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NUMERICAL MODELLING OF THE SEISMIC PERFORMANCE OF ROMANIAN TRADITIONAL TIMBER-FRAMED BUILDINGS

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Abstract. *Traditional architecture made of timber-framed masonry (TFM) system is widespread around the world and has already been recognized as a unique cultural heritage to be preserved. These structures have shown a good seismic performance compared to other typologies because their configuration and construction details were constantly updated as soon as the builders addressed the causes of damage mechanisms when earthquakes occurred. Regarding this typology, Romanian TFM structures can be considered a representative example also because they experienced several seismic events showing their good earthquake-resistance. Although these buildings are still constructed and inhabited nowadays, no recommendation is provided in the Romanian building code and its structural behaviour is not properly characterized yet. Bearing in mind that the building's global response depends on many parameters such as the performance of its structural elements and their interaction, the calibration of shear walls is crucial to define the non-linear behaviour under cyclic loading. A simplified modelling strategy was chosen to simulate TFM wall response consisting of an equivalent frame with linear elastic elements and non-linearities lumped at the joints by using OpenSees. After calibrating the wall response according to the experimental campaign performed at Technical University of Civil Engineering of Bucharest, the panel was adapted to model a representative Romanian TFM building whose dynamic properties were evaluated by eigenvalue analysis and their potential calibration is proposed based on the ambient vibration tests.*

1. INTRODUCTION

Timber-Framed Masonry (TFM) structures can be found in many seismic countries even though their characteristics may vary according to the available materials, techniques and knowledge resulting in several structural configurations of timber frame (dimensions and arrangement of elements) and type of infill [1]. The system has been iteratively improved by local builders by damage observation after seismic events that let them understand the basic concepts of earthquake engineering [2].

Considering the number of earthquakes, their return period and energy, Romania can be considered as a country with moderate to high seismic risk, especially Vrancea region in the fore-Arc of Carpathians, where the most active tectonic processes are lumped [3]. Many TFM buildings can be observed across this area validating again the correlation between the typology and seismic hazard, and they were classified in five different types during the field investigation in [4].

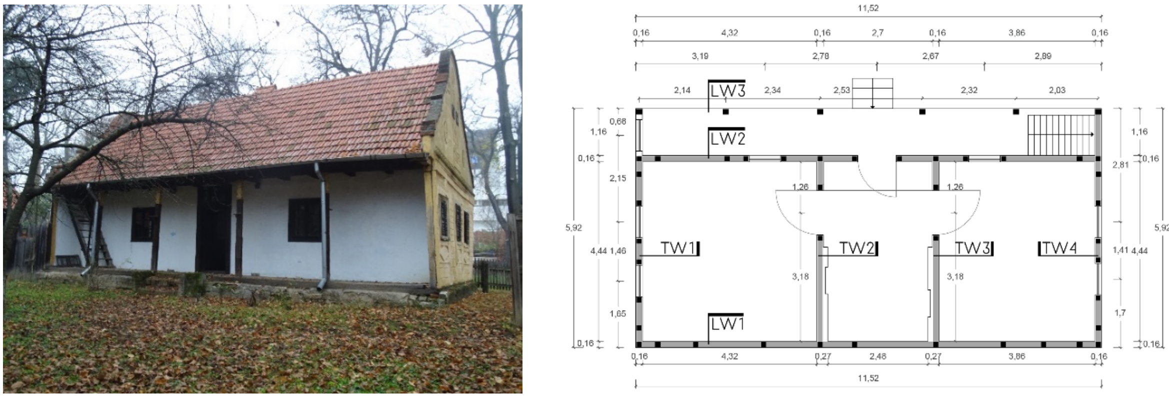
Although Romanian TFM structures have experienced several earthquakes, their performance is still not properly assessed and neither design nor retrofitting guidelines, are provided by Romanian building codes. The present paper aims at evaluating the dynamic response of a representative Romanian TFM building by applying a simplified modelling strategy consisting of an equivalent frame approach with non-linear springs lumped at the connections. This strategy is reliable since it is based on the initial numerical calibration of the in-plane cyclic behaviour of Romanian masonry panel with the outcomes resulting from the experimental tests in [5], [6], and it is feasible in terms of computational effort.

In this work, the Romanian TFM system is briefly presented by describing the geometrical and structural characteristics of a representative case study [7]. A building is modelled in the software OpenSees by adapting a calibrated mixed panel whose properties are discussed by comparing its local and global response with the experimental results [8]. The building dynamic properties were predicted by eigenvalue analysis and compared to those estimated by ambient vibration tests [7]. A dynamic study and potential calibration was performed as well [8].

2. DESCRIPTION OF A REPRESENTATIVE ROMANIAN TFM BUILDING

The Romanian timber-framed masonry architecture studied in this paper can be defined as a half-timber masonry structure due to the participation of the wooden skeleton to the resisting system not only under dynamic actions but also under static loads [9], [10]. Although the architectural and structural features can vary from structure to structure, this system is characterized in terms of geometry, structural elements, construction details as well as state of conservation by describing a representative building in the Sarbova area, Timis county, which was dismantled and rebuilt in the National Village Museum, Bucharest [7].

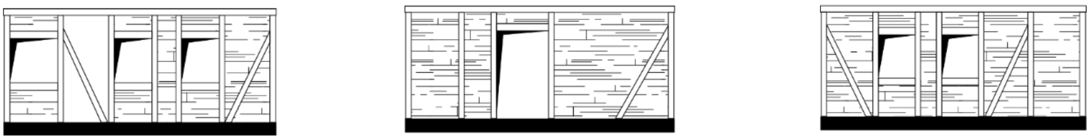
The building was a residential house built around 1900-1930 with the typical dimensions of Romanian TFM architecture: one-storey height and rectangular in shape (11.50x6.00 m), but not completely symmetric in plan with respect to the longitudinal and transversal axis [7]. Figure **1** shows the front view and the plan configuration re-elaborated from the drawings by Technical University of Civil Engineering of Bucharest.



(a) (b)

Figure 1: Romanian TFM building: front view (a) and plan configuration (b) [7]

The superstructure is usually raised from the ground to avoid moisture related issues and it is supported by a dry-stacked stone masonry platform or continuous wall-footing with lower beams named “soles” made of hardwood (oak or acacia) which distribute the loads uniformly. Since the building presents timber-framed panels with brick masonry infill, it can be classified as Type 1 according to [4]. This configuration can have bracings not perfectly aligned with the diagonal of each module, and, in this case, they are not restraint by two consecutive vertical posts, Figure 2 and Figure 3. It is worth to stress that diagonal bracings are effective only in compression since they detach from the frame in tension due to the nailed connections. Moreover, they are confined by masonry infill although it can detach from the timber skeleton due to the low adhesion at the interface resulting in potential out-of-plane mechanisms [5], [6]. During the field investigation on TFM buildings across the Vrancea seismic area in [4], a variation of brick masonry bond was also observed due to construction reasons and to increase the friction in the upper part of the wall as well as the stiffness [5]. However, this feature cannot be seen in the case study since a layer of plaster is applied on both sides of the wall to cover the timber skeleton.



(a) (b) (c)

Figure 2: Elevation of TW1 (a), TW2 and TW3 (b), TW4 (c) wall [8]

(a) (b)

Figure 3: Elevation of LW1 (a) and LW2 (b) wall [8]

The horizontal diaphragms consist of timber beams supporting wooden planks or wattle and daub, or sometimes reed and an additional layer of mud plus straw [5] In the case study, primary beams (cross-sectional area about 110x140 mm) are aligned along the transversal direction and transmit loads to the secondary beams on top of each longitudinal wall, Figure

4a. The roofing system is made up of timber as well. The most common configurations are hip or gable roof with king post trusses with no struts [5]. The studied building presents a porch type with loads transmitted to the longitudinal wall LW1 and LW3, Figure 1b. It consists of rafters with a cross-section of around 90x120 mm and a spacing ranging from 90 to 102 cm which support longitudinal timber strips (45x30 mm) nailed to them and applied to fix the upper ceramic tiles, Figure 4b. Diagonal bracings can be also observed in the fields of the roof to increase their in-plane stiffness while the chimney made of clay brick masonry mass may influence the global response of the building due to its non-symmetric location and heavy mass, Figure 4b.


(a)


(b)

Figure 4: Primary beams with timber planks (a) and roofing system (b) [7]

The connections between timber elements are similar to those observed during the field investigation already mentioned [4]. The inferior stringers are likely to be laid inside grooves in the continuous wall-footing to prevent any sliding. Vertical posts are driven into the lower beams with mortise and tenon joints, while the latter are half-lap splice type, Figure 5a and b, respectively. Lintels are likely to be simply connected with nails to vertical posts as well as diagonals that sometimes were applied as a propping system during the construction process resulting in fast and unprocessed connections [5]. Cross-halved joints can be observed between primary and secondary beams of the horizontal diaphragm, Figure 5c, but also between rafters and diagonals on the roof fields.


(a)


(b)


(c)

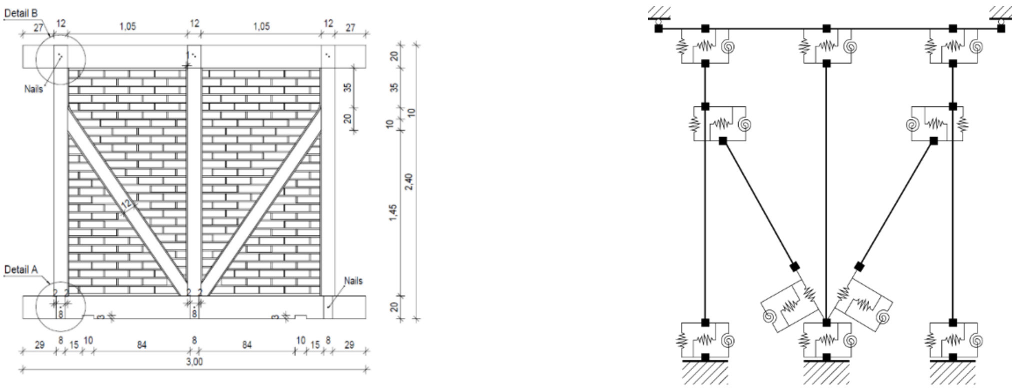
Figure 5: Mortise and tenon (a), half-lap splice (b) and cross-halved connection [7]

Most of the existing TFM buildings analysed in [4] were in poor conditions due to moisture related problems leading to irreversible decay of timber elements. Since the case study is poorly maintained, it presents the same issues such as biological colonization, coloration, rising damp, minor cracks due to stress concentrations, different construction phases or excessive shrinkage of infill. These cracks can result in partial detachment of masonry infill also due to the different physical and mechanical properties between the two materials.

3. ROMANIAN TFM WALL CALIBRATION

Although the global response of Romanian TFM buildings is influenced by the previously described structural elements and their connections, the shear walls play an important role and their behaviour was investigated in an experimental campaign, detailed presented in [5], [6], that performed cyclic tests on four mixed panels varying the type of infill and arrangement of diagonals. The results of the timber-framed wall with masonry infill and lower bracings in Figure 6a, S1 in [5], [6], are taken as a reference for the following numerical calibration.

A simplified strategy was used by modelling the panel through an equivalent frame with the non-linear behaviours lumped at the connections, and the model was implemented in OpenSees [11] as proposed in [8], [12]. Figure 6b shows the frame schematization consisting in linear elastic timber elements with translational and rotational springs at their connections. Posts, beams and diagonals are modelled as linear elastic elements since no damage was observed along them during the testing sequence. The choice of neglecting the masonry infill can be validated by the low adhesion at the interface with the timber skeleton, its low mechanical properties and by the presence of lower bracings that mainly control the deformation capacity of the wall [5], [6]. However, the potential underestimation in terms of stiffness due to the confinement effect for both elements and their connections, was considered by modifying the related parameters after the initial calibration of connections based on unconfined configurations. A similar updating procedure was performed for the cumulative dissipated energy that may result lower as well, due to the friction between frame and infill and along the joints. It was increased by making the hysteresis models of each connection larger with a steep slope of the unloading stiffness branch.



(a) (b)

Figure 6: Specimen S1in [5] (a) and its structural scheme (b) [8]

The pure shear behaviour was investigated by restraining the vertical displacements of the upper beam to simulate the pantograph system of the reaction frame, Figure 6b. Moreover, the top beam can be defined as rigid element to make the entire wall move with the same horizontal displacements while bracings are truss elements. It is worth mentioning that the latter ones can slide vertically at the upper connections as observed during the experimental test up to around 65 mm and they can detach at the central lower joint when the tension exceeds the withdrawal capacity of the slant nailing. The mechanical properties of Romanian fir (parallel modulus of elasticity equal to 8.9 GPa, specific weight of 385 kg/ m^3) and those of masonry infill (specific weight of 19.6 kN/m^3) were assumed consistent with experimental tests performed on similar materials [13]. The difference in total mass between numerical and experimental wall is negligible (less than 0.5%)

3.1 Calibration of connections

Since the global behaviour of TFM walls is mostly controlled by the connections, tee-halved (TH) type at the top and mortise and tenon (MT) type at the bottom, they were calibrated according to representative experimental tests [13], [14], respectively. These campaigns were aimed the study of the moment resisting behaviour and tensile capacity of the joints, but present some differences in terms of geometrical dimensions, types of fasteners and timber species.

Each structural joint was modelled with three springs, one per degree of freedom and aligned according to global or local axis as shown in Figure 6b, considering a 2D model. They are characterized by uniaxial non-linear hysteretic materials such as *SAWS* (developed by Patxi Uriz and converted from FORTRAN, code originally written by Bryan Folz) and *Pinching4* in OpenSees library whose properties resulting from the initial calibration, were iteratively modified to approach the local and global behaviour observed during the experimental tests [5], [6]. Moreover, the central lower connection between post and bracing was characterized by a spring aligned along the diagonal with an *Elastic-No Tension* uniaxial material that allows detachment in tension and prevents overlapping in compression [8]. The vertical spring at the upper joints was modelled to let the bracings slide along the external posts up to the maximum value (around 65 mm) observed during the test.

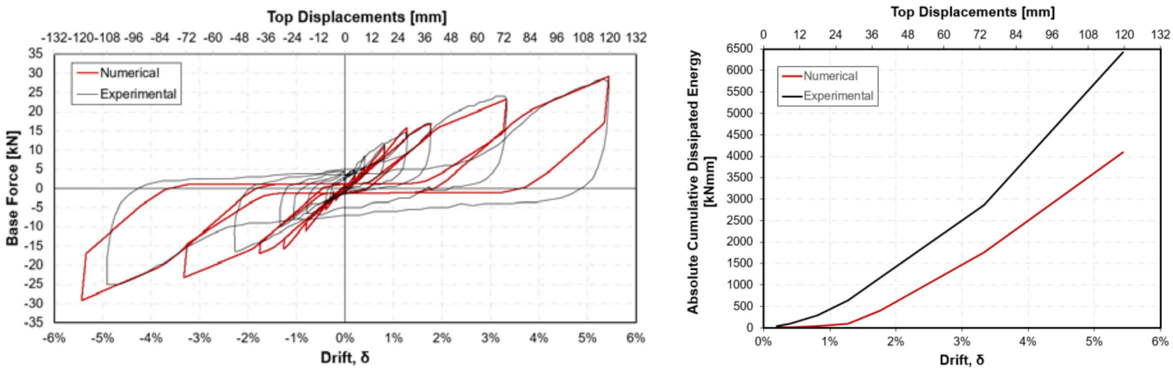
3.2 Non-linear cyclic analysis

The cyclic response of the simplified numerical model was calibrated with the inverse fitting procedure to approach the local and global behavior of the tested specimen in terms of hysteretic curve, its envelope, the damage mechanisms and the deformed shape. This iterative method consisted in updating the non-linear properties of hysteretic materials at the connections. Non-linear cyclic analysis was performed after pushover analysis to consider the accumulation of damage and degradation effects related to cyclic loadings.

The experimental curve was approached initially in terms of global envelope and then in detail, approximating the reloading and unloading behavior per cycle considering the degradation properties of the non-linear hysteretic materials applied at the connections [8]. Indeed, the wall response is mainly influenced by the moment resisting behavior of the tee-halved and mortise and tenon joint, as well as the sliding effect between diagonals and external posts. Thus, some parameters of their non-linear materials were updated increasing the initial and yielding stiffness from the initial calibration based on representative experimental tests [13], [14]. This modification can be justified by the additional contribution of masonry infill that confines each connection and stiffens the panel. The hysteretic curve resulting from the non-linear cyclic analysis, in red, is overlapped with the experimental one in Figure 7a showing a good approximation between them in terms of maximum base shear per displacement-cycle peak and reloading stiffness even though there are some discrepancies for the dissipated energy due to the different unloading stiffness and pinching effect. The latter phenomenon relates with the experimental reloading branches that are significantly pinched because the cyclic loading makes the joints weaker with the formation of gaps around the nails due to wood fiber crushing.

It is worth mention that the numerical analysis was performed by applying the same horizontal top displacement vector in positive and negative direction. This assumption eventually resulted in larger differences when the loading reverses in terms of negative peak displacements, shear forces and dissipated energy. The energy calculation is also affected by the non-symmetric behavior in the transition between positive and negative cycles of the experimental hysteretic curve. Figure 7b shows the comparison in terms of absolute cumulative dissipated energy: the numerical curve is always below the experimental one.

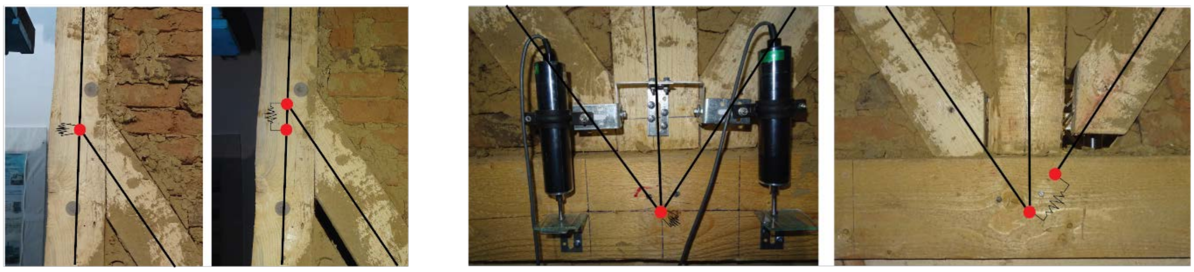
The absolute cumulative difference is not perfectly comparable (-36%) even for the maximum drift (5.43%) due to the already mentioned pinching effect and the vertical unloading of the experimental cycles. The different behavior should be investigated in detail to address the causes leading to the change of unloading stiffness in the numerical model. Regarding the lower stiffness of the simplified model, it is due to the initial expert assumption to consider as a reference the third cycle neglecting the first ones mostly influenced by the specimen setting and equipment testing.



(a) (b)

Figure 7: Experimental and numerical hysteretic curve (a) and absolute cumulative dissipated energy (b) [8]

The equivalent frame model shows the same deformed shape of the tested wall with the expected behavior of diagonal bracings. The damage mechanisms are comparable as well. Figure 8a pictures the vertical sliding along the external post whose values (63 mm) are consistent with those observed experimentally (around 65 mm), while the axial detachment at the base is lower than the numerical one (88-107 mm), Figure 8b. This may result from neglecting the withdrawal capacity of the slant nailing and some inaccuracies in the calibration of the upper connections bracing-to-post.



(a) (b)

Figure 8: Vertical sliding along the post (a) and detachment along the diagonal axis (b): initial and final position

4. SEISMIC MODELLING OF THE REPRESENTATIVE ROMANIAN TFM BUILDING

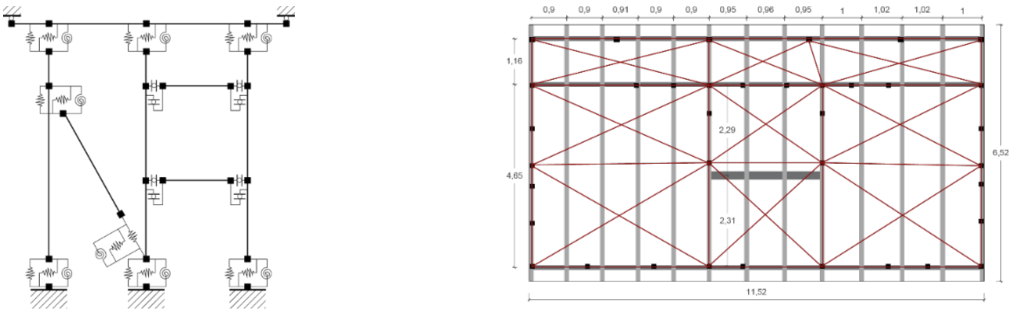
The seismic behavior of the representative Romanian TFM building in the National Village Museum, Bucharest, was predicted by applying the same simplified modeling strategy to compare its dynamic properties with those estimated by ambient vibration tests [7].

Since the global response is influenced by many parameters such as geometry, mass distribution, configuration of structural elements (TFM walls, floor and roofing system) as well as the connections between them and state of conservation, additional expert assumptions and simplifications were made to evaluate the building structural performance. Indeed, the case study presents some differences with the calibrated wall of Section 3 as following:

i. infill made of clay bricks instead of mud bricks;
ii. bracings are not always constraint by two consecutive posts;
iii. presence of openings with related timber lintels;
iv. flexible floor;
v. large variation in mass distribution.

Thus, the representative structural scheme of each wall as well as its boundary conditions were slightly updated as shown in Figure 9a. It is worth mentioning that the set of connections resulting from non-linear static analysis of the wall specimen was applied since the properties of the non-linear hysteretic materials at the springs were updated again to perform non-linear cyclic analysis considering the degradation effects. Moreover, timber lintels are pinned at their ends to the vertical posts, while non-constraint bracings have same connections both at the top and bottom assuming equivalent potential vertical or horizontal sliding, respectively.

The lower nodes are fixed to the ground since neither differential foundation settlements, nor tipping ones, were observed during the field investigation due to the light weight of the structure and its continuous wall-footing. No horizontal sliding was considered as well, but this assumption may not be conservative if the lower beam is not connected to the continuous brick masonry wall footing. Since the confinement and stiffening effect of the masonry infill was considered in the calibration, out-of-plane behavior was neglected as well, even though this mechanism is very likely to occur in case of earthquake due to the low adhesion at the interface between timber frame and infill [5], [6]. Figure 9b shows the horizontal diaphragm modelled as rigid by applying a tying system with high axial stiffness. Vertical uplift was prevented, to be consistent with the wall calibration performed, considering its pure shear behavior.



(a) (b)

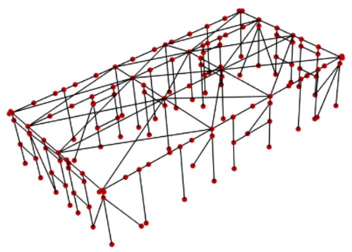
Figure 9: Structural scheme of walls (a) and floor (b)

An increased specific weight of masonry infill was defined as 20 kN/m^3 for clay brick instead of 19.6 kN/m^3 for mud brick in S1 specimen in [13] and was distributed along the vertical posts considering their tributary area to simulate the actual mass distribution of each wall. Since timber beams are made of hardwood (oak or acacia), their specific weight is assumed as 600 kg/m^3 and their parallel modulus of elasticity equal to 12 GPa.

Regarding the additional weight of horizontal diaphragm and roofing system, it was concentrated at the structural nodes representing the connections between the transversal primary beams and the secondary ones on top of each longitudinal wall LW1, LW2 and LW3 (Figure 1b) according to their tributary areas. The same procedure was applied to determine vertical loads from the roof trusses considering the actual tributary area of the roof field, not its horizontal projection, due to the steep slope. No horizontal thrust was considered since the roofing system approaches a truss structure with inclined rafters tied at the base by the transversal beams of the floor diaphragm. The presence of the brick masonry chimney and attic walls was considered as lumped weight at its contour posts. Only dead loads were applied to be consistent with the environmental and loading conditions during the dynamic identification. The total weight of numerical model was compared with the one determined by hand calculations resulting in a negligible difference lower than 5%.

The dynamic properties of the simplified model were determined by eigenvalue analysis and compared with those resulting from the dynamic identification. This procedure and the testing equipment are explained in detail in [7], but, in this paper, only the resulting frequencies and mode shapes are presented. Since the calibration procedure was not based on updating neither the non-linear properties of the already calibrated connections, nor the linear material properties due to the state of conservation, it was performed by modifying the elastic stiffness of a fictitious material representing the additional contribution of clay brick masonry infill.

Two simplified models were made to evaluate the differences between two complementary modeling strategies: equivalent frame with non-linear properties lumped at the connections (EF) and the same model infilled with shell elements (EFI), Figure 10a and b. The inverse fitting procedure was performed by modifying the Young's Modulus (E) of infill until the estimated frequencies were obtained. The choice of considering the contribution of a fictitious infill is validated by the higher mechanical properties of clay brick masonry compared to those of mud masonry applied in the wall specimen [8]. Since ambient vibration tests record micro-tremors of structures in their elastic range and the initial stiffness of TFM buildings is mainly controlled by the characteristics of masonry infill, EF model may result in larger differences in terms of frequencies compared to the estimated ones.


(a)

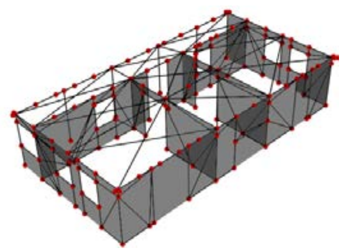
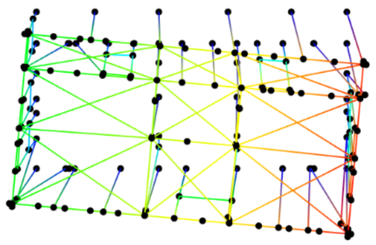

(b)

Figure 10: EF (a) and EFI (b) models

The calibration resulted in the final value of *E* equal to 58.5 MPa, very low compared to the one related to clay brick masonry (1.2 ÷ 1.8 GPa according to Table C8.5.I in [15]). The value is almost negligible since the numerical model for the wall was already calibrated with the presence of masonry infill. Bearing in mind that the first periods are 0.17 s for the transversal direction and 0.15 s for the longitudinal one and since there are no mode shapes in the already mentioned report [7], the first and second periods with the corresponding mode shapes are compared between EF and EFI models, Table 1, Figure 11 and Figure 12. The periods of EF model are higher than the estimated ones in [7] and the corresponding mode shapes show slight torsional effects due to non-symmetric layout and mass distribution. This effect is even more evident for EFI model where both periods are comparable to those in [7]. Moreover, in EF model, periods and corresponding mode shapes are actually flipped; the highest period is actually longitudinal not transversal and vice versa.

Table 1: Transversal mode shape for EF (a) and EFI (b) models

Model	Mode	Type	Period [s]	Difference [%]	Reference
EF	1	Longitudinal	0.374	220	0.17s (Transversal)
EFI	1	Torsional-Transversal	0.17	-	0.17s (Transversal)
EF	2	Transversal	0.333	222	0.15s (Longitudinal)
EFI	2	Torsional-Longitudinal	0.154	2.6	0.15s (Longitudinal)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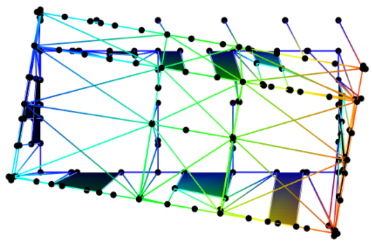
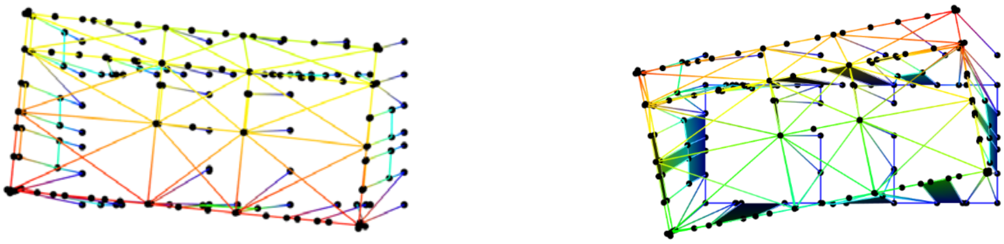

(b)

Figure 11: Transversal mode shape for EF (a) and EFI (b) models



(a) (b)

Figure 12: Longitudinal mode shape for EF (a) and EFI (b) models

5. FINAL CONSIDERATIONS

The structural typology with timber skeleton with masonry infill (TFM) has already proven the effectiveness of its seismic performance empirically improved by local builders, but this seismic capacity is still not assessed quantitatively. This research work is aimed at evaluating the seismic behavior of Romanian TFM buildings following the previous studies and existing experimental campaigns in [4]-[7], [10], [13] which determined the non-linear hysteretic response of TFM walls subjected to in-plane cyclic loading.

A representative Romanian TFM building was modelled in the software OpenSees as an equivalent frame structure with non-linear properties lumped at the connections (EF). This model is based on a numerical calibration of the wall specimen S1 tested in [5] by applying the inverse fitting procedure to approach both global and local response [8].

Some additional assumptions were considered to model the Romanian building and estimate its dynamic properties, which were compared to those resulting from ambient-vibration tests. A potential calibration was also proposed by infilling the panel (EFI model) due to the different type of masonry observed in the existing structure, even though the actual objective is to use the most simplified modelling approach to predict the seismic behaviour of Romanian TFM architecture.

The resulting deformed mode shapes of both models showed a significant torsion effect due to non-symmetric mass distribution and layout which should be properly addressed with future analysis. It is worth to mention that the dynamic properties of the equivalent frame model with no infill (EF) are influenced by the connection calibration that presents some limitation of the applied hysteretic materials in the correlation between applied loads and global response, but the wall calibration was performed with a representative vertical load.

The simplified modelling strategy has other limitations such as it does not take into account the interface between the timber frame and masonry infill, but its contribution in terms of dissipated energy was considered in the calibration of connections. In addition, since the masonry infill detaches from the timber frame even in the curing process, the out-of-plane mechanisms are not prevented especially in case of earthquakes, thus a further implementation of the numerical model is required. However, this approach can still provide reliable results, especially for the wall, and allows to capture local failures of connections if the demand exceeds their deformation capacity. The calibration process may even run automatically if the user understands the role of each parameter and connection. Regarding non-linear numerical analysis on simplified models, they may not require a large computational effort thus their application can be feasible to whole buildings.

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