figure 6a) with or without earth mortar; always the base of timber frames is done from hard- wood (oak tree, locust tree, etc.), able to sustain the moisture conditions, and the other are generally from softwood (pinewood); the timber structure is not embedded in foundations, it is only simply supported; the timber frame structure is built by vertical and horizontal elements and bracings, which are positioned at the corner’s and intersections’ structure, but not always in coherent distribution (Figure 6b); the joints are cross-halved (Figure 6c); steel clamps are added to increase the resistance and stiffness of the joint; almost all traditional house investigated have veranda; the roof covering of a traditional house, in traditional solution was made by wood shingle, but now most of them are replaced because they were damaged (biological decay).

a) b) c)

Figure 6. a) Stone foundations (river rocks); b) bracings positioned at the corner's house; c) cross-halved joints

From locals' testimony (personal communication) the past earthquakes didn't affect seriously this type of traditional houses. Some damages occurred after earthquakes, which were repaired immediately, but the serious damages were caused by the xylophage bacteria attack and moisture (lack of drainages, roof damages). Most of them are abandoned and they aren't maintained properly; also, the geotechnical phenomena (settling, landslides etc.) mainly caused cracks and walls overturning. Anyway, traditional houses withstand over years due to their reduced size dimensions (one story and maximum ~7 x15 m^2), remarkable constructive details, but also due to their ecological feature (natural material: stone, wood, earth). Today they are still building, but more rarely, because they are not promoted properly and the people are losing more and more the traditional construction method.

3. EVALUATION ACCORDING TO THE REGULATIONS IN ROMANIA

This chapter will present the evaluation of one wall according to the regulations in Romania. We choose to evaluate a timber framed wall with masonry infill, tested in Japan in a static cyclic regime. The analysis is compared with the experimental results [6], so later the evaluation can be adapted for the Romanian specific timber frames with infills. The present research is a continuation of the one conducted in Japan. The dimensions of the specimen are presented in Figure 7. The bricks were made in Japan whit the dimensions 210x100x60mm and the mortar recipe is 1:2:6 (cement:lime:sand). According to material tests the average compressive strength (fm) and Young's Modulus of mortar is 8.35 MPa respectively 13.01 GPa and the compressive strength and Young's Modulus of bricks is 57.6 MPa, respectively, 16.8 GPa. Young's Modulus for both mortar and bricks was calculated as the secant stiffness corresponding to one third of the maximum strength. There were also prism compression tests. The results are presented in Table 1 and Table 2 [6].

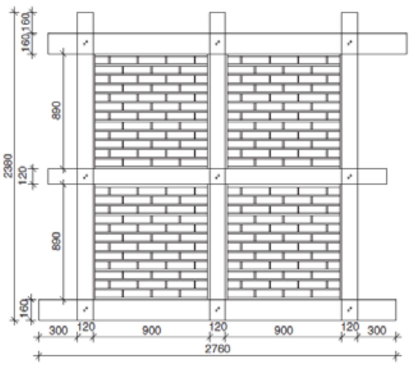


Figure 7. Timber-framed masonry wall [6]

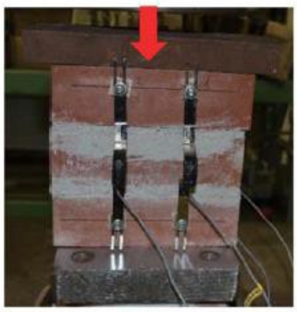


Figure 8. Prism compression setup [6]

Table 1. Test Results for materials

Material	Strength (f_m) [MPa]	Young's Modulus [GPa]
Mortar	8.35	13.01
Bricks	57.6	16.8

Table 2. Prism Compression Test Results

Specimen	Compression strength [MPa]	E masonry [GPa]
1	42.6	1.7
2	31.4	1.3
3	36.4	2.6

3.1. CR6-2006 [7]– Design code for masonry structure in Romania

Below are summarized the equations proposed by this regulation.

$$f_d = m_z \frac{f_k}{\gamma_m} \tag{1}$$

where: f_d = design compressive strength, m_z = coefficient of working conditions, f_k = characteristic compressive strength, γ_m = safety factor for material

$$f_{vd} = m_z \frac{f_{vk}}{\gamma_m} \tag{2}$$

where: f_{vd} = design shear strength, m_Z = coefficient of working conditions, f_{vk} = characteristic shear strength, γ_m = safety factor for material.

Taking into account the low number of tests performed on the materials, we will consider the lowest value of the characteristic compressive strength obtained from experimental test. According to Table 2 this value is f_k = 31.4 MPa.

Design compressive strength is calculated by reducing the characteristic compressive strength by the coefficient of working conditions and by the safety factor for material.

m_z depends on the brick section and mortar recipe. In this case, m_z was consider = 0.85 because the brick section is less than 0.30 m^2

γ_m depends on brick type, mortar recipe and limit state. For ULS (ultimate limit state), γ_m is 2.5.

Characteristic shear force (f_{vk}) is obtained from standard compressive strength, and initial shear strength considered 0.3 N/mm^2 and normalized axial force. Axial force was considered=60 kN.

This code does not require the calculation for diagonal tension cracking. Figure 9 and Figure 10 present the results given by CR6-2006.

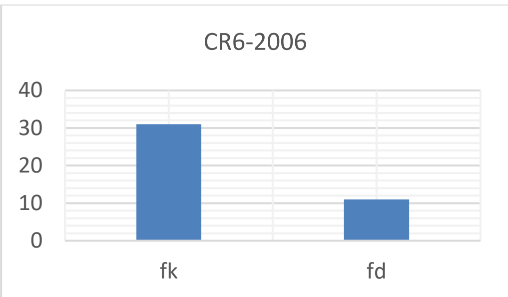


Figure 9. Compressive strength according to CR6-2006

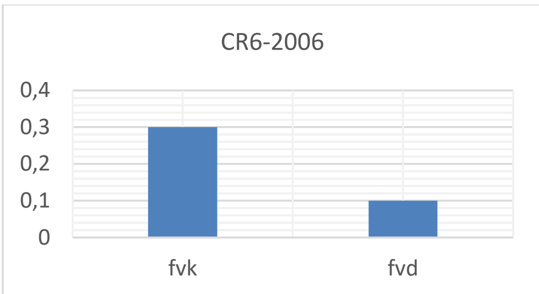


Figure 10. Shear strength according to CR6-2006

Design shear force capacity V_{Rd} is calculated with the following equation:

$$V_{Rd} = f_{vd} * t * l_c \tag{3}$$

where: V_{Rd} = design shear force capacity, f_{vd} = design shear strength, t = wall thickness l_c = the length of the compressed area of the wall.

Because of the length of the compressed area of the wall, V_{Rd} depends on the axial load. Because the whole wall is compressed, l_c = 2.16 m. It results that V_{Rd} = 216 kN

3.2. CR6-2013 [8] – Design code for masonry structure

This regulation replaces CR6-2006 and brings some modifications.

Beside the design compressive strength (noted f_d) and shear sliding in the joint (noted f_{vdl}) calculated as above (Equation 1 and Equation 2), in this case, CR6-2013 proposes the calculation of diagonal tension cracking (noted f_{vdi}). Its equation is found below.

$$f_{vdi} = m_z \frac{f_{vki}}{\gamma_m} \tag{4}$$

where: f_{vdi} = diagonal tension cracking strength, m_Z = coefficient of working conditions, f_{vki} = characteristic tension cracking force, γ_m= safety factor for material, and here is 2.2.

Characteristic tension cracking force (f_{vki}) is obtained from masonry tensile strength and normalized axial force. For both shear sliding and diagonal tension cracking force, axial load was considered = 60kN.

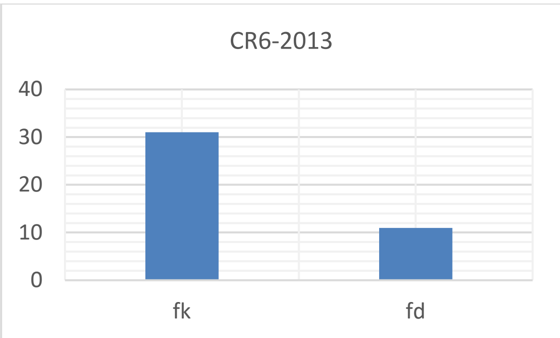


Figure 11. Compressive strength according to CR6-2013

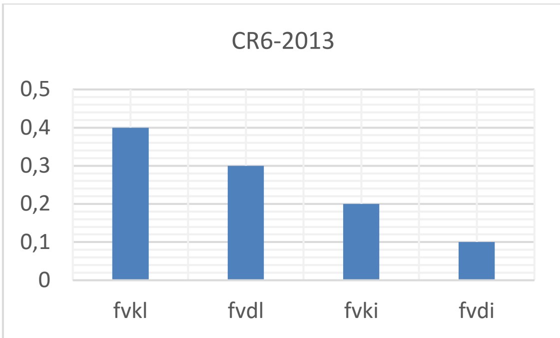


Figure 12. Shear strength according to CR6-2013

Design shear force capacity V_{rd} is minimum between shear sliding and tension cracking strength. Design shear sliding strength $V_{rd,l}$ is calculated with the following equation:

$$V_{Rd,l} = f_{vd,l} * t * l_c \tag{5}$$

where: $V_{rd,l}$ = design shear sliding force capacity, $f_{vd,l}$ = design shear sliding strength, t = wall thickness, l_c = the length of the compressed area of the wall

Because of the length of the compressed area of the wall, $V_{Rd,l}$ depends on the axial and lateral load.

Diagonal tension cracking force capacity $V_{rd,i}$ is calculated with the following equation:

$$V_{Rd,i} = \frac{A_w}{b} f_{vd,i} \quad (6)$$

where: $V_{rd,i}$ = diagonal tension cracking force capacity, $f_{vd,i}$ = diagonal tension cracking strength, b = coefficient depending on wall dimensions, A_w = area of the wall section.

In this case b=1 and $V_{Rd,i}$= 18.36 kN

3.3. P100-3/2008 [9] – Seismic evaluation of existing buildings

This seismic evaluation code uses as a reference point the mean value of compressive strength to calculate the standard compressive strength (Equation 7). As test showed, this value is 36.8 MPa (Table 2).

$$f_b = 0.85 f_m \quad (7)$$

where: f_b = standard compressive strength, f_m = mean value of compressive strength

This time, in order to obtain the design compressive strength, one should divide the mean value of compressive strength by two factors. (Equation 8).

$$f_d = \frac{f_m}{CF * \gamma_m} \quad (8)$$

where f_d = design compressive strength, f_m = mean value of compressive strength, CF= confidence factor, γ_m = safety factor for material

Regarding CF, we can choose from three confidence factors depending on how much we know about the building. If tests were done on materials and also exact measurements on site, it can be considered that there is a full knowledge over the building and CF = 1.

The safety factor for material, according to P100-3/2008, depends on the age of the brick and mortar recipe. In our situation, we can choose γ_m = 2.75. This value corresponds to old buildings (1900 to 1950) and mortar with lime and cement.

To evaluate the shear strength, we should determine the failure mechanism: shear sliding or diagonal tension cracking.

Equation 9 and Equation 10 describe the calculation method for shear sliding, respectively, diagonal tension cracking.

$$f_{vd} = \frac{f_{vk}}{CF * \gamma_m} \quad (9)$$

where: f_{vd} = shear sliding, f_{vk} = characteristic shear force, CF = confidence factor, γ_m = safety factor for material

f_{vk} (characteristic shear force) is obtained from standard compressive strength, initial shear strength considered 0.045N/mm^2 and normalized axial force. Axial force was again considered 60 kN.

$$f_{td} = \frac{0.04 * f_m}{CF * \gamma_m} \quad (10)$$

where: f_{td} = tension cracking, f_m = mean value of compressive strength, CF = confidence factor, γ_m = safety factor for material

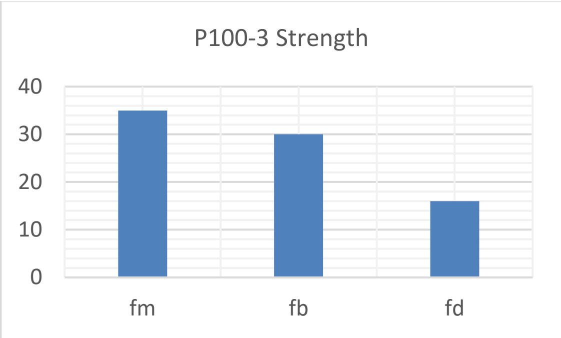


Figure 13. Compressive strength according to P100-3/2008

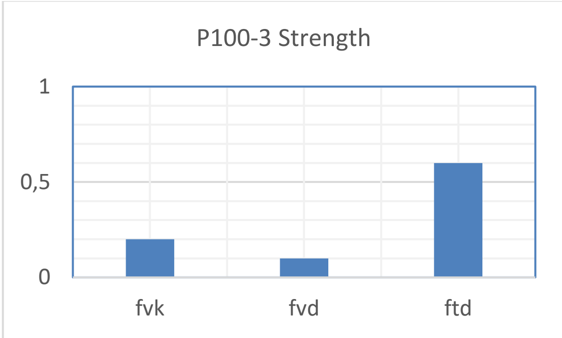


Figure 14. Shear strength according to P100-3/2008

Design shear force capacity V_{Rd} is minimum between shear sliding and tension cracking strength.

Design shear sliding strength V_{f21} is calculated with the following equation:

$$V_{f21} = \frac{1.33}{CF*\gamma_m}\left(f_{vk,0}\frac{l_{ad}}{l_c} + 0.7*\sigma_d\right)t*l_c \tag{11}$$

where V_{f21} = design shear sliding force capacity, CF =confidence factor, γ_m = safety factor for material, $f_{vk,0}$ = initial shear strength considered, l_c = the length of the compressed area of the wall, l_C = adhesion length, σ_d = normalized axial force, t = wall thickness If $l_{ad} \leq 0$ than V_{f21} will have the following value:

$$V_{f21} = 0.93\frac{N_d}{CF*\gamma_m} \tag{12}$$

where: V_{f21} = design shear sliding force capacity, CF = confidence factor, γ_m = safety factor for material, N_d= axial force

All these terms, except adhesion length have been discussed above. This term depends also on axial and lateral force.

Tension cracking strength V_{f22} is calculated with the following equation:

$$V_{f22} = \frac{t*l_w f_{td}}{b}\sqrt{1+\frac{\sigma_0}{f_{td}}} \tag{13}$$

where: V_{f22} = tension cracking force capacity, t = wall thickness, l_w = wall length, f_{td} = tension cracking strength, b = coefficient depending on wall dimensions, σ_d = normalized axial force. In this case V_{f22} = 38.27 kN.

3.4. Comparison between CR6-2006, CR6-2013 and P100-3/2008

As it can be seen in the charts below, evaluation of existing building code uses a compression strength higher than the design codes for new buildings. It is an expected outcome taking into account the for new buildings we follow the condition that at least 95% of the test to be above the value of fk and for evaluating existing building we use the mean value of compressive strength which is higher.

One can also notice the difference between design codes for new masonry structures in terms of design strength. CR6-2013 [8] uses a safety factor with a lower value than CR6-2006 [7] resulting in a higher design strength. The new design code, from 2013 gives higher design strength for shear and uses a value for tension cracking.

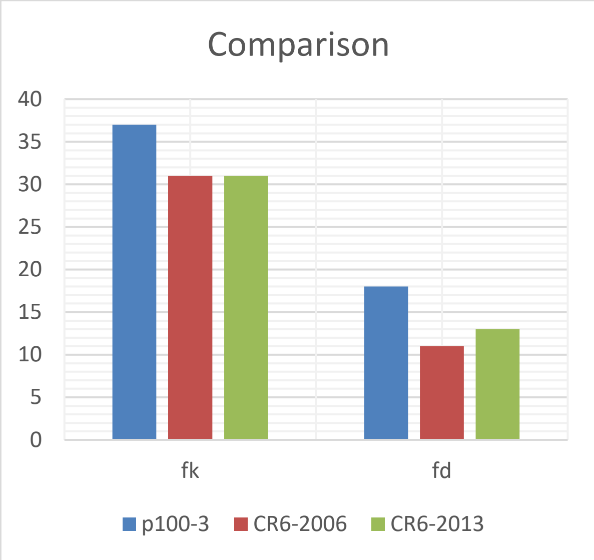


Figure 15 : Comparison of compressive strength

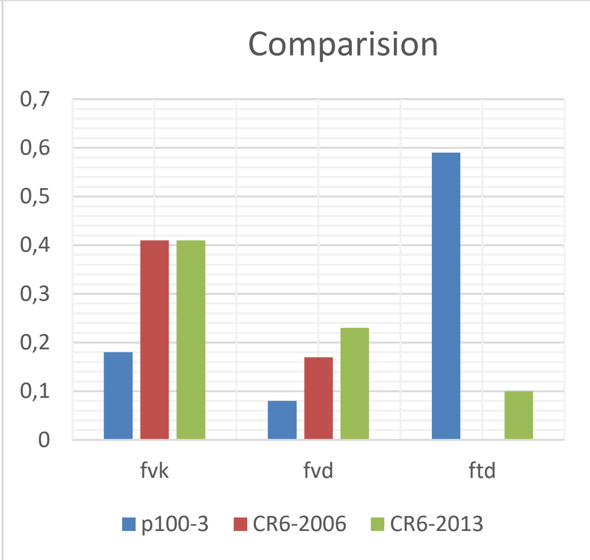


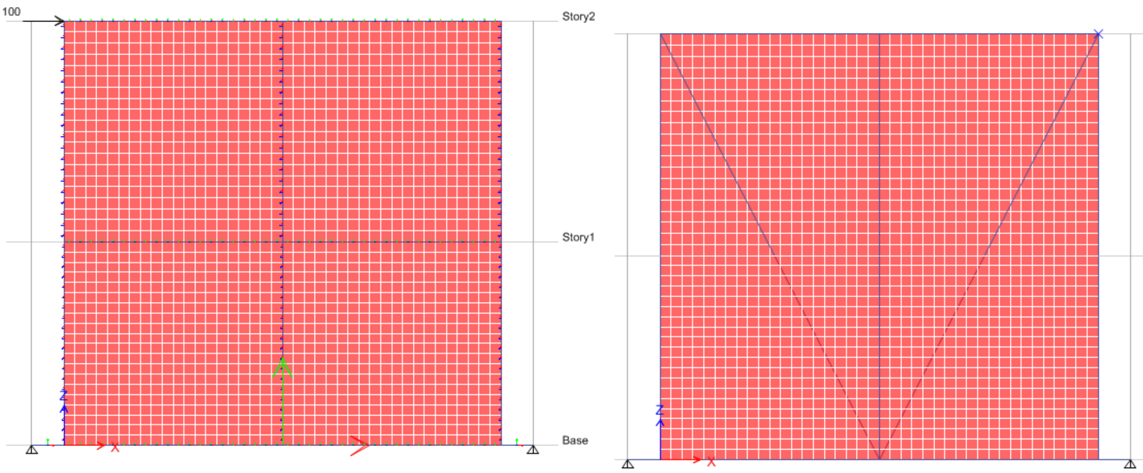
Figure 16 : Comparison of shear strength

4. FINITE ELEMENT MODELING

For numerical model, finite element software ETABS was used. The aim was to reproduce the test model's behavior whit a minimum computational level as everyone uses in current design.

Timber frame (beams and columns) were modeled as linear elements that intersect each other in their central axis. For simplicity, timber was defined as an isotropic material with Young's modulus obtained from test (11.92 GPa). The masonry panels were modeled using shell element with Young's modulus corresponding to characteristic compressive strength (1.3 GPa). Fixed support conditions were assumed as the experimental set-up of the wall [6].

Figure 17 a) presents the model made in CSI ETABS for the Japanese specimen and Figure 17 b) shows a conformation found in traditional Romanian houses. For more accurate results, masonry panels were meshed. The force was applied in the top left corner. For Romanian model ere used the elasticity characteristics for a mud masonry brick, determined through tests, and the minimum elasticity characteristics for timber since these tests were not made yet.



a) b)

Figure 17. ETABS Model for a) Japanese specimen, b) Traditional Romanian house

Figure 18 shows the envelope of the experimental result against the ones obtained using Romanian designing codes and the numerical models for both Japanese and Romanian conformation. It can be noticed that using the full stiffness for timber and masonry as an input for the finite element program, we obtain significantly lower comparing with the experimental behavior. To calibrate it to the test results, the ETABS model was modified by using only a fraction of stiffness for each type of element. The aim was to obtain a numerical model with an output that approximates the test results.

The best option in this case was the use of 10% of the actual stiffness of the elements. This input helps to model a wall with approximately the same stiffness as the one tested. The slope of the Top Force-Displacement chart is close to the real curve.

Similar results were presented in [10], where it is also presented a comparison between test result and numerical modelling, in this case using SAP2000 software. The situation described above (lower displacements comparing with experimental behavior) was the same, the authors reduced the geometrical stiffness of the timber frames and masonry panel by a factor of 37 to make the overall model stiffness the same as the experimental target value [10].

Regarding the values of the capable shear forces, calculated according to the Romanian codes, one can observe major differences between the three of them and between them and the real curve. However, it should be kept in mind that these regulations provide an approximate global calculation with coverage coefficients to avoid both design and execution errors.

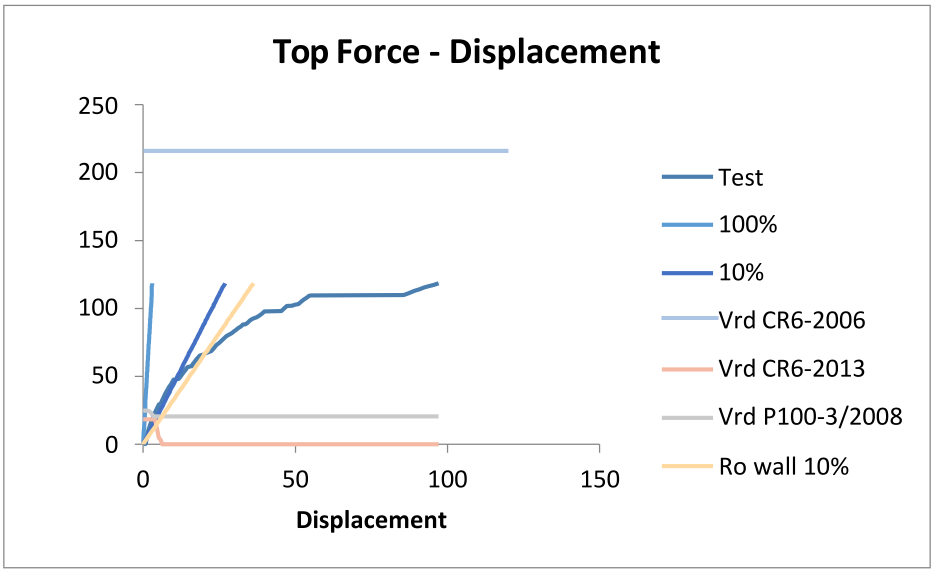


Figure 18. Top Force-Displacement for different fractions of rigidity

These low values could have various explanations, but the most important is that the masonry infill detaches quickly from the timber elements. In this case, the frame and shell do not act like a unified wall anymore. The masonry will be only an infill, and not a structural element. It is also interesting to see which is the appropriate fraction of stiffness to be use for different types of frames. For this rectangular frame, 10% of initial stiffness was obtained, while for a frame with St. Andrew's crosses, 2.7% was obtained [10].

5. CONCLUSIONS AND FUTURE DIRECTIONS

As it was presented at the beginning, timber frames with various infill houses has been and still are quite used around the world. Although it may seem an outdated method, the rustic look, the low price and the ease of purchasing the materials turn them into a desired home.

In Romania, the current regulations exclude such houses from the beginning. Materials in this combination, are considered too weak to meet the resistance and stability requirement. The characteristic values of compressive strength and shear force are far inferior to the materials commonly used during this period.

It is easy to see the evolution of the legal regulation regarding the new constructions but also the existing ones. CR6-2013, the newest code for masonry buildings, comes with more requirements (the calculation of diagonal tension cracking) but also with lower values of the coefficients for limit states. P100-3/2008, the only regulation for existing buildings, refers to CR6-2006 for equations and coefficients, even if is not used anymore. The major difference between these two is that P100-3/2008 uses as a reference point the mean value of compressive strength to calculate the standard compressive strength the same way we use now for nonlinear analysis for new buildings.

For the finite element modeling, there is also a discrepancy between real behavior and computational modelling. Software commonly used for design do not capture all the small phenomena, so in order to achieve an output closer to reality, it is necessary to use a more advanced program, in which each element to be modeled separately, and especially the interaction between them. However, for regular engineers, simplicity of modeling is very important.

6. ACKNOWLEDGEMENT

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