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# Comparison of seismic models for a historic masonry building

# Comparación de modelos sísmicos para un edificio histórico en mampostería

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**Abstract**

In this project, a historic masonry building in Sardinia, Italy has been considered as a case study for the comparison of two approaches for modeling, static and seismic analysis. Two software with different modeling approaches were employed with the purpose of comparing and discussing the results. SismiCad12 was used to simulate the structural behavior of the historic masonry building. SismiCad12 uses the Finite Element Method (FEM) that allows to model and analyze most types of 3D structures, and it is suitable for masonry structures. On the other hand, a different and innovative modeling approach called Frame Macro Elements (FME) was also applied using the 3Muri software, specially designed for assessing the linear, nonlinear, and seismic behavior of masonry structures.

Assuming the same hypothesis to construct the 3D model of the structure in each code, the results of the static analysis show a different distribution of the vertical loads in the structure, which are more realistic in the FEM modeling. This different criterion of evaluation of the vertical loads carries a mechanism of "soft floor" in the pushover analysis in the FEM modeling, and therefore, a lower ultimate displacement corresponding to the collapse of the structure. On the other hand, in dynamic analyzes, FME modeling is more receptive to reality, involving a massive percentage of masses participating in the first vibration modes.

**Keywords:** macro-elements; historic structures; pushover analysis.

**Resumen**

En este proyecto se ha considerado como caso de estudio un edificio histórico de mampostería en Cerdeña, Italia, cuyo modelado, análisis estático y sísmico han sido desarrollados. En este estudio se han empleado dos programas informáticos con distintos enfoques de modelización con el propósito de comparar y discutir los resultados. Como primer simulador del comportamiento de la estructura en mampostería se ha elegido SismiCad12, que es una suite de Elementos Finitos (FEM, por sus siglas en inglés) que permite modelar y analizar la mayoría de los tipos de estructuras 3D y es adecuado para estructuras de mampostería. Por otro lado, se ha aplicado un método de modelado diferente e innovador denominado Frame Macro Elements (FME) con el *software* 3Muri, diseñado específicamente para evaluar el comportamiento lineal, no lineal y sísmico de las estructuras en mampostería.

Suponiendo la misma hipótesis para construir el modelo 3D de la estructura en cada código, los resultados de los análisis estáticos muestran una diferente repartición de las cargas verticales en la estructura, las que son más realísticas en el modelado FEM. Este diferente criterio de evaluación de las cargas verticales lleva un mecanismo de “piso suave”

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en el análisis de *pushover* en el modelado FEM, y por lo tanto un desplazamiento ultimo inferior correspondiente al colapso de la estructura. Por lo contrario, en los análisis dinámicos el modelado FME es más receptivo a la realidad, involucrando un porcentaje masivo de masas participantes ya en los primeros modos de vibración.

**Palabras clave:** macroelementos; estructuras históricas; análisis de *pushove*.

## 1. Introduction

The problem of the safety of existing masonry buildings is a matter of fundamental importance worldwide due to their high vulnerability to seismic actions [1], [2], [3], [4] and the high historical, artistic and economic value of the existing building heritage [5], [6]. Currently, one of the fundamental problems when considering a masonry building is to implement an efficient and reliable modeling strategy that takes into account the main features of the materials in use, the mutual link between bearing walls of the structure and the layering due to building history [7], [8], [9].

In this study, two commercial software were chosen to address this type of problem: 3Muri by S.T.A. Data S.r.l. and Sismicad12 by Concrete S.r.l. Static and seismic checks at the Ultimate Limits were done following the European Committee for Standardization (CEN) guidelines, Eurocode 6 (EC 6) [10] and Eurocode 8 (EC 8) [11] with these software. 3Muri uses the so-called Frame by Macro Elements (FME) approach [12], [13], specifically tailored and therefore, more specific, whereas Sismicad12 uses a general-purpose approach, the Finite Element Method (FEM) [14], [15], [16] which implies a more significant computational burden. Both codes allow linear and non-linear analysis with the Pushover Method provided by EC 8, as well as Italian national norms with the so-called N-Method. Plasticity is in both cases molded in concentrated form at the ends of the elements or macro elements.

The building used as a case study (Figure 1) is a historic masonry building located in the historical downtown of the city of Sassari, Italy, dated back to the middle of the nineteenth century, inserted into an aggregate context of historical buildings with similar characteristics and belonging to the same historical period. It is a very articulated structure that develops on four levels, characterized by massive irregularities of mass and stiffness in plan and height and with a large internal cavity that guarantees the illumination of the central part of the building. The building faces streets on two sides while the remaining walls are adjacent to the contiguous buildings, one of which has been recently reconstructed, and therefore constitutes a structural unit of its own.

A discussion is needed considering that often professionals of the sector consider the results of a single calculation software, which could lead to inappropriate results or significantly different results from those that would give another software. This work aims to evidence the criticalities that may arise in a seismic analysis of a geometrically complex case study due to different approaches proposed by two commercial software.

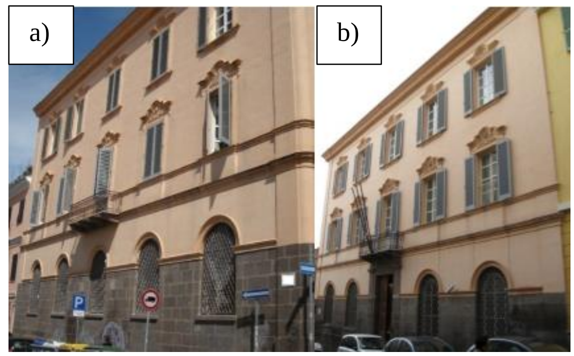


**Figure 1.** Exterior view of the historical masonry building in this case study: a) side façade; b) front façade.

## 2. Materials and methods

The wall structure is characterized by masonry panels of considerable thickness, varying between 0.60 and 1.00 m, and a story height ranging between 4.00 and 5.00 m. The first-level floors are almost exclusively made of barrel vaults, in perforated brick blocks; above the vaults is a loose or slightly loose filling. There are frenels on which a horizontal bricklayer rests, on which the screed, the substrate, and the paving itself weight. In the next two levels, the floors are constructed through a system of Roman Vaults (a pavilion vault dissected from a horizontal plane). As far as the roof is concerned, this is a sloping roof that is divided into three parts, with the height of the grid plan almost always at 2 m, compared to the attic level, with three different peak heights. The top roof is made of traditional roof tiles. The wall structure is predominantly made up of soft stone slats (limestone) entrenched with cement mortar.

The 2008 Italian Technical Construction Regulations (NTCs, by its initials in Italian) were used as a regulatory reference for the analysis and verification of the results, which is consistent with the Eurocode (6 and 8), concerning seismic brickwork and in particular the section dedicated to existing buildings [17]. In the absence of specific survey campaigns, generally expensive and invasive, especially for historic buildings such as the one in question, the Italian legislation provides standard mechanical parameters for a certain number of historical wall types and correction coefficients that allow to take into account the characteristics of detail, level of knowledge of the structure and any past consolidation interventions [18]. As a result, the parameters adopted for the study are given in Table 1.

**Table 1.** Material properties [18].

| | $f_m$ (N cm$^{-2}$) | $\tau_0$ (N cm$^{-2}$) | E (N mm$^{-2}$) | G (N mm$^{-2}$) | W (kN cm$^{-3}$) |
|---|---|---|---|---|---|
| Standard | 140 | 2,8 | 900 | 300 | 16 |
| Adopted | 233,3 | 4,6 | 1500 | 500 | 16 |

Where $f_m$ is compression strength, $\tau_0$ is shear strength, $E$ is longitudinal elastic modulus, $G$ elastic cutting modulus, and $W$ specific masonry weight. From the standard values to the adopted values shown in Table 1 (named as "Masonry with soft stone bricks of limestone" the Ministerial Circular [18]) a reduction coefficient equal to 1,35 has been applied for poor level of knowledge of the material, and two amplification coefficients of 1,5 for mortars in good condition and thin joints (<10 mm) respectively. $W$ coefficients were not applied [18]. The project resistances will be elaborated by taking into account the γM security coefficient and a corrective factor correcting the level of knowledge gained in the said Confidence Factor. Considering load combination of the NTC called ultimate limit status (SLU, by its initials in Italian) the analysis of the floor loads is calculated (1):

$$Q = G_1\gamma_{1+} G_2\gamma_2 + G_3\gamma_3 = 11.95 kNm^{-2} \quad (1)$$

Where $G_1$ is the structural permanent load, $G_2$ is the non-structural permanent load, $G_3$ the variable load, with their respective amplification coefficients γ.

Once the necessary site visits and relief operations have been carried out, including some limited inspection tests aimed at the knowledge of the composition of the masonry and horizons, the two models have been developed with the two reference software. The level of knowledge that has been achieved has been very superficial (LC1) because of the inability to conduct an extensive and exhaustive investigation campaign. The reference seismic action was determined from the attribution of a 50-year Useful Reference Life (VN, by its initials in Italian) and a Use Class "II," that is a class of importance as compared to the functions and the potential level of crowding. The geographic location of the building in the Italian territory allowed to identify the seismic zone (seismic zone 4). Thus, the seismic baseline site hazards, represented in Italian law by the seismic parameters are reported in Table 2, regarding the States Life-Saving Limits (SLV, by its initials in Italian), Damage (SLD by its initials in Italian), and Operational (SLO, by its initials in Italian) Limit. Table 2 includes the probability values (PVR, by its initials in Italian) to exceed the seismic intensity in the reference period (VR, by its initials in Italian) assumed (which is 50 years), and allow to trace its elastic response spectrum in acceleration and displacement of the horizontal components of the earthquake, taking into account a correction for geotechnical and topographic features of the site under review.

Analysis were carried out in both software. Tests for Non-Seismic Load Scenarios, Linear Dynamic Spectrum Analysis were respectively done with a spectral response, as well as a Non-Linear Static Seismic Analysis (called Push-Over) [3], [19], [20].

**Table 2.** Seismic parameters used.

| | SLV ($PV_R$=10%) | SLD ($PV_R$=63%) | SLO ($PV_R$=81%) |
|---|---|---|---|
| $A_g$ (m s$^{-1}$) | 0.05 | 0.02 | 0.02 |
| $F_0$ | 2.88 | 2.67 | 2.61 |
| Tc*(s) | 0.34 | 0.30 | 0.27 |
| Tr (s) | 475 | 50.0 | 30.0 |
| Ss | 100 | 1.00 | 1.00 |
| Tb (s) | 0.11 | 0.10 | 0.09 |
| Tc (s) | 0.34 | 0.30 | 0.27 |
| Td (s) | 1.62 | 1.61 | 1.61 |

In the Linear Dynamic Spectrum Analysis, in accordance with the national regulations [18], the vibration modes taken into account had to involve a mass >5 % of the total mass of the structure, and the sum of the total mass involved in the vibration modes considered had to be at least 85 % of the total mass of the structure. The effects of individual modes have been combined based on the complete quadratic combination (CQC). For linear analysis, particularly for the dynamic, the Sismicad12 code allows both frame modeling and modeling with 4-node 3D shell elements.

The push-over analysis according to the N-method consists of applying two separate horizontal force systems distributed at each level of construction, proportional to the forces of inertia and having a bottom shear ($F_b$) result. Such force systems are concomitant with permanent vertical loads. Such force systems are concomitant with permanent vertical loads. These forces are applied in the $X$ direction and $Y$ direction separately and incremented monotonically step by step until the local collapse of the individual structural or global elements is reached to form a mechanism following the formation of a number of plastic hinges. Global verification is carried out by moving, monitoring the maximum displacement ($d_c$) of a control point of the structure, generally coinciding with the last-level mass center. The $F_b$-$d_c$ diagrams plotted for the different scenarios represent the corresponding structure's capacity curves.

## 3. Results and discussion

### 3.1. Out-of-plane for non-seismic actions check

#### 3.1.1. Sismicad12

For the FEM calculation, a 3D shell element modeling with six degrees of freedom (dof) per node was adopted. Thus, assigning a maximum mesh size of 400x400 mm and obtaining a mathematical model with 23,903 nodes and 24,220 elements (Figure 2).

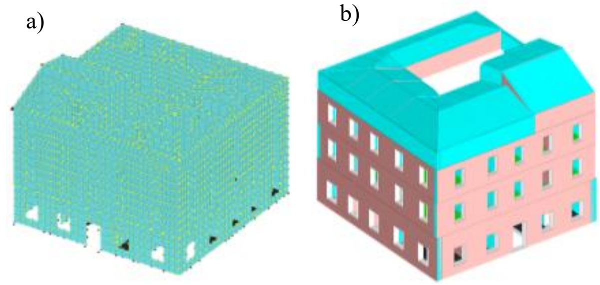


**Figure 2.** Sismicad12 simulation overview: a) FEM model; b) 3D model. **Source:** Own elaboration.

The tensile framework obtained from the analysis is integrated into the mesh sections corresponding to the wall panels automatically by the post-processor in order to perform the checks on the panels provided by EC 6, EC 8 and NTC. At this stage, the primary SLU resistance tests are those off-plane compression carried out according to the so-called □ method (EC 6 and NTC), by defining structural and conventional eccentricities. The result of verifications in non-seismic scenarios highlights some aspects of the structure, particularly on the first-level masonry columns and the fourth-level perimeter wall (shown in red in Figure 3), with projected stresses exceeding 40 % the resistances.

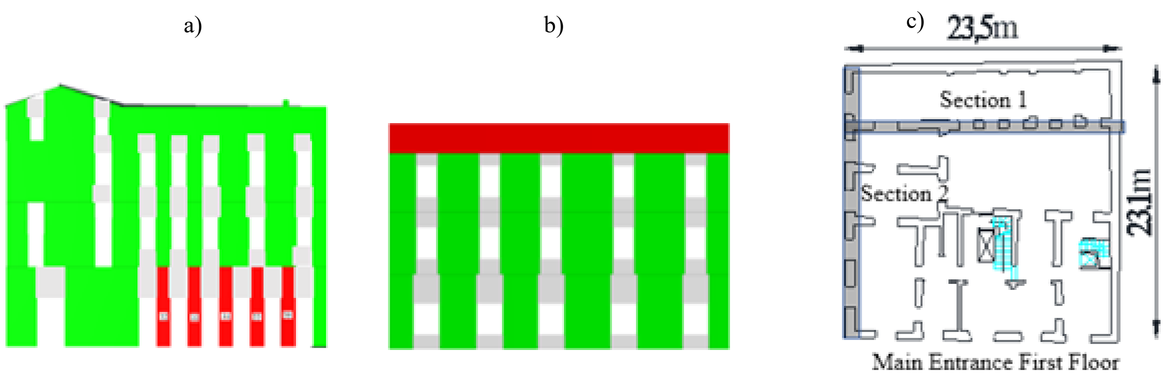


**Figure 3.** Results of the review of the most critical sections: a) Section 1; b) Section 2; c) First floor.

#### 3.1.2. 3Muri

In this case, modeling is performed on the effects of all types of analysis with macro elements, and the model is characterized by only 197 knots and 359 macro elements (Figure 4).

Following the checks at the SLU, this structure is now verified under vertical loads with some minor issues spread in the first level. The same columns of the first level shown in Figure 3, even in 3Muri are not verified. However, the projected stresses exceed the resistance by just 9 %, in this case.

### 3.2. Linear Dynamic analysis with response spectrum

Both software allow to calculate the project response spectrum automatically by introducing the input data previously calculated and then determine the stresses for each mode.

a) b)

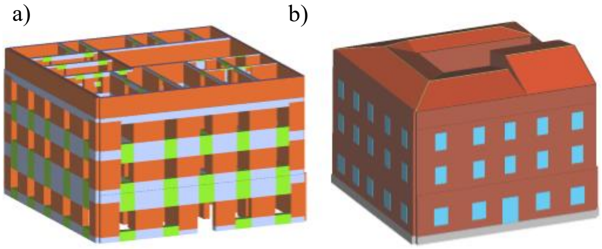

**Figure 4.** 3Muri simulation overview: a) FEM model; b) 3D model. **Source:** Own elaboration.

### 3.2.1. Sismicad12

The linear dynamic analysis was carried out by the Sismicad12 program as defined by the NTC. A careful analysis considering a number of vibrational modes equal to 22, either in the X direction or in the Y direction, to involve a participating mass ≥ 85 % is required by the regulations. The masses in Z direction are not taken into account by the software. Table 3 shows only the most significant values of the masses, concentrated between the vibration modes 19 and 22.

**Table 3.** Results of Sismicad12 simulation.

| Mode | Period (s) | Mass X (%) | Mass Y (%) |
|---|---|---|---|
| 19 | 0.36 | 15.7 | 0.02 |
| 20 | 0.29 | 0.53 | 24.9 |
| 21 | 0.21 | 63.6 | 1.67 |
| 22 | 0.19 | 0.95 | 56.8 |

In this case, the rocking and shrinkage tests in the plan for seismic action are taken into account in conjunction with vertical loads. The first level elements highlighted in red in Figure 5 are unverified.

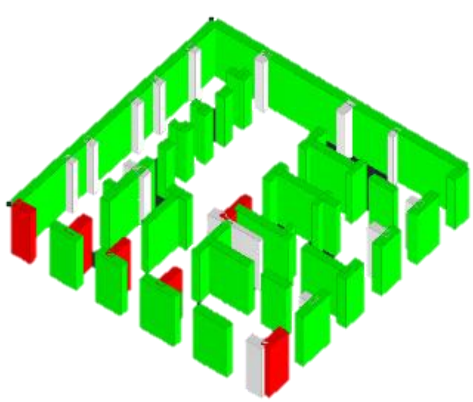

**Figure 5.** First-level verification views.

Regarding the white masonry piers, they do not meet the geometric requirements required by the regulations. Therefore, they do not contribute to the resistance of the structure to seismic actions. Similar results have been found in other levels.

### 3.2.2. 3Muri

The linear dynamic analysis was carried out by the 3Muri program as per the NTC. Given the simplicity of modeling and the smaller number of GDLs in this case, it is satisfactory to consider a number of vibrational modes equal to 3 in the X direction and 2 in the Y direction to involve a participating mass ≥ 85 %. However, it is required by the NTC to consider modes to stimulate at least 5 % of the participating mass. Analyzing the results, it can be seen here that the structure fully satisfies the press-reflection and cut-off checks in the plan for seismic action (Table 4).

**Table 4.** Results of 3Muri simulation.

| Mode | Period (s) | Mass X (%) | Mass Y (%) | Mass Z (%) |
|---|---|---|---|---|
| 1 | 0.3558 | 76.99 | 3.61 | 0 |
| 2 | 161.73 | 4.81 | 81.9 | 0 |
| 3 | 216.17 | 6.43 | 1.54 | 0 |
| 12 | 0.0989 | 0.02 | 0.07 | 13.4 |
| 13 | 0.0974 | 0.05 | 0 | 63.0 |
| 17 | 0.0884 | 0.07 | 0 | 7.40 |

## 3.3. Nonlinear static analysis (push-over)

### 3.3.1. Sismicad12

In this case, the code of the software converts the mathematical model into an equivalent frame pattern, according to the Salonikios et al. [21] with concentrated plastic hinges. The elements adopted are common beam elements with a formulation that takes into account the contribution of the shear deformation, connected by rigid bracts representing the nodal intersection zones of the males and planes, within which no deformations occur. The most critical capacitance curves obtained according to the approach described, it is, in the X and Y directions, are shown in Figure 6. The total number of combinations, and therefore, the capacity curves taken into account is 8.

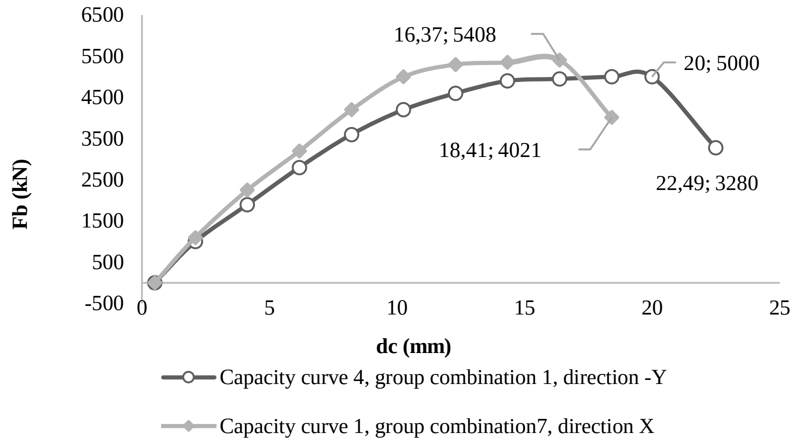


**Figure 6.** Sismicad12 capacity curve.

### 3.3.2. 3Muri

In this case, the most critical capacitance curves in X and Y directions are shown in Figure 7.

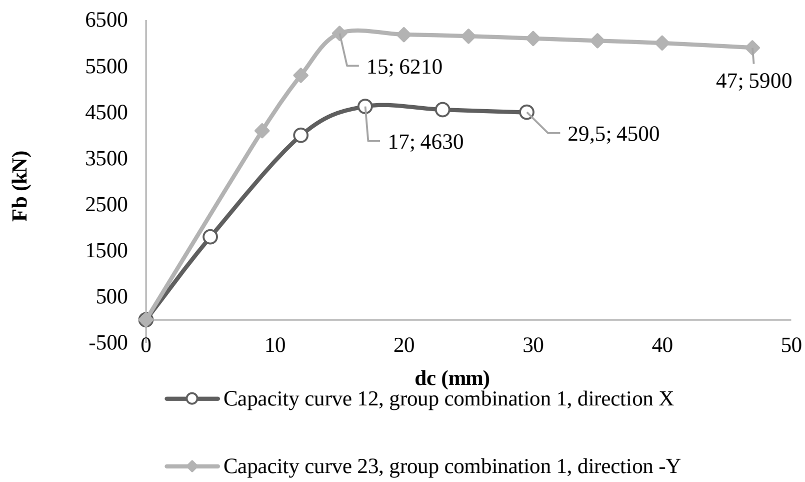


**Figure 7.** 3Muri capacity curve

## 3.4. Comparison of results

Regarding the static tests carried out with the two models and summarized in the previous paragraphs, there is some consistency between the unverified elements, although the soliciting actions appear to be in some cases markedly different. It has been observed that in relation to the distribution of vertical loads, the pertinence of the individual elements is processed differently by the algorithms of the two software. In any case, the similarity between the results of the checks is probably because in the case of study, and as it is the case of historical buildings, the elements are considerably oversized over the loads. Conversely, if the building had been dimensioned in a more optimized way and closer to the verification limits, there would undoubtedly be more apparent differences between the static tests of the two models.

By analyzing the results of the dynamic analysis carried out with both models (Tables 3 and 4), it can be seen that the vibrational modes along the X and Y directions have very different periods and masses of participants, which results reasonably because of the disparity of the structure. In Sismicad12 the modes of vibration 19 and 21 in the X direction, as well as 20 and 22 in the Y direction involve enormous participant masses compared to the remaining vibration modes. In fact, the first 18 modes are local and involve single walls, which are not very significant from a global point of view. This is because roof tiles have been interpreted as deformable and, therefore, do not constitute an adequate retention for the walls of the top floor. This assumption was derived from the analysis of the characteristics of the hedging structure, and it is not of a general nature. However, these vibration modes, while having a mass less than 5 %, contribute to 85 % of the participating mass, and as envisaged by the legislation cannot be neglected, therefore, they must be taken into account for the determination of the seismic action. With 3Muri software instead, in the first three modes of vibration, both in the X direction and Y direction, there are enormous participant masses that exceed the 85 % of the standard. The different modeling style proposed by the two codes in terms of approach and complexity is, therefore, a source of significant differences in dynamic analysis. In light of the above, it seems inappropriate to compare the modes of vibration in the order of calculation but may be more appropriate a comparison of the modes that involve more significant participating masses, which, as seen for both software, focuses on 2 or 3 modes. In the case of 3Muri all the checks are verified, but in the case of Sismicad12, there are several negative tests, especially with shear in the plane. The difference in modeling approach, on the other hand, also affects mass distribution, for which a lumped approach is adopted, due to the different number of nodes offered by the two models.

By comparing the push-over analysis performed with the two models, considering the same control point, the last shift and the number of steps, it was noted that in the FEM approach of Sismicad12, 16 capacity curves are provided, of which 8 for group 1 and 8 for group 2 and considering directions X and Y, and eccentricity along X and Y.

In FME modeling, instead, 24 capacity curves are provided, in addition to the previous, eight curves do not take into account the general eccentricity provided by the standards. As indicated by the NTCs, seismic action must be applied for each direction, in both possible directions and the most unfavorable effects of the two analysis should be considered. Comparing the heaviest capacity curves presented by the two software, both in direction -Y, one can see a marked difference in behavior (Figure 8). While in the initial part of the case study curve, there is a substantial equality (indicative of a similar representation of elastic stiffness), there is a significant difference in the evaluation of the maximum cut in the base of the last displacement and the decay of the capacity curve same.

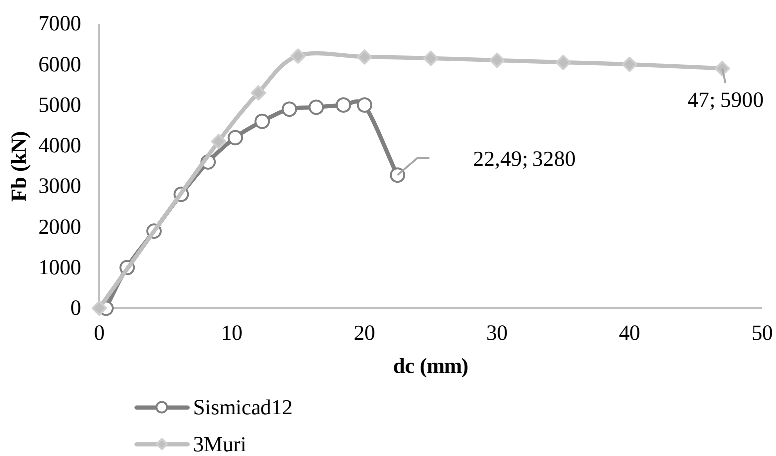


**Figure 8.** Comparison of capacity curves similarly loaded.

The reason lies in the fact that Sismicad12 modeling shows the existence of a soft floor mechanism (Figure 9), with the plasticization focused on the first level, causing a rapid decay of the overall structural ductility to achieve cut-off deformations in the plane of the wall elements involved, while the walls of the overlying planes do not appear to be particularly affected by the boost.

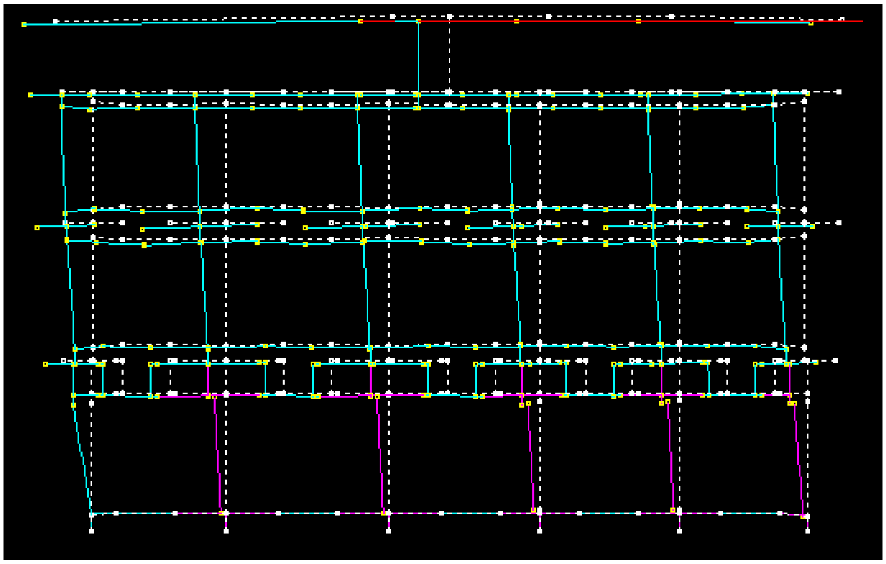

**Figure 9.** Soft floor mechanism of Sismicad12 in a representative section.

By analyzing the modeling performed by 3Muri, it can be observed that the mechanism that generates a decay of the structure capacity under the seismic shear action at the base (Figure 10), results in the achievement of local buckling mechanisms and the last relative displacement with contemporary shear plasticization of several male wales, but rarely reaching the conventional breakdown or displacement provided by the NTC.

A key role in such a marked difference in results is thought to be possible by the already discussed mode of vertical computing actions that is a markedly different between the two software. The normal agent action dramatically influences the behavior under horizontal actions of the individual murals, and therefore of the overall behavior, to the extent that the higher the vertical loads are, as it is generally the case in the software 3Muri, the higher will be the shear and compression resistance of the males, in the field of low stresses. Viceversa, where it is lesser, as in the Sismicad12 software, there is a predominant shear breakdown concerning the buckling action.

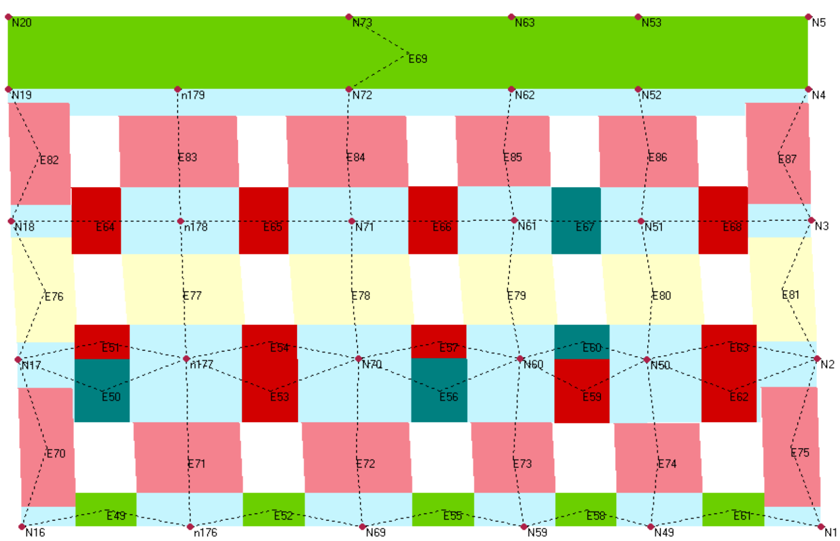


**Figure 10.** Decay of the structure capacity of 3Muri in a representative section.

## 4. Conclusions

The two modeling approaches lead to very different results, both qualitatively and quantitatively. Static checks reveal substantial differences in the evaluation of vertical action acting on individual wall elements due to a marked difference between the two algorithms. Such checks seem to be more in line with reality if done with the model to the finite elements, in fact, this model allows to take into account the effects of the mutual link, the mutual collaboration between walls of the box, and redistribution of stresses in a concrete way easily guessed that finds real practical consistency.

As far as modal analysis is concerned, there are reservations about the FEM method because the first 18 vibration modes have little significance since they are initially considered to be the single walls of the last level. It would seem, moreover, that the FME model provides results that are more responsive to reality, involving a very massive percentage of participants already in the first modes of vibration, as would have been expected.
In the push-over analysis, the appearance of the soft floor mechanism in Sismicad12 and the different evaluation of the vertical loads between the two software lead to qualitatively and quantitatively different results, especially regarding last shift.

Lastly, it is concluded that the modeling of existing masonry buildings is particularly complex and burdensome. Commercial software provides reasonable approximations of the actual behavior of the structures, but it is crucial to have a high level of knowledge of the structure, to know the modeling types adopted, to recognize its limits and to understand the results. Therefore, a comprehensive awareness and caution by the operator is needed, necessary to understand and better define the output results of the software itself.

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