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MODELLING THE IN-PLANE BEHAVIOUR OF A MASONRY FAÇADE VIA A MULTI-UNIT DISCRETIZATION WITH SIMPLIFIED INTERFACE LAYOUT

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Abstract

Based on a benchmark test, this paper discusses the validation process of a new numerical modelling procedure developed for simulating the behaviour of periodic masonry structures. The peculiarity of the analysed MUDis (Multi-Unit Discretization) procedure stands on the possibility to discretize any masonry walls with periodic arrangement using a limited number of repeated modules covering more than a single masonry unit, thus greatly reducing the inherent sources of nonlinearity and minimizing the computational effort. The modules are made of four linear elastic polygonal units separated by nonlinear interface elements whose pre-established layout allows to reproduce all the typical in-plane collapse mechanisms of masonry walls subjected to lateral loads. The mechanical parameters of the interface elements are described through the "combined cracking-shearing-crushing" model proposed by Lourenço & Rots and widely used in the FEM-based simplified micromodeling of masonry structures. Three of these parameters have been suitably modified by the Authors through parametric formulas in order to adapt the original constitutive model to the MUDis modelling. The reliability of the procedure, already validated for two-dimensional panels, is here evaluated by resorting to the experimental data of a real masonry façade for which results are available in the literature.

Keywords: Masonry modelling; Seismic vulnerability; Nonlinear analysis; Masonry buildings; pushover analysis; FEM analysis.

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1 INTRODUCTION

Masonry structures, which represent a considerable part of the historical architectural heritage worldwide, are particularly vulnerable to destructive actions such as seismic events [1]. Thus, one of the main research topics in structural engineering concerns the analysis of masonry buildings under different loading conditions in order to better understand their structural behaviour and the possibility of developing damage [2, 3].

Advanced structural analyses are a fundamental tool for investigating the response that masonry structures might exhibit against seismic loads, as well as the possible failure mechanisms that they might undergo under this type of actions. Such analyses are a necessary step to determine the most suitable retrofitting strategies to ensure structural and occupant safety, both in the prevention and emergency phases.

Due to the complexity of masonry mechanics, to simulate the in-plane behaviour of unreinforced masonry (URM) structures under seismic loading, different modelling approaches at distinct scales of representation have been proposed over the years, ranging from very simplified and quick analyses, up to detailed and complex models. The choice of the modelling strategy is an important aspect that affects both the accuracy of the results and the computational effort; hence the selection is not trivial. Generally, this selection is carried out according to the complexity of the structure to be analysed, but also to the level of detail to be achieved and to the type of outcomes required.

Among the different strategies proposed in literature that one based on the finite element method (FEM) covered a wide space. Within FEM-based approaches, a distinction can be made between block-based models and continuum homogeneous models, as proposed in D'Altri et al [4]. The first category is certainly the one that provides higher modelling detail as it is based on the reproduction of the wall texture explicitly, unit by unit. Within this category there are, for example, detailed micro-models that involve the accurate reproduction of all the elements that constitute the masonry (unit material, mortar and all interface surfaces) [5, 6] or even simplified models where masonry is considered as a two-phase material, in which the units are like expanded bricks, while the mortar and its characteristics are included in zero-thickness joints, such as the simplified micro-model proposed by Lourenço and Rots [7]. The second category, instead, includes approaches that model masonry structures as continuous deformable bodies made of a homogeneous material, without distinction between blocks, mortar layers or interfaces. The constitutive law adopted for the material can be defined through calibration [8, 9] or multi-scale homogenization procedures [10].

This classification is not intended to be exhaustive. Indeed, other types of modelling approaches in literature exist, which are halfway between the two afore-mentioned macro categories. For instance, there are simplified models that start from the idea of discretizing the masonry, as it happens in micro-models, but by merging macro areas into larger portions and using different types of linear elements separated by non-linear elements. One of the most common examples in this regard is the AEM model [11] which is based on the discretization of the masonry as a set of rigid elements connected by non-linear spring interfaces where failure can occur. This modelling technique laid the foundations for other recently developed models such as the one proposed by Malomo and DeJong [12].

The modelling approach discussed in this paper, named MUDis (Multi-Units Discretization) approach, belongs to this last group, aiming at discretizing masonry through modules of pre-established geometry and shape capable of accurately reproducing the main mechanisms of failure. This methodology has been already validated by the authors for the simulation of the in-plane behaviour of masonry panels of different aspect ratios [13, 14] while the validation for more complex structures is still in progress.

In the current paper, a step forward is made by applying the proposed discretization procedure to a masonry wall with a central door, which represents a compulsory yet challenging case considering that openings can adversely affect the in-plane behaviour of URM walls. A further degree of complexity is here introduced due to the necessity to correctly modelling both piers and spandrels resulting from the presence of the door opening. In fact, as it is already known in literature, spandrels play an important role in the seismic response of masonry buildings as they influence the behaviour of the entire structure by coupling the response of masonry piers, thus influencing their boundary conditions, lateral capacity, and cracks propagation [15, 16, 17]. Particularly, the in-plane capacity of masonry walls with openings depends on the spandrels: their stiffness, strength, and deformation capacity and also the way they interact with the piers [18].

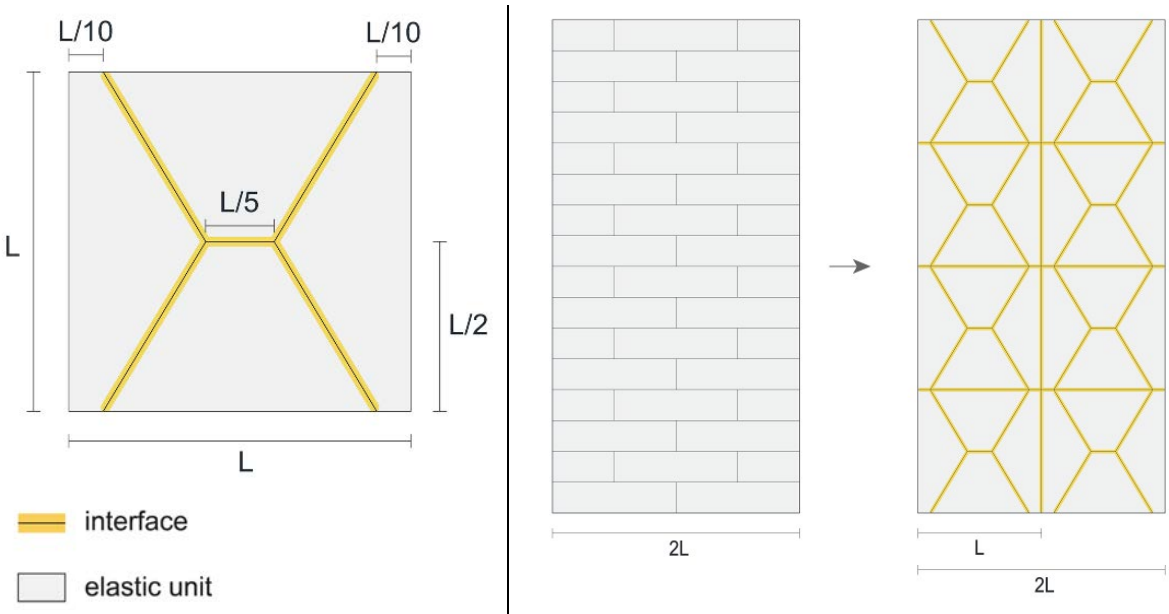
Three different types of spandrel-pier coupling are commonly distinguished [19]. The first type, representative of new buildings with rigid floor diaphragms, provides reinforced and very resistant spandrels, capable of activating an effective shear and bending behaviour like beam elements. In this case, a perfect coupling between males and bands can be assumed, and the piers result the weakest elements that collapse first. The second type is typical of old buildings characterised by flexible floors that provide, at the most, coupled horizontal displacements for vertical elements. In this case, the piers are assimilated to cantilever beams fixed at the base and are decoupled from the spandrels which, lacking any tensile resistant element, act as truss elements capable of resisting only axial forces, resulting therefore the weakest element of the wall and collapsing first [20]. The third type of spandrel-pier coupling is also found in ancient buildings, but those which have effectively anchored horizontal tensile stress-resistant elements. In this situation, spandrels enable to prevent decoupling with piers [21]. These limit cases provide an idea of the schematic behaviour of masonry walls with openings in their own plane, but they cannot embrace all possible non-linear responses, hence the behaviour of the structure must be studied exhaustively case-by-case [22].

These above considerations highlight the importance of a full understanding of the masonry behaviour in the presence of openings, and particularly of the measure up to which the position, type, and size of spandrels as well as elements that resist to horizontal tension, e.g., lintels, can influence the in-plane response of the entire wall. Errors in the definition of the type of pier-spandrel coupling can lead to errors in the definition of their boundary conditions and, consequently, to unrealistic predictions of the global response [22] and to the design of incorrect retrofitting measures.

Within this research framework, it was deemed necessary to further validate the proposed MUDis procedure accounting for the presence of openings in periodic masonry walls in order to ensure that the calculation model could adequately catch the real behaviour of the spandrels, on which in turn depends the response of the entire structure. To this end, a benchmark experimental test involving a regular masonry wall with a single central opening is reproduced. The selected structure allowed investigating in detail each modelling aspect starting from different hypotheses up to reaching a close simulation of the experimental outcome.

2 THE PROPOSED MUDIS PROCEDURE

The objective of the proposed MUDis procedure is to discretize the masonry walls through repeatable modules, each covering more than a single unit of masonry. This is done to considerably simplify the numerical representation of periodic walls allowing to reproduce with good approximation the typical in-plane failure mechanisms of masonry under horizontal and vertical loading but without incurring into burdensome analyses. Unlike micro-modelling strategies, in MUDis the masonry constituent materials are not modelled separately. In the bidimensional case, the basic module is a square composed of four polygons separated by interface surfaces as shown in Figure 1(a). The particular shape and internal division of the modules enable, once they are assembled, to catch the most common crack patterns of masonry walls subjected to lateral loads, including crushing, tensile cracking and shear sliding along the joints, diagonal cracking, and the simultaneous occurrence of diagonal cracking and sliding. An illustrative example of the module assemblage to represent a wall with aspect ratio 2:1 is shown in Figure 1(b).



(a) (b)

Figure 1: The MUDis procedure: (a) elementary module; (b) example of module assemblage (adapted from [14] and [23]).

The four polygonal units describe the behaviour of the brick/stone elements that compose the masonry wall, being therefore represented through linear elastic materials featuring case-specific values of Young's modulus and Poisson's ratio. The units are separated by nonlinear zero-thickness interfaces representative of the behaviour of both mortar joints and mortar-brick interface surfaces, where all basic types of masonry failure phenomena can occur. The constitutive model adopted to describe the interface behaviour in MUDis is the composite interface (CI) model formulated by Lourenço and Rots [7]. This model, also known as “combined crack-shearing-crushing model”, was conceived for simplified micro-modelling strategies, thus resulting inaccurate for a multi-unit discretization, given both the reduced number and different position of units and failure surfaces. This led to the definition of transformation relationships in order to modify some of the key mechanical parameters that define the CI formulation and make it suitable for a multi-unit approach. Detailed information about this calibration process can be found in [14].

3 SIMULATION OF THE IN-PLANE RESPONSE OF A FULL-SCALE URM WALL WITH OPENING

The proposed procedure has been thoroughly validated for two-dimensional masonry panels [14] and preliminary analyses have been implemented to assess its performance in case of full-scale masonry walls [23]. Currently, additional numerical investigations are being carried out to extend the procedure to more complex structures and 3D analyses, also by introducing relevant factors. As explained before, in the present work, a masonry wall provided with a symmetric door opening is used as a case study aiming at assessing the MUDis capability of correctly reproducing the behaviour of the spandrels, hence the entire response of the wall, under in-plane actions. The results obtained from these initial analyses are widely discussed in the following.

3.1 Description of the experimental benchmark

The structure selected for the investigation is a non-reinforced masonry wall with a central opening experimentally tested by Parisi et al. in 2012 [24].The specimen geometry and test setup were designed to foster the truss behaviour of the spandrel and to develop most of the damage exactly within the region above the opening. Such aspects made this experimental test an ideal benchmark for evaluating the suitability and reliability of the MUDis procedure for the aforementioned purposes.

The specimen consisted of a solid 0.31 m thick single-leaf masonry wall made of yellow tuff stones and hydraulic mortar composed of natural sand and pozzolana-like reactive aggregates. Overall, the specimen was 5.10 m long, 3.62 m high and it was composed of a central spandrel bounded by two piers, as schematized in Figure 3a. To support the flexural behaviour of the spandrel, a wooden lintel was inserted across the top of the door opening with an anchorage length of 150 mm in both piers. Furthermore, rigid steel beams were positioned both at the base to connect the structure to the ground, and at the top of the piers to ensure uniform loading distribution and prevent stress concentrations.

The experimental tests were conducted in two conditions: as built and pre-damaged. The as-built specimen was tested under monotonically increasing displacements: first, a vertical force of 200 kN was applied onto the piers; then, displacement-controlled horizontal loading was applied under constant vertical forces. Once damage occurred, the monotonic test was interrupted, and a cyclic test was conducted on the pre-damaged specimen until near collapse.

All tests were performed according to a quasi-static displacement-controlled loading protocol to adequately investigate the nonlinear behaviour of the wall at increasing deformation demand. The results showed an extensive crack pattern localised at the spandrel, as was expected. Result obtained from the experimental test are illustrated in Figure 2.

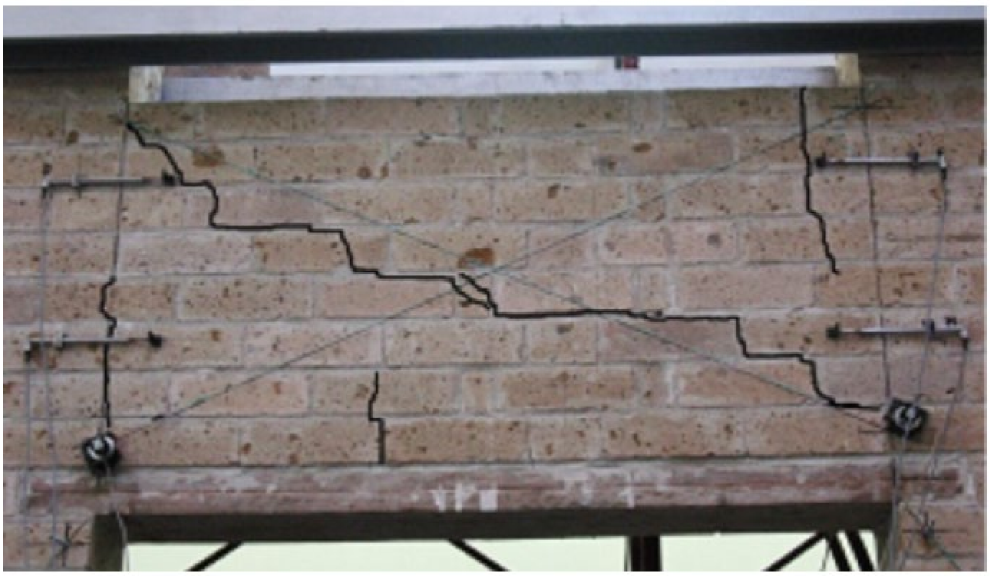


Figure 2: Crack pattern of the experimental test considered. From [24].

3.2 Numerical modelling of the wall

The numerical simulation of the experimental benchmark was carried out in the nonlinear software MIDAS-FEA [25] which allows to consider the "combined cracking-shearing-crushing" model for the interface elements. Given the geometrical dimensions of the specimen, squared modules of 250 mm side length were adopted to discretize the entire masonry wall (Figure 3b). The selection of this length derived from a sensitivity analysis.

To model the elastic units, six-node triangular shell elements with three Gaussian integration points were used, whereas two-nodes zero-thickness elements with a general formulation were adopted for the interfaces. The latter were placed both among the four units composing the modules and along their outer boundaries in order to simulate the interaction between adjacent modules. Furthermore, to prevent vertical sliding between modules, a rigid constraint was imposed in the vertical direction along these boundaries. Finally, with the aim of reliably reproducing the test boundary conditions, the numerical model was fixed at the bottom by simulating the experimental rigid beam placed at the base, and another rigid beam element was modelled at the top of the piers for the same reason.

To investigate the in-plane behaviour of the structure following the experimental loading protocol, a unidirectional pushover analysis was performed. Accordingly, the load was applied in two phases: vertical loads were statically imposed in the first phase; then, an increasing lateral displacement was applied monotonically to the top of the façade in the second phase. The numerical analyses were carried out resorting to the Newton-Raphson method and using an energy control criterion for the first phase and a displacement control criterion for the second phase.

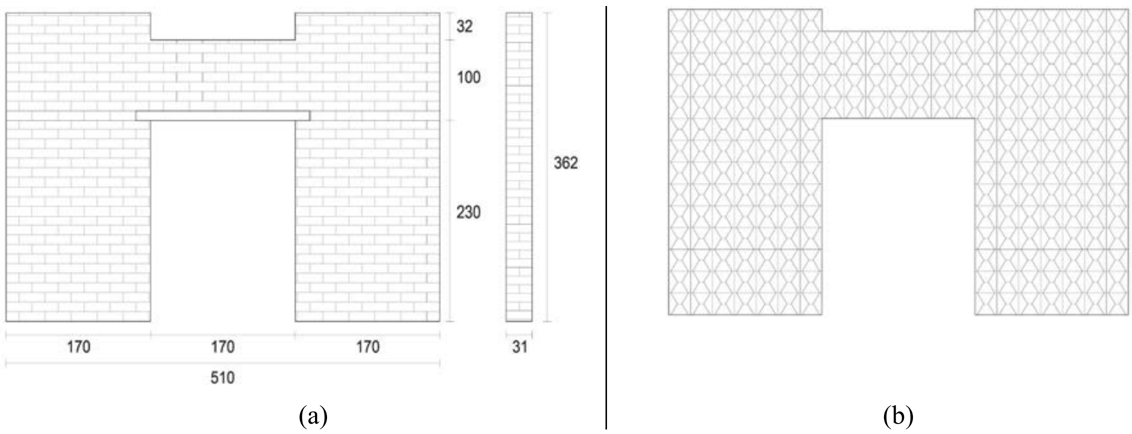


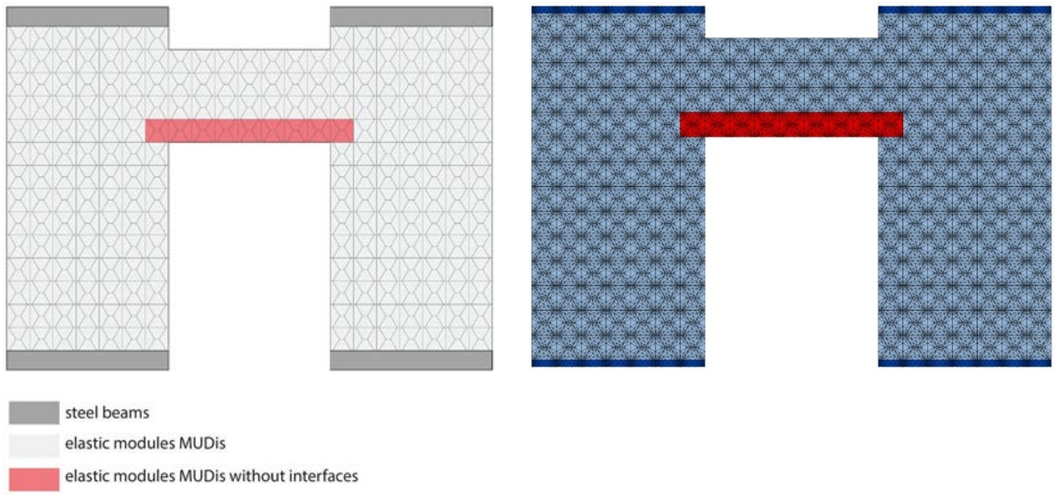
Figure 3: (a) Geometry of the benchmark case study (front and lateral view), adapted from [24]; (b) multi-unit discretisation of the wall with modules of 250 mm edge.

3.3 Modelling hypotheses for the spandrel

As previously highlighted, position and size of the openings can soundly influence the overall response and in-plane capacity of masonry walls. It follows that the elements between openings must be adequately modelled to avoid errors in the numerical predictions. Concerning the present case, a wooden lintel is placed above the window, below the spandrel, with an anchorage length of about 150 mm on both sides, as previously mentioned. In accordance with the state of the art, this type of lintel leads to the case of strong pier-weak spandrel masonry wall, where the spandrel indeed collapses first.

Several numerical tests were therefore carried out to understand which modelling assumption could be the most suitable for reproducing the observed behaviour of the specimen and get the exact collapse mechanism of the spandrel. This calibration procedure involved distinct analyses with three lintel idealizations:

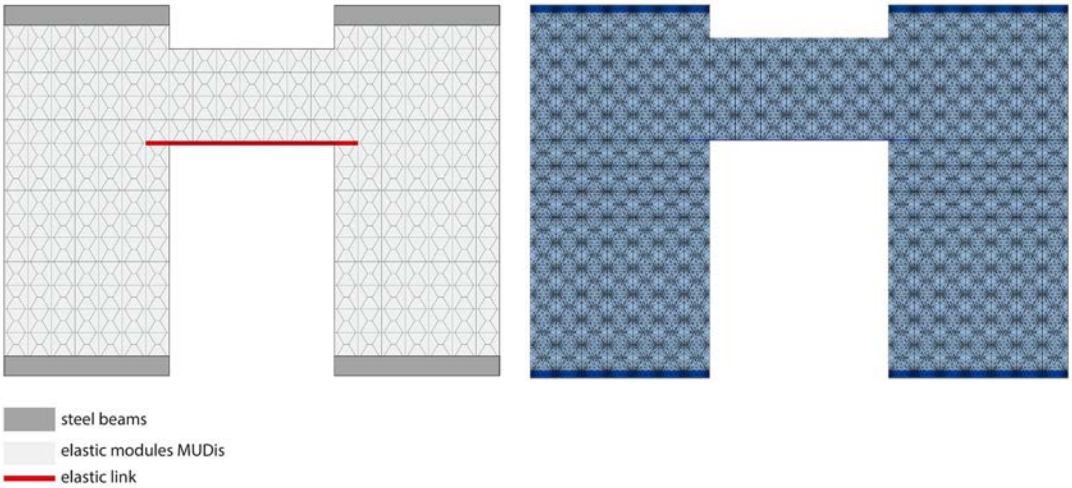
- Hypothesis 1: the lintel is modelled implicitly by assigning MUDis modules to the region where this element is located. The entire wall, lintel included, is therefore discretized with modules of the same size (250 mm side length). The difference between the masonry region (shown in light colour in Figure 4) and the lintel area (shown in red in the same figure) lies in the attribution of diverse elastic parameters between masonry and timber. In addition, to ensure that the lintel area remains in the elastic field, no interface element is placed between the modules. This way, one can numerically simulate the presence of a continuous and elastic element which performs the function of a lintel. However, the main limitation of this hypothesis lies in the size itself of the module: by choosing to discretize the entire wall with equal modules of 250 mm, the height of the lintel ultimately exceeds the real one.



(a) (b)

Figure 4: First hypothesis of modelling: implicit modelling of the lintel. Numerical idealization (a) and software implementation (b) of the façade with the MUDis strategy.

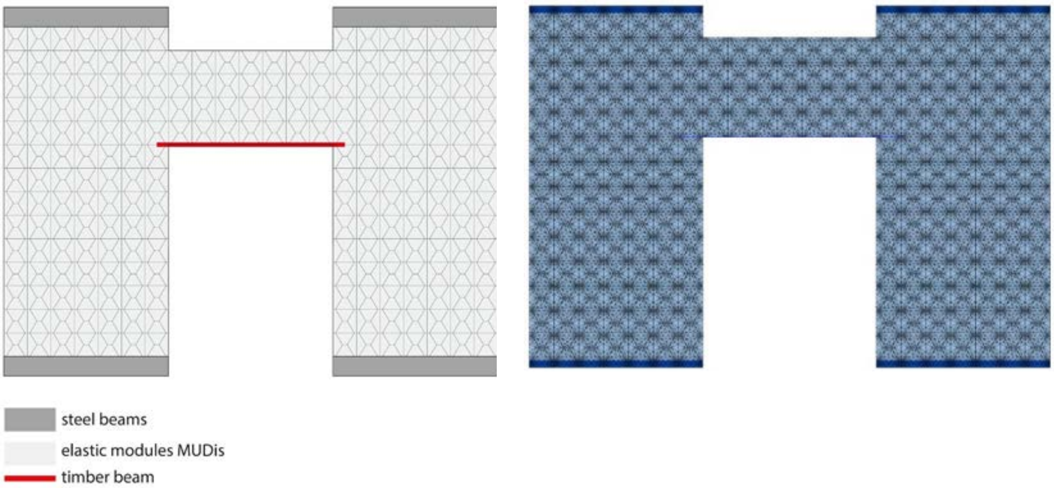
- Hypothesis 2: the lintel is simulated by inserting elastic links, namely connecting nodes two-by-two in order for them to act as a unique element with pre-assigned stiffness. In this way, all the nodes above the opening are constrained in the vertical direction to simulate the flexural strength of the real short wooden spandrel against gravitational loads. Graphic idealization and implementation in the software are shown in Figure 5.



(a) (b)

Figure 5: Second hypothesis of lintel modelling: elastic links. Numerical idealization (a) and software implementation (b) of the façade with the MUDis strategy.

- Hypothesis 3: the lintel is modelled through a beam element having the same cross section (310 mm x 6 mm) and length as the real lintel. The elastic mechanical properties assigned to the beam are the orthotropic characteristics of a low-strength class timber. This type of modelling, shown in Figure 6, is the most accurate among those proposed hitherto as it allows to faithfully reproduce all the characteristics of this element, even though great attention must be paid in the modelling phase in order to ensure the perfect merge between the mesh nodes of shell and beam and not to incur errors during the analysis.



(a) (b)

Figure 6: Third hypothesis of lintel modelling: timber beam. Numerical idealization (a) and software implementation (b) of the façade with the MUDis strategy.

4 RESULTS AND DISCUSSION

In Figure 7 the outcomes resulting from the pushover analyses of the benchmark masonry wall modelled through the MUDis approach are presented. The shear-displacement curves (continuous lines) obtained from all the three models (each corresponding to a different modelling assumption for the lintel) are plotted and compared. To measure the goodness of the results, the experimental curve (dashed line) provided by [24] is shown on the same graph and used as a comparative metric.

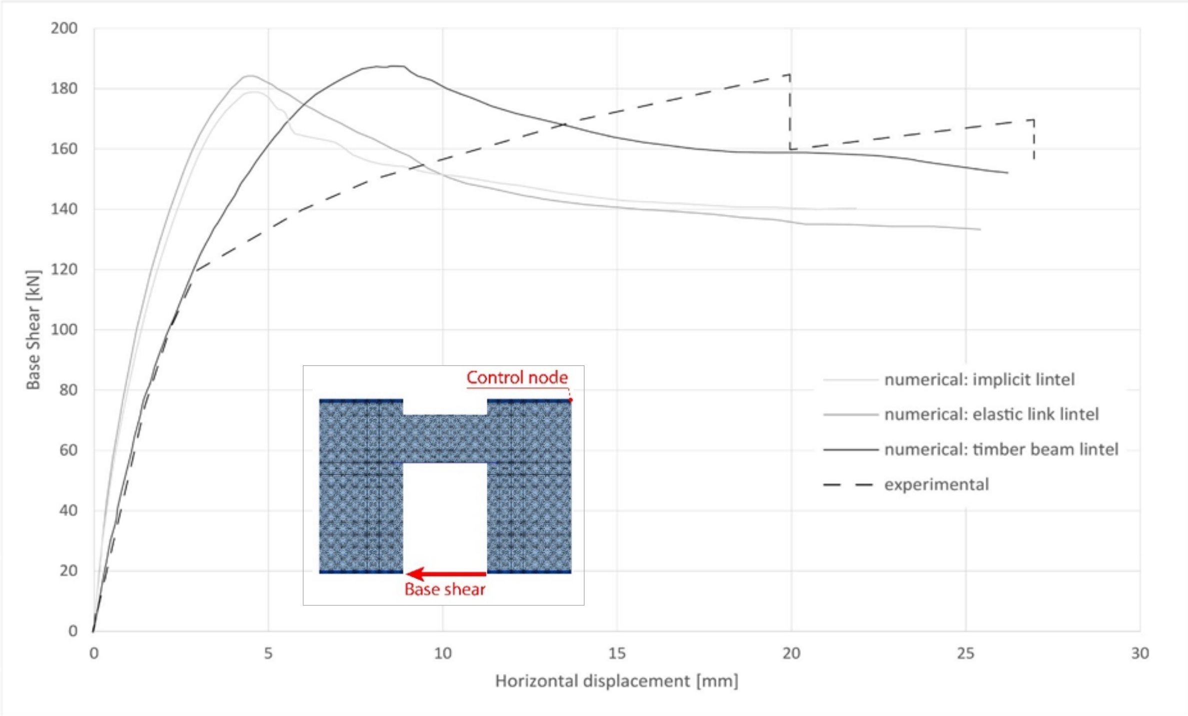


Figure 7: Experimental vs numerical results: comparison between load-displacement curves.

Observed (experimental) and predicted (numerical) crack patterns were also compared to weigh the accuracy and reliability of the MUDis procedure in simulating the collapse mechanism of the masonry wall as close as possible (Figure 8).

By interpreting the results, it is found that:

- For the first modelling hypothesis, it can be seen that the numerical curve is not very far from the experimental one neither in terms of maximum resistance nor in terms of ultimate displacement capacity (Figure 7). However, the behaviour of the structure is completely different from the real one (Figure 8b). In fact, the piers collapse before the spandrel. This means that the implicit modelling of the lintel is not appropriate for the considered structure because it stiffens the area above the door opening too much. This is essentially due to the size of the module, which inevitably leads to a lintel with greater dimensions in proportion to the size of the spandrel. Likely, in case of larger structures, where the module size would not affect that much the actual lintel size, this type of modelling could represent a valid alternative.

- Regarding the second modelling hypothesis for the lintel, similar results are obtained in terms of ultimate resistance and displacement capacity (Figure 7), but once again the final crack pattern is not comparable with the experimental evidence. As shown in Figure 8c, the failure mechanism of this model is again controlled by the collapse of the piers that occurs before the collapse of the band, as the latter is likely too stiff due to the presence of rigid links (even though they only constrain nodes in the vertical direction). This result could be improved by calibrating the elastic parameters of the link with a trial-and-error procedure until suitable values are found to simulate the actual deformation occurring in the timber lintel under vertical loads. However, besides giving problems in joining the nodes in correspondence of the interface elements, rigid links cannot be easily calibrated in the absence of an experimental counterpart regarding the base material of the lintel, thus resulting questionable for unsupervised numerical predictions.

- The last modelling is the one that gave the best results as it allowed to satisfactorily reproduce the overall in-plane response of the masonry wall in terms of maximum resistance and ultimate displacement capacity (Figure 7) as well as the collapse mechanism of the masonry (Figure 8d). In fact, this is the only case in which the spandrel collapses before the piers, as observed experimentally.

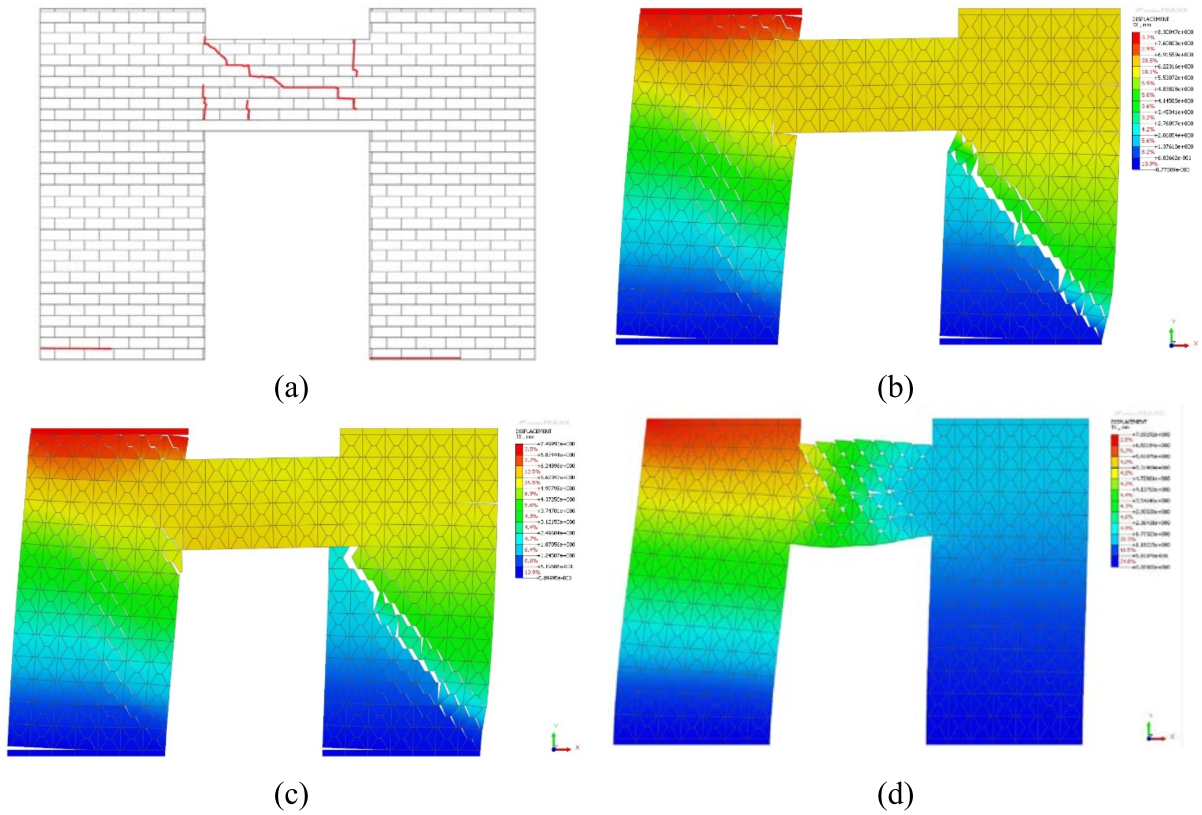


Figure 8: Experimental vs numerical crack pattern and ultimate displacement for the three models: (a) original specimen (adapted from [24]; (b) implicit modelling of the lintel; (c) lintel modelled with vertical rigid links; (d) lintel modelled through a beam element.

5 CONCLUSIONS

In this paper, a multi-unit discretization (MUDis) procedure, recently developed by the authors, was applied in order to assess its capability of correctly reproducing the in-plane behaviour of a masonry wall with a central door opening, which was tested by other authors and that has been considered as benchmark test. In fact, it is widely known that openings can adversely affect the in-plane capacity of URM walls, thus the adequate modelling of spandrels and piers is pivotal to obtain accurate predictions of the masonry behaviour against horizontal loadings.

Three different modelling assumptions are adopted and widely discussed. From the analyses carried out the following conclusions can be drawn:

- The MUDis modelling approach is truly effective in reproducing the lateral behaviour of the considered masonry walls, both in terms of structural response and of failure modes.

- To effectively reproduce the behaviour of the spandrel, it is necessary to simulate well the presence of the lintel. When applying the MUDis procedure, the best method to simulate the lintel is to model it explicitly with a beam element as close as possible to reality in terms of material and geometrical properties.
- Other modelling techniques might be applied for lintels, such as elastic links or implicit modelling by attributing different material characteristics between adjacent MUDis modules, but they are not equally effective and accurate as the explicit modelling nor quick to perform.

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