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(Article begins on next page)

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3 **NONLINEAR MODELING OF THE SEISMIC RESPONSE OF MASONRY**  
4 **STRUCTURES: CRITICAL REVIEW AND OPEN ISSUES TOWARDS**  
5 **ENGINEERING PRACTICE**

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### Abstract

33 This paper provides a comprehensive review of the critical aspects of nonlinear modeling for evaluating the  
34 seismic response of masonry structures, emphasizing the issues relevant to engineering practice. Currently, the  
35 specialized technical community shares the opinion that, for a performance-based approach, numerical models  
36 are the only tools sufficiently effective to support the seismic assessment of existing buildings. However, their  
37 potential often falls short when attempting to accurately describe the behavior of masonry structures. In fact,  
38 these structures feature highly complex architectural configurations, different masonry types, and various  
39 structural solutions, meaning that extra care is required in numerical modeling. This is especially true when the  
40 modelers do not have a solid background in the software chosen and may not be practiced using the vast variety  
41 of options offered by the software houses. They are often unaware of the consequences that questionable

42 modeling choices may have on the results obtained by the models. These extremely complex topics are treated  
43 in the paper from an engineering practice perspective, providing an in-depth overview of the challenging issues  
44 related to the use of different modeling strategies. The paper covers strategies ranging from the Equivalent  
45 Frame approach (used widely in common engineering practice) to more refined techniques like 2D and 3D FE  
46 procedures based on continuous, discrete, and micro-mechanical approaches. Critical aspects in the modeling  
47 of both in- and out-of-plane responses of masonry, as well as the critical issues in wall-to-wall connections and  
48 diaphragm roles are investigated. All the examined issues are clarified through numerical examples highlighting  
49 also how a consistent and integrated use of different procedures may be beneficial. Finally, some of most  
50 relevant challenging issues concerning the use of numerical models in seismic assessments with the nonlinear  
51 static approach are presented and discussed.

52  
53 **Keywords:** *masonry structures; modeling strategies; equivalent frame approach; continuum*  
54 *approach; discrete approach; wall-to-wall connection; diaphragms modeling; nonlinear analyses*  
55

## 56 **1 Introduction**

57 At present, numerical models represent an essential tool that practitioners and researchers can  
58 utilize when analysing and interpreting the results of the simulation of the structural response of  
59 buildings. The ability of numerical models to correctly reproduce the actual seismic behavior of  
60 buildings is fundamental for the effective assessment of seismic risk analyses and the supporting  
61 of mitigation policies. Such a requisite is essential, in general, for existing buildings and,  
62 especially, for the unreinforced masonry (URM) ones.

63 Masonry buildings are distinctly multifaceted in terms of:

- 64 - Architectural configurations. A large variety in the layouts is observed not only in ordinary  
65 buildings, but also in monumental ones (like palaces, churches, fortresses, towers, see  
66 Lagomarsino et al. 2011 for a classification in seismic areas), as well as in aggregate  
67 masonry structures, which are quite ubiquitous in historical city centers.
- 68 - Masonry typologies. A rough classification distinguishes between “regular” and  
69 “irregular” masonry types, but it is well-known that there are many other factors, such as  
70 mortar quality, blocks type and shape, bond pattern, and transversal connections in multi-  
71 leaf walls (just to mention a few) that affect the overall seismic response of a building (as  
72 discussed, for example, in Borri et al. 2015, Cardani and Binda 2015, Krzan et al. 2015).
- 73 - Structural solutions for carrying gravity loads, such as the presence of different lintel  
74 typologies above the openings, segmental arches, timber/steel or reinforced concrete  
75 beams, etc.
- 76 - Structural solutions that influence the global behavior. For instance, the presence (or lack)  
77 of tensile resistant elements at floor level, steel tie rods or r.c. beams, diaphragms (*e.g.*,  
78 vaults, timber floors, r.c. slabs, iron beams and hollow bricks capped by r.c. slabs) and their  
79 connection to the walls, as well as the wall-to-wall interconnections.

80 From the previous considerations, it emerges how any numerical model, to be used in engineering  
81 practice, should be versatile enough to effectively describe a wide variety of masonry structures,

82 maintaining the ability to satisfactorily predict their actual behavior, and still being relatively  
83 simple to use.

84 Many surveys after past earthquakes (see for example D’Ayala and Paganoni 2011, Penna et al.  
85 2014a, Sorrentino et al. 2019) have shown how all the above-mentioned factors play a decisive  
86 role in defining the actual seismic response of URM buildings. Such factors are usually associated  
87 to two main categories of response: the in-plane one and the occurrence of local mechanisms  
88 (mainly associated to out-of-plane failures of single parts). More specifically, a large amount of  
89 data, made available by the Italian Department of Civil Protection through the Da.D.O platform  
90 (Dolce et al. 2019) on the observed seismic damage, has been recently used in statistical studies  
91 aimed at deriving empirical fragility curves (*e.g.*, Rosti et al. 2021, Del Gaudio et al. 2019,  
92 Lagomarsino et al. 2021). Such data gave the possibility of quantitatively addressing the influence  
93 that combined factors, like masonry quality (regular or irregular), type of diaphragms (flexible or  
94 rigid), and the systematic presence (or lack thereof) of connecting devices (*e.g.*, tie rods/tie r.c.  
95 beams) can have on the seismic response of masonry buildings. In fact, the results in terms of  
96 fragility curves have highlighted that masonry quality has a significant role on the seismic  
97 response, that there is a general tendency for vulnerability to decrease with increasing diaphragm  
98 stiffness (provided the quality of masonry is above a minimum), and, moreover, that there is a  
99 positive role of the presence of connecting devices. In addition, as discussed in De Felice (2011),  
100 it is known how the role of masonry quality is crucial in determining the actual morphology of  
101 damage associated to the out-of-plane response. In case of poor mortar quality, characterized by a  
102 low connection between leaves, it is needed to verify if the out-of-plane response can be due to  
103 the rigid-block idealization or to a masonry crumbling.

104 It is clear that in order to interpret such complex phenomena, first of all the analysts should have  
105 a sound knowledge of the recurring failure modes that may occur in order to properly identify the  
106 key features of the structure examined and, consequently, to address the modeling choices.  
107 Moreover, even if this paper is essentially numerical oriented, it cannot be disregarded that an  
108 appropriate knowledge phase also constitutes the preliminary but crucial requisite to support the  
109 modeling choices, addressing at the same time the matter of uncertainties (as discussed for  
110 example in Cattari et al. 2015a). In most of the cases, such phase presupposes the integration of  
111 various tools, *e.g.* historical analysis, in-depth visual structural in-situ surveys and experimental  
112 tests. Recent emblematic examples are illustrated for example in Lorenzoni et al. (2020), Ponte et  
113 al. (2021) and Camara et al. (2021), where it is stressed how nowadays the issue has to be faced  
114 more and more from a multidisciplinary perspective.

115 Instead, concerning the models, they must be effectively versatile and accurate in their ability to  
116 describe all possible behaviors and the cause-effect relationship with the corresponding structural

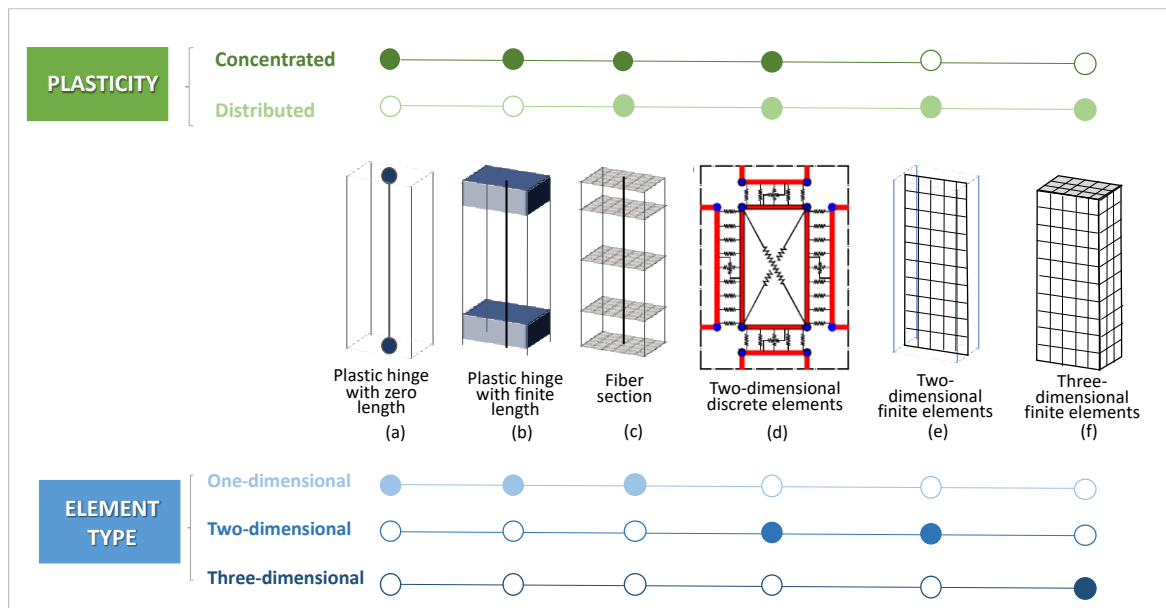
117 solutions and masonry typologies. Concerning the last aspect, there are many possible strategies  
118 that can be adopted. In the literature, studies devoted to the classification of the most widely  
119 adopted modeling strategies already exist (see, for example, Lourenço 2002, Roca et al. 2010,  
120 D’Altri et al. 2020a). In some of them, such a classification is provided as a function of  
121 architectural type (*e.g.*, in Lagomarsino and Cattari 2015a, for monumental buildings), and going  
122 into the detail on the state of the art for specific approaches (*e.g.*, in Quagliarini et al. 2017 for the  
123 equivalent frame approaches). Taking advantage of such robust background, the main goal of this  
124 paper is instead to provide the reader with a wide perspective on how to model effectively URM  
125 structures from a seismic and engineering practice point of view. To this aim, the use of various  
126 modeling strategies is critically reviewed in order to investigate how different solutions can be  
127 applied to simulate the above-mentioned phenomena and also highlighting the potential of  
128 integrating their use. Several numerical examples are illustrated and discussed to better  
129 demonstrate how the treated issues can influence the outcome of seismic verification. Many of  
130 them have been selected from the results of a wide research program, synthetically named in the  
131 following as "*URM nonlinear modeling – Benchmark project*" (Cattari and Magenes 2021). The  
132 latter has been carried out, starting in 2014, by several research units coordinated by the Authors  
133 of this paper and involved in the ReLUIS project (*Rete dei Laboratori Universitari di Ingegneria*  
134 *Sismica* - Italian Network of University Seismic Laboratories). As a matter of fact, the project gave  
135 the whole research group the opportunity to gain insight into various critical issues in modeling  
136 strategies and in the interpretation of the seismic response of URM buildings. The selection  
137 proposed in the paper relates to the challenging issues of URM modeling derived from this  
138 experience and has been integrated with the expertise on the topic by the Authors.

139 More in detail, the paper examines models working at different scales, that can be preliminarily  
140 classified into the following two main categories:

- 141 - Equivalent Frame (EF) models, defined at structural element scale identifying one-  
142 dimensional macroscopic structural elements, namely piers and spandrels, where all  
143 nonlinearity is concentrated, and whose size (length) is in the order of magnitude of the  
144 interstory height or the size of the openings (doors, windows). These elements are one-  
145 dimensional in the sense that they are mainly formulated as beam-column elements. Piers  
146 are the elements with a vertical axis, designed primarily to resist gravity loads and, in case  
147 of an earthquake, to transfer significant horizontal actions from the foundation to the  
148 elevation. Spandrels are masonry beams characterized by a horizontal axis, connecting  
149 contiguous piers. The elements are mutually interconnected by rigid links representing  
150 crossing areas between piers and spandrels.

151 - Models with a more “refined” discretization with 2D or 3D elements, in which masonry  
152 may be described through different degrees of accuracy using for instance: the following  
153 approaches: (1) fictitious homogeneous materials (also known as macro-scale modeling)  
154 by adopting a continuous mesh of 2D or 3D finite elements (FE) with isotropic or  
155 anisotropic laws; (2) discrete elements strategies, based on 2D or 3D macro-elements; and  
156 meso-scale (or micro-mechanical) approaches, in which the single components (mortar  
157 joints and blocks) are modeled separately. In this paper, the adjective “refined” aims at  
158 synthetically grouping a class of models that, differently from EF models, do not strictly  
159 require any *a priori* identification of piers and spandrels and that can describe the structure  
160 at macro or micro scale. For the sake of brevity, these models are referred to in this paper  
161 as “refined”, in a relative comparison with equivalent frame approaches.

162 Regardless the adopted modeling approach, another essential key feature is its capability to  
163 accurately reproduce the nonlinear response typical of masonry structures. Actually, masonry is  
164 characterized by very low tensile strength, which causes the onset of cracking phenomena for low  
165 levels of stress and, therefore, a nonlinear behavior even at early stages of incremental analyses  
166 and for low levels of loading, especially in the presence of horizontal actions. That being said, this  
167 requisite is essential in the seismic engineering field when dealing with the performance-based  
168 assessment (PBA). Figure 1 illustrates a possible classification of the element types used in EF  
169 and refined models as a function of the different approaches adopted for describing the nonlinearity  
170 (*i.e.*, distinguishing the concentrated or distributed nonlinearity). The selection of models  
171 represented in Figure 1 refers to the options more commonly adopted also in the engineering  
172 practice and available in commercial software. Figure 1 is not intended to be exhaustive of all  
173 possible solutions implemented at research level, like for example the block-based approach or  
174 other ones discussed in sub-paragraphs of Section 2.4. Moreover, as discussed in more detail in  
175 Section 2.1, solutions a, b and c of Figure 1 are typical of EF models, while the others are more  
176 commonly adopted in refined models. Such classification can also adapt to the different  
177 components that constitute the buildings (*i.e.*, vertical –*walls*, horizontal- *diaphragms*, and  
178 connections).



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Figure 1 – Possible classification of models commonly adopted by professionals and researchers for the seismic analysis of URM structures

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Within this general context and to pursue the aforementioned goals, the paper is organized into two main parts. The first gives an overview of challenging issues in URM modeling, focusing on criticalities aspects related to:

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- the modeling of structural elements (piers and spandrels) in EF models (Section 2.1) and the main issue of the proper calibration of parameters for a consistent cross-comparison between EF and refined models (Section 2.2.);
- the rules adopted to *a priori* define piers and spandrels and the challenges still open in the case of irregular layout of openings (Section 2.3);
- the modeling of wall-to-wall connections (Section 2.4);
- the modeling of the out-of-plane response of masonry walls (Section 2.5);
- the modeling of the diaphragms (Section 2.6).

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All the topics mentioned above are relevant whatever the method of analysis adopted for seismic assessment (linear, nonlinear, static, or dynamic). Conversely, the second part investigates some concerns related to the use of URM models within the specific field of seismic verification carried out through nonlinear static analysis (NLSA). Among the possible alternatives, the paper focuses only on the static method since it currently represents the most widespread approach adopted not only by professionals but also by the scientific community. In particular, three topics related to the NLSA are discussed: issues related to convergence and algorithmic aspects (Section 3.1), the application of horizontal loads (Section 3.1) and the definition of displacement thresholds on the pushover curve (Section 3.2). Since the focus in the paper is given only on the static approach, issues related to the description of the nonlinear behaviour are essentially limited to the monotonic field.

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## 206 **2 Overview on challenging issues of URM modeling in engineering practice**

### 207 **2.1 Critical issues for pier and spandrel modeling**

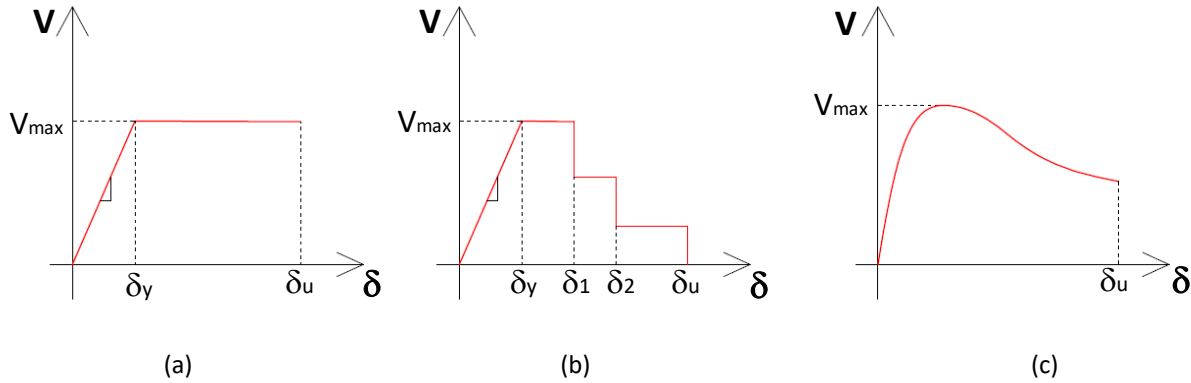
208 The rigorous identification of pier and spandrel elements is strictly necessary in the modeling  
209 phase only when dealing with EF models (as introduced in Section 1); however, it also turns out  
210 to be essential for refined approaches when the final aim is the seismic verification performed  
211 according to the procedures defined by the codes. In fact, seismic codes (*e.g.* NTC18 (2018), EC8-  
212 3 CEN (2005), ASCE 41-17 (2017), NZSEE (2017)) provide limit conditions that, for both  
213 strength and displacement capacity, commonly refer to quantities and parameters associated with  
214 the scale of masonry panels/walls rather than with the scale of the material, *i.e.*: generalized forces  
215 ( $V$ -shear,  $M$ -bending moment and  $N$ -axial load) for strength, and drift ( $\delta$ ) values for deformation  
216 and displacement capacity. Thus, if the analysis is carried out by means of refined models, the  
217 panel drift demand should be compared with the limits defined by the technical codes. In addition,  
218 the use of refined models will require the calculation of the generalized forces through the  
219 integration of the nodal forces and the selection of the position where the displacements are  
220 monitored. On the other hand, it is evident that the definition of what “piers and spandrels” are in  
221 a wall is purely conventional and in principle not required in refined models. The observation of  
222 the damage pattern predicted by the refined model may be useful in guiding the *a posteriori*  
223 identification of the structural elements, similar to the basis of rules usually adopted in EF models,  
224 as discussed in more detail in Section 2.3.

225 First of all, regardless of the approach adopted, the correct modeling and interpretation of the piers  
226 and spandrels seismic behavior requires proper knowledge of the possible failure modes which  
227 can occur under seismic actions, namely those associated with diagonal shear cracking, bed joint  
228 sliding, or flexural response (*i.e.*, rocking or crushing). As well known, failure modes are usually  
229 interpreted according to simplified analytical strength criteria, as specified in detailed reviews like  
230 those presented in (Magenes and Calvi 1997, Calderini et al. 2009), for piers, and in (Beyer and  
231 Mangalathu 2013), for spandrels. The width-to-height slenderness ratio, the applied axial force,  
232 the boundary conditions and the strength properties play fundamental roles in defining the  
233 structural capacity of these elements. Different combinations of such factors can lead to different  
234 failure modes. Moreover, in the case of spandrels, the lintel typology and the presence of other  
235 tensile resistant elements at floor level have to be added to the aforementioned list of factors. A  
236 differentiation between piers and spandrels, although conventional, is convenient, given the  
237 different role that they play in the “load path” under the application of both vertical and horizontal  
238 loads. Piers are subjected to significant compressive axial forces due to gravity, whereas for



239 spandrels, the axial forces are often negligible, except when other tensile resistant elements or very  
240 stiff diaphragms are present.

241 Figure 2 illustrates the different possible idealizations made for the shear force – displacement  
242 behavior of a panel through the adoption of the various modeling strategies synoptically shown in  
243 Figure 1.



246 Figure 2 – Possible idealization of the shear behavior of URM panels obtained through: (a/b) nonlinear  
247 beams with lumped plasticity; (c) fiber model or refined 2D- or 3D- models. Where:  $V$  shear force,  $\delta$   
248 displacement or drift,  $V_{max}$  maximum shear force of the panel,  $\delta_u$  ultimate displacement/drift capacity,  $\delta_y$   
249 displacement/drift capacity at yielding.

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251 The simplest idealization refers to the nonlinear beam scheme with lumped plasticity. It represents  
252 the most common solution in EF models implemented in commercial software packages. If zero-  
253 length hinges are used, three different hinges are usually placed along the vertical beam element  
254 (which represents the pier): two flexural hinges at the edges of the beam and, assuming that the  
255 shear force is constant along the pier, the third one (shear hinge) can be in any position along the  
256 beam (usually in the middle). The force (moment or shear) - displacement (chord rotation or drift)  
257 relationship of the element generally results in an elastic-plastic behavior (similarly to what  
258 depicted in Figure 2a), with or without a softening branch that may be characterized by a stepped  
259 drop behavior in strength (similarly to that depicted in Figure 2b, as explicitly mentioned for  
260 example in CNR DT212 2014). This model is usually based on the following assumptions: the  
261 interaction between the axial force and the bending moment is accounted for through simplified  
262 approaches, or sometimes neglected, and no accurate shear-flexure interaction failure mechanisms  
263 are accounted for.

264 As far as the initial stiffness is concerned, usually conventional secant values are assumed. An  
265 alternative for the flexural behavior idealized through a hinge is the finite length approach, which,  
266 in the majority of cases, is based on the so-called fiber approach. In this case, the progressive  
267 degradation in the first branch of the response (before the attainment of the maximum shear  
268 strength,  $V_{max}$ ), associated with a progressive flexural cracking of the sections, can be explicitly  
269 modeled (Figure 2c).

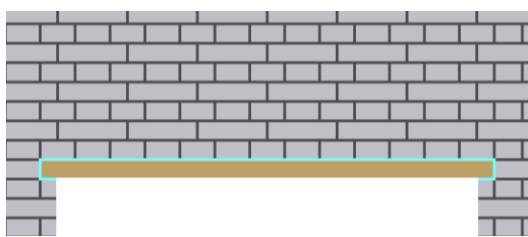
270 According to this general framework and focusing the attention on EF models, various proposals  
271 have been presented in the literature: the nonlinear beam model with lumped (*e.g.*, Magenes and  
272 Della Fontana 1998, Lagomarsino et al. 2013, Cattari and Lagomarsino 2013, Vanin et al. 2020a)  
273 or spread plasticity (*e.g.*, Belmouden and Lestuzzi 2007); the use of nonlinear springs in series or  
274 in parallel (*e.g.*, Chen et al. 2008); the fiber approach (*e.g.*, Raka et al. 2015); or other hybrid  
275 macro-elements that combine the use also of a mechanical approach (*e.g.*, Penna et al. 2014b,  
276 Bracchi et al. 2021, Bracchi and Penna 2021). Some of the aforementioned models include also  
277 specific formulations to simulate the influence of strengthening, *e.g.* with FRP strips (Grande et  
278 al. 2011). Although an exhaustive review on this topic is out of the scope of the paper, it is possible  
279 to highlight some distinctive features of the available literature: i) the adoption of a mechanical or  
280 phenomenological approach to describe the nonlinear response; ii) a less or more accurate  
281 kinematic relationship among variables (*e.g.* able to take into account the coupling between the  
282 axial displacement and the rotation due to flexural cracking that is responsible of the uplift  
283 phenomenon in piers dominated by a flexural failure mode); iii) the formulation of an appropriate  
284 hysteretic response, essential for performing nonlinear cyclic pushover and dynamic analyses.  
285 Conversely, refined models can usually describe more accurately the progressive degradation of  
286 both stiffness and strength (Figure 2c). The counterpart is the increase of the computational burden  
287 needed and a larger number of parameters to set in the constitutive laws characterizing the material  
288 behavior, as for example the need to introduce a fracture energy parameter, in FEM based  
289 approaches, for governing the softening branch. Section 2.2 addresses this issue in-depth for the  
290 purpose of performing a calibration of parameters able to guarantee, as much as possible, cross-  
291 consistency in the use of EF and refined models in practical applications.

292 The modeling of spandrels deserves some additional attention. In fact, although such elements can  
293 be considered as secondary (their failure does not usually imply the attainment of an ultimate limit  
294 state of the structure), they significantly affect the static scheme of piers, as a function of their  
295 strength and stiffness properties, with possible situations ranging from the weak-spandrel-strong-  
296 piers (WSSP) behavior to the strong-spandrel-weak-piers (SSWP) behavior. In the case of weak  
297 spandrels (low stiffness or low resistance – leading to WSSP), piers tend to behave as cantilever-  
298 like elements, mainly dominated by the flexural failure mode. On the contrary, as the stiffness and  
299 strength of the spandrels increase (until the idealized case of SSWP), a frame-like behavior of the  
300 wall arises, piers are more affected by shear failure modes. These issues, together with the role of  
301 horizontal coupling elements in determining the overall response of URM walls and buildings, has  
302 been already preliminary investigated in the earliest works of '90 (Magenes and Della Fontana  
303 1998, Magenes et al. 2000). In such works, based on nonlinear EF models, it has been shown that,  
304 as expected, the effects of the variation in some basic properties and modeling hypotheses of URM

305 spandrels and coupling beams would significantly change the response of multistorey masonry  
306 structures in terms of base shear capacity and overall failure mechanism, calling for further  
307 research, in particular experimental (at that time almost completely absent).  
308 Most of the design/assessment codes explicitly recommend including spandrels in the modeling  
309 phase only if an effective lintel (*i.e.*, properly anchored at the end sections and able to support the  
310 weights above it) is present. As testified by the experimental campaigns available in the literature  
311 (*e.g.*, Beyer and Dazio 2012, Graziotti et al. 2012), spandrel behavior is significantly affected by  
312 the interaction with the lintel. Moreover, the post-peak softening phase strongly depends on the  
313 lintel type and presents a steep degradation in the case of segmental arch systems and a more  
314 gradual softening in the presence of timber/steel lintels (as also discussed in Beyer 2012). The  
315 spandrel-lintel modeling interaction phenomena is treated in a very different way passing from EF  
316 to refined models. In fact, in the first case (EF models), according to a phenomenological approach,  
317 it is usually accounted for in an indirect way by properly calibrating the properties of plastic hinges  
318 (*e.g.*, introducing in the nonlinear behaviour of the beam a residual strength after the peak).  
319 Conversely, in the second case (refined models), both elements must be meshed separately,  
320 introducing some complexities in the definition of the nonlinear model but avoiding the a priori  
321 distinction between piers and spandrels. In refined models lintels should be able to slip, in order  
322 to avoid the transfer of fictitious stresses to the surrounding masonry. This behavior can be  
323 achieved by inserting an interface or a weak layer between masonry and lintel, as shown in Figure  
324 3a. Thus, Figure 3b shows that, in this way, the damage behavior is predicted correctly; clearly,  
325 this behavior cannot be captured if no slip is possible between lintel and masonry.

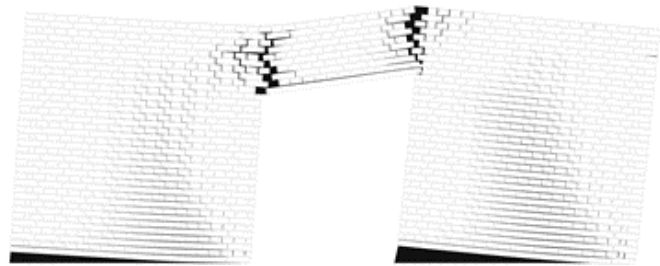
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b)

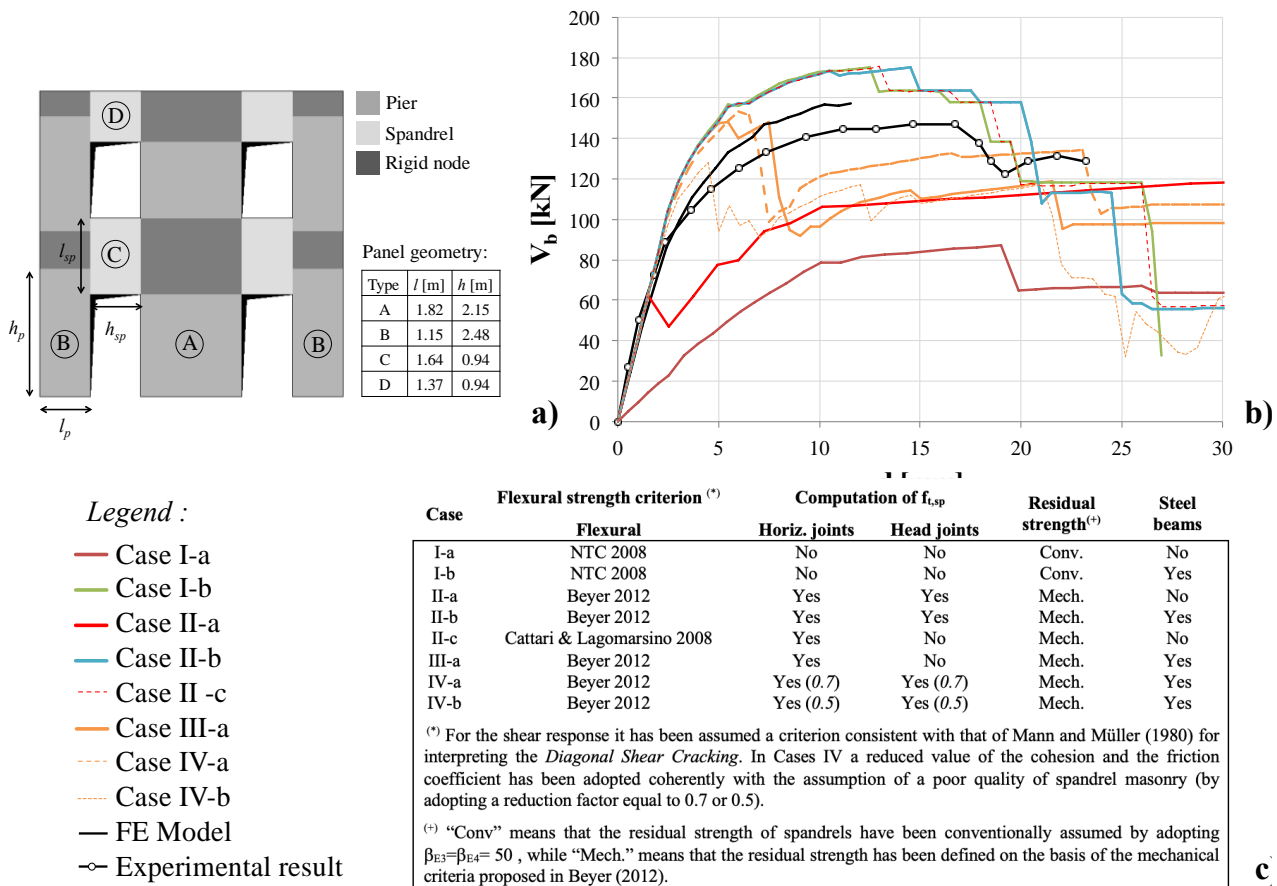


329 Figure 3 –Example of lintel-spandrel interaction simulated by a micro-mechanical approach: a) interface  
330 between masonry spandrel and lintel (in cyan); b) simulated response

331

332 Another relevant issue is the capability of the model to reproduce the actual failure mechanisms  
333 that may occur in the spandrel, also as a function of its interaction with other tensile resistant  
334 elements, possibly coupled (*i.e.* tie rods or r.c. ring beams). This is particularly relevant for a  
335 reliable simulation of the flexural behavior in EF models. In fact, in the past, the common approach  
336 was to adopt the same analytical criteria adopted for piers, leading to an unrealistic (if compared

337 with the observed seismic damage) predominance of the flexural failure mode in weak URM  
 338 spandrels (*i.e.*, not coupled to other tensile resistant elements). Conversely, recent codes (*e.g.*,  
 339 NZSEE 2017, MIT 2019), based on the latest experimental evidence, propose to consider the  
 340 contribution of an equivalent horizontal tensile strength ( $f_{t,sp}$ ) of the spandrel that can be produced,  
 341 even in absence of other coupled tensile resistant elements, by virtue of the interlocking  
 342 phenomena generated at the end sections of spandrel with the adjacent masonry portions (*e.g.*,  
 343 Cattari and Lagomarsino 2008a, Beyer 2012, Krzan et al. 2015). Such a contribution tends to  
 344 become more relevant in the presence of a regular bond masonry pattern, good quality mortar  
 345 joints and adjacent masonry portions subjected to high vertical stresses (*i.e.*, the positive effect is  
 346 much more evident for lower levels than for upper floors). The inclusion of the  $f_{t,sp}$  contribution in  
 347 the spandrel modeling may significantly alter the response of URM walls as depicted for  
 348 illustrative purposes, in Figure 4, where such parameter has been computed according to various  
 349 formulations available in the literature (*e.g.*, Cattari and Lagomarsino 2008a, Beyer 2012). The  
 350 figure also shows the effect produced by the insertion of a steel tie rod coupled to the spandrel;  
 351 more details can be found in Cattari and Beyer (2015).



352  
 353 Figure 4 – Effect of accounting for the equivalent tensile strength of the spandrel ( $f_{t,sp}$ ) and the presence of  
 354 a coupled tensile resistant element in the seismic response of a two-story URM wall (adapted from Cattari  
 355 and Beyer 2015)  
 356

357 When a tensile resistant element is coupled to the spandrel, all modeling strategies are unanimous  
358 in predicting a predominance of the shear failure modes. In EF models, such an outcome is a  
359 consequence of the analytical strength criteria adopted, that usually are based on the development  
360 of a diagonal strut in the spandrel. In refined models, the appearance of the strut depends on the  
361 explicit interaction with the steel tie-rod or the r.c. beam that also significantly alters the axial  
362 force acting along the spandrel.

363 Figure 5 shows the variation of the behavior of a multi-story URM wall passing from the case of  
364 weak URM spandrels (Figure 5 a and d) to a model that considers the addition of elastic floor  
365 beams at each level (except the roof level) (Figure 5 b and e) and, finally, to a model that considers  
366 nonlinear concrete floor beams and elastic lintel beams (Figure 5 c and f). In the case of very weak  
367 masonry, the achievement of the tensile strength on the spandrel can lead to a full separation of  
368 the wall in different portions, since no constraint is a priori assumed. A very consistent response  
369 is simulated by a discrete macro-element approach (DMEM, Figure 5a/b/c) and a micro-  
370 mechanical approach, respectively (Figure 5d/e/f). For further details, interested readers may refer  
371 to Occhipinti et al. (2021). Such behavior, which can also be exhibited by continuous FE and  
372 mesoscale approaches, is also influenced by the actual position of the r.c. or floor beams modeled.  
373 More specifically, in the DMEM method, spandrels are modeled with a single or a mesh of macro-  
374 elements depending on the presence of lintels or floor beams. Each macro-element can simulate  
375 the shear-diagonal and/or the flexural response of the spandrels, including the tensile capacity of  
376 masonry in the horizontal direction, when appropriate. The presence of floor beams or lintels is  
377 considered in their correct position by introducing an inelastic frame element interacting with the  
378 macro-elements (Occhipinti et al. 2021).

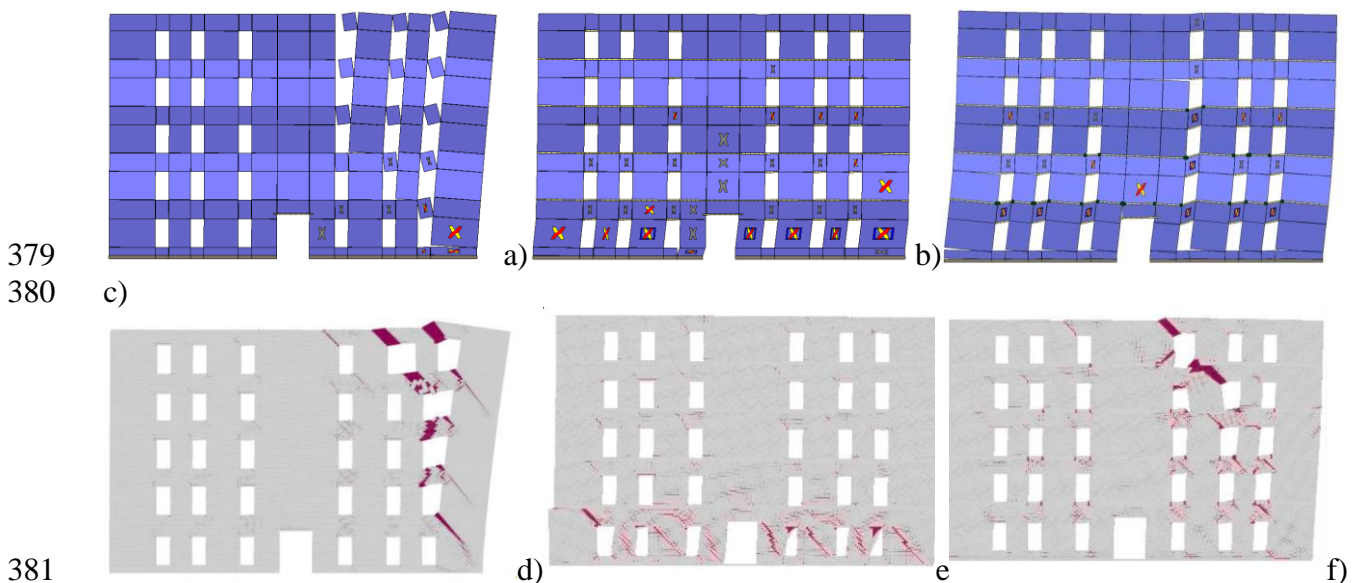


Figure 5 – Response of a multi-story URM wall varying some modeling options for spandrels: a), d) weak spandrels; b), e) elastic floor beams at each level except the roof level; c), f) nonlinear concrete floor beams and elastic lintel beams coupled to spandrels (adapted from Occhipinti et al. 2021)

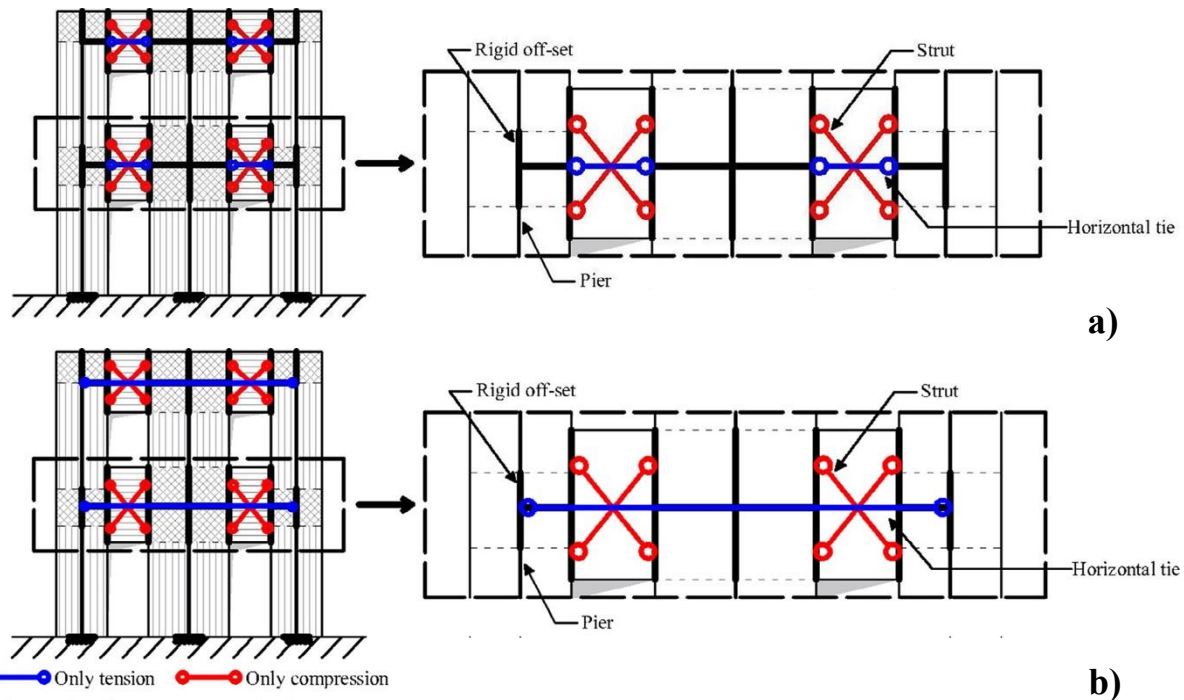
386 Another problematic point is the correct evaluation of the axial load acting in the spandrel. This  
387 aspect is usually roughly approximated in EF models (due to oversimplifications usually made in  
388 the modeling of the floors), while a more reliable prediction is provided by refined models (when  
389 the interaction with all the other structural elements is properly simulated).

390 Finally, other modeling solutions aimed at more accurately predicting the spandrel response are  
391 under development by researchers, based on experimental and numerical tests (Calderoni et al.  
392 2019a). In the framework of the EF approach, in order to better simulate the spandrel behavior  
393 within the masonry wall without losing the easiness of the frame model, an alternative EF hybrid  
394 scheme has recently been proposed (Sandoli et al. 2020a). In this scheme, the piers are modeled  
395 with one-dimensional vertical frame elements, while the spandrels are modeled with strut and tie  
396 truss elements (acting only in compression or in tension, respectively, and exhibiting nonlinear  
397 behavior) instead of equivalent nonlinear beams (Figure 6). This EF hybrid scheme (Sandoli et al.  
398 2020b) gives the following advantages with respect to a “classic” EF scheme (especially when  
399 effective tensile-resistant elements are present in the spandrels):

- 400 - the axial force in the diagonal strut (which is related to the structural engagement of the  
401 spandrel) is expressly evaluated up to the diagonal strut failure, giving the control of the  
402 internal stress state of the spandrel;
- 403 - the partialization of the end cross-sections due to material cracking is considered through  
404 the dimensions of the equivalent diagonal strut (*i.e.*, effective compressed area at the panel  
405 edges) and its position inside the panel, defined according to a specific theoretical  
406 formulation (Calderoni et al. 2011);
- 407 - the tensile axial force in the ties is directly given by the analysis, and different positions of  
408 ties, r.c. ring beams, and different typologies of ties (bonded continuously to the masonry  
409 or unbonded, *i.e.*, connected to the wall at the edges only) can be easily accounted for;
- 410 - it allows for the identification of the component (*i.e.*, compressed masonry and/or tensile-  
411 resistant element) triggering the spandrel failure, and consequently, the overall failure  
412 mechanisms of the entire equivalent frame can be better identified, also in terms of  
413 displacements.

414 However, this type of hybrid modeling is, at present, not readily available in commercial EF  
415 software for the analysis of masonry buildings, but needs to be specifically implemented by the  
416 user through available general purpose software using nonlinear frame and truss elements.

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Figure 6 – Hybrid EF model in the case of: a) bonded ties; b) un-bonded ties (adapted from Sandoli et al. 2020a)

## 2.2 Parameters calibration for a consistent cross-comparison between refined and equivalent frame models

This section discusses some calibration and modeling issues in the nonlinear analysis of URM structures. A consistent cross-comparison between different numerical strategies with reference to specific modeling aspects is reported in the following sub-sections.

### 2.2.1 Basics and target for the calibration procedure

In this sub-section, some issues related to the calibration of mechanical parameters are addressed to guarantee, as much as possible, cross-consistency between refined and EF models. The topic is particularly useful for practitioners since, in most cases, the adopted reference mechanical properties of masonry types directly refer to the scale of masonry panels and are ready-to-use in the simplified analytical strength criteria mentioned in Section 2.1. Typical reference values may be found in Codes (*e.g.*, MIT 2019) or in the literature (*e.g.*, Augenti et al. 2012, Krzan et al. 2015, Vanin et al. 2017, Morandi et al. 2018, Rezaie et al. 2020, Boschi et al. 2021). The same mechanical parameters are also those derivable from in situ experimental tests, after their proper interpretation and the careful treatment of data, which often constitutes another delicate point of the assessment (*e.g.*, as discussed in Brignola et al. 2009, for the diagonal compression test). As a consequence, in the case of an EF approach, these parameters can be directly adopted as input data without needing any specific calibration of the panel. On the contrary, for refined approaches, which usually refer to the material scale, the basic material parameters, expressed in terms of elastic modula, tensile and compressive strength and fracture energies, have to be considered.

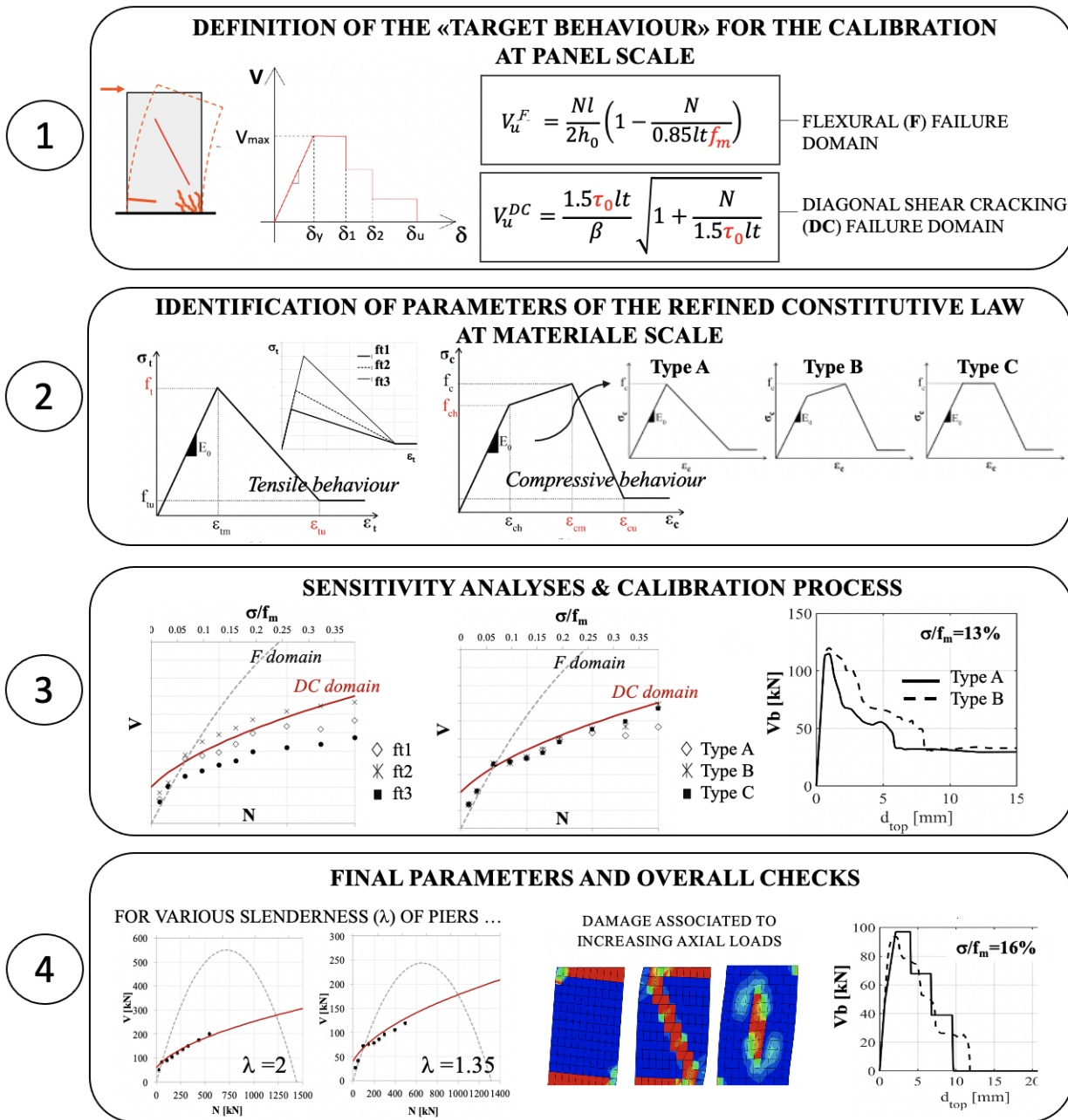
442 The main steps expected in this calibration phase are summarized in Figure 7 and consist of the  
443 following points:

- 444 1. Definition of the “target behaviour” for the calibration. Keeping in mind that both EF  
445 models and refined models aim to reproduce the same masonry, consistent with code-  
446 defined parameters, first a choice is made of the most relevant simplified code-based  
447 strength criteria to be adopted to describe the main failure modes expected in a pier for the  
448 masonry typology of interest. Then the target behavior is set, as a consequence of such  
449 definition of the mechanical parameters, *i.e.*, strength parameters (like those marked in red  
450 in Figure 7-step 1) and stiffness properties as well as those necessary for the description of  
451 the post-peak behavior of panels (like those of the drift thresholds and corresponding  
452 strength drops associated to different damage conditions). In Figure 7, among the possible  
453 behaviours, the flexural response ( $V_u^F$ ) and the diagonal shear cracking ( $V_u^{DC}$ ) are shown  
454 together with some common strength criteria adopted in codes, based on the compressive  
455 strength ( $f_m$ ) and the shear strength ( $\tau_0$ ), respectively, as mechanical parameters. Of course,  
456 the post-peak behavior can be more or less complex, as depicted in Figure 2, as a function  
457 of the “target behavior” assumed.
- 458 2. Identification of the main parameters on which the constitutive laws of the refined model  
459 employed are based. As an example, Figure 7-step 2 depicts an isotropic plastic-damaging  
460 3D continuum model based on the proposal by Lee and Fenves (1998) and implemented  
461 in the Finite Element FE commercial software ABAQUS. The parameters investigated in  
462 the following step 3 are highlighted in red. In particular, they consist of: the tensile strength  
463 of the material ( $f_t$ ) and the uniaxial compressive stress corresponding to the point of initial  
464 yield ( $f_{ch}$ ); the values of uniaxial compressive strain corresponding to the reaching of the  
465 maximum strength ( $\epsilon_{cm}$ ) and to the end of the softening branch ( $\epsilon_{cu}$ ). Different choices on  
466 such parameters may lead to various alternative options of the behaviour (e.g. Type A, B  
467 and C indicated in Figure7-step 2), whose repercussions may be investigated through  
468 targeted parametric analyses. Obviously, such parameters have to be particularized to the  
469 constitutive law adopted in the different cases.
- 470 3. Execution of sensitivity and parametric analyses aimed at least to evaluate *i*) the maximum  
471 lateral strength of the panel ( $V_{max}$ ) and *ii*) the associated base shear – top displacement ( $V_b-$   
472  $d_{top}$ ) curves (Figure 7-step 3). The analysis has to involve a set of piers characterized by  
473 various in-plane aspect ratios ( $\lambda$ ) and various boundary conditions subjected to various  
474 different values of the applied axial load (N or  $\sigma/f_m$  ratio). Such a step aims to optimize the  
475 calibration of the parameters to reasonably match the “target behaviour” set in step 1.



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4. Overall verification of the effectiveness of the set of parameters chosen at the end of step 3.



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Figure 7 – Schematic flowchart of the calibration process for guaranteeing the cross consistency of parameters used at material and panel scales

482 A detailed exemplification of the aforementioned calibration procedure is described in D’Altri et  
483 al. (2021), where the differences resulting in the use of five constitutive laws of refined models  
484 are investigated, and in Cattari et al. (2021a), where specific reference to the aforementioned  
485 constitutive law and implemented in ABAQUS is made.

486 It is worth underlining that a perfect calibration between the simplified and the refined models in  
487 all the regions of the failure domain of panels is in general not possible. In fact, while in the EF  
488 approach, the flexural and shear behavior are completely decoupled, being associated with failure

489 criteria based on independent parameters, this does not happen in the refined FE approaches based  
490 on continuum mechanics, where the mechanical parameters ruling the flexural response can also  
491 affect the shear behavior (for example the tensile strength, see also Section 2.2.2). However, a  
492 rather good correspondence can be obtained if the calibration is performed by considering a limited  
493 portion of the strength domain of the piers within the range of variation of the normal stress for  
494 practical application. The latter can be estimated as the expected range of variation occurring in  
495 the panels of the structure under examination after the execution of some preliminary analyses.  
496 Step 3 is carried out by varying specific parameters of the constitutive law characterizing the  
497 continuum material in order to find the best solution for the calibration in terms of:

- 498 - *elastic stiffness*, to reproduce the initial elastic behavior of masonry piers;
- 499 - *strength*, to reproduce the failure domain of the piers considered, defined according to  
500 the analytical strength criteria adopted in the EF approach;
- 501 - *displacement capacity*, to simulate, for different values of the applied axial load, the  
502 post-peak behavior of the piers described through the assigned drift thresholds and  
503 corresponding strength drops;
- 504 - *post-peak behavior*, including damage and degradation.

505 A reliable description of the post-peak response of masonry piers subjected to shear is a  
506 challenging task since it is extremely variable and it depends on many factors (*e.g.*, masonry type,  
507 masonry bond/texture, axial load ratio, failure mode, etc.).

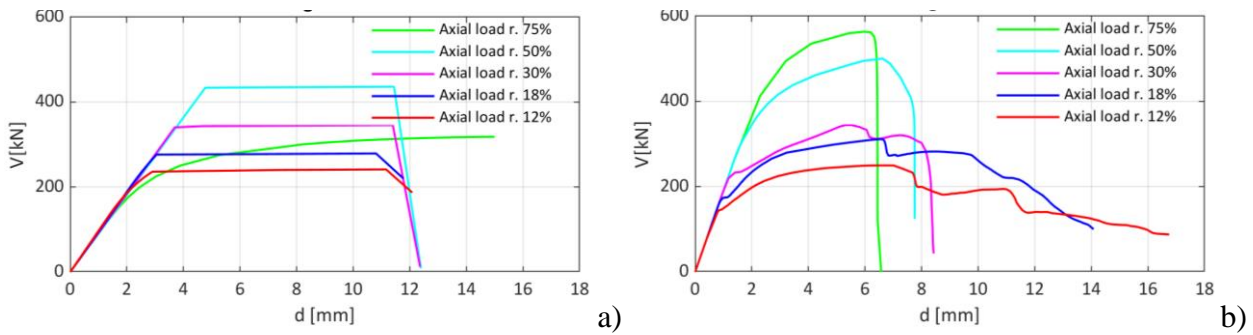
508 The drift limits and strength degradation parameters assigned as reference for the selected masonry  
509 type can be directly assumed as input data in the EF models. Conversely, when the masonry panel  
510 is modeled at the material scale, numerical simulations may produce a softening behaviour  
511 characterized by a more or less gradual strength and stiffness degradation, due to the progressively  
512 occurred damage and the specificity of the adopted parameters. With the final goal of guaranteeing  
513 a cross-consistency between simplified and refined models, the aim of the calibration is to  
514 approach the envelope represented by the simplified code-prescribed piecewise-linear behavior  
515 assumed at the panel scale.

516 In the EF approach, with panels described by nonlinear beams based on a phenomenological  
517 approach, the calibration process is straightforward, being univocally determined as the failure  
518 mode associated with the minimum between the flexural and shear strength for the current value  
519 of the axial load. As mentioned in Section 2.1, in the EF models, the ultimate drift (or those  
520 associated with the progressing strength decay in the case of multi-linear laws) is conventionally  
521 defined according to Codes (*e.g.*, EC8-3 CEN (2005), MIT 2019, ASCE 41-17 2017),  
522 recommendations (CNR DT 212/2013 (2014)) or literature proposals based on a large dataset of  
523 experimental campaigns (Vanin et al. 2017, Morandi et al. 2018). Moreover, in general, in EF

524 models, a sudden transition between prevailing flexural and prevailing shear response occurs, thus  
525 also implying a sudden transition in the displacement capacity associated with the considered  
526 element. As a rule, the utilization of hybrid modes is not managed by such an approach, apart from  
527 very few cases like that in (Cattari and Lagomarsino 2013) and exemplified in (Cattari et al. 2018,  
528 Brunelli et al. 2021), where, for hybrid modes average values of the drift thresholds are proposed  
529 on the basis of a heuristic criterion. In this proposal, the hybrid mode is detected as a function of  
530 the axial load acting on the panel, when it is in the transition region of the domain in which the  
531 strengths predicted by the flexural and shear analytical criteria are very close.

532 On the other hand, in the case of the refined approach, the assignment of a precise failure mode to  
533 a given panel is not needed since the collapse mechanisms are related to the adopted constitutive  
534 law for the continuum and not a priori identified according to the panel geometry and masonry  
535 material properties. The identification of the failure type can be considered as unequivocal only in  
536 the presence of pure flexural response (with the sole flexural cracking of the end sections). In all  
537 other cases, the formation of shear diagonal cracks is almost always associated with a more or less  
538 pronounced flexural cracking of the end sections, thus indicating failure modes that appear to be  
539 closer to hybrid situations rather than pure shear failures. Therefore, when using a refined  
540 approach, the obtained damage pattern, which is usually more complex and closer to reality, has  
541 to be properly interpreted case by case. This can be done, in general, only on the basis of qualitative  
542 analyses or conventional criteria (as discussed in Castellazzi et al. 2021).

543 Moreover, the post-peak response in material-scale models (*e.g.*, continuum models) is ruled by  
544 the fracture energy and has, in general, a less regular behavior than the simplified EF models, and  
545 more similar to the experimental one. The issue is to some extent controversial since, on the one  
546 hand, the ultimate drift is a panel-scale property but, on the other hand, the fracture energy is a  
547 material-scale property. Accordingly, it appears very complex (or nearly impossible) to calibrate  
548 fracture energies in order to have a constant target ultimate drift and vice versa. Indeed, while most  
549 codes (*e.g.*, Eurocode 8-2 CEN (2005), NTC18 (2018), ASCE 41-13 2017) conventionally assume  
550 ultimate drifts *a priori* defined and invariant with the acting axial load (except for the SIA 266  
551 (2015)), the resulting ultimate drift obtained in continuum models, and so ruled by the fracture  
552 energy, typically depends on the axial load ratio (Figure 8–*b*). Indeed, the influence of other factors  
553 on the drift capacity in addition to the axial load, like boundary conditions, slenderness, and  
554 masonry type, is still debated in the literature (see Petry and Beyer 2014, Wilding and Beyer 2018,  
555 Dolatshahi et al. 2018) and the fixed drift limits defined in design codes are only lower bounds of  
556 quite scattered experimental results.



557

558 Figure 8 – Masonry panel force-displacement response: (a) modeling strategy with a priori definition of  
 559 ultimate drift where the ultimate drift depends only on the failure mode, and (b) material-scale modeling  
 560 strategy where the resulting ultimate drift is influenced by the axial load ratio (adapted from D’Altri et al.  
 561 2021)  
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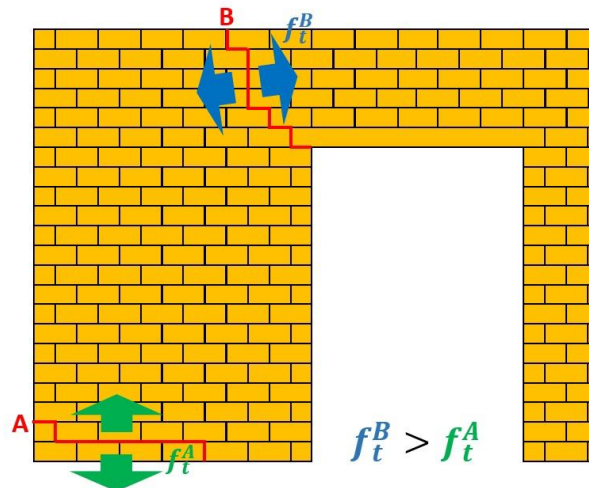
### 563 2.2.2 Additional issues on the use of isotropic and orthotropic material models

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565 When homogeneous isotropic material models are used to model masonry, a criticality arises in  
 566 stiffness calibration. In an isotropic linear elastic continuum, the following well-known  
 567 relationship between the three elastic constants  $E$  (Young’s modulus),  $G$  (shear modulus), and  $\nu$   
 568 (Poisson’s coefficient) holds:  $G = \frac{E}{2(1+\nu)}$ . However, given the anisotropic and heterogeneous  
 569 nature of masonry, the use in such a relationship of values of  $E$  and  $G$  deduced from experimental  
 570 tests or literature studies, would produce values of  $\nu$  that are clearly unrealistic if related to an  
 571 elastic homogenous isotropic material (*e.g.*, greater than 0.5 instead of typical values between 0.15  
 572 and 0.3). Therefore, it does not always appear feasible to directly use measured (or suggested from  
 573 literature)  $E$  and  $G$  values and derive a realistic value of  $\nu$  from the isotropic relationship.  
 574 Accordingly, a possible strategy could be to use a realistic value of  $\nu$  for masonry (*e.g.*, 0.2) and  
 575 to choose only one of the two measured (or suggested) moduli ( $E$  or  $G$ ) to be adopted in the  
 576 analysis, depending on the geometry, loading and boundary conditions of the structure, as  
 577 suggested in D’Altri et al. (2021).

578 Moreover, in isotropic material models, although strength could be distinguished between  
 579 compression and tension, only one value of uniaxial tensile strength and one value of uniaxial  
 580 compressive strength can be defined. Conversely, masonry generally shows anisotropic strength  
 581 (Page 1981). For example, the tensile strength perpendicular to bed joints could be significantly  
 582 lower than the tensile strength parallel to bed joints (see Figure 9), their ratio reaching in some  
 583 cases values near 5 (Lourenço 1997). Accordingly, isotropic material models cannot account for  
 584 these differences, and only one value of tensile strength has to be used. However, it appears  
 585 possible to calibrate the tensile strength of the material (*e.g.*, following the procedure suggested in  
 586 Section 2.2.1 and exemplified in D’Altri et al. 2021) in order to keep the level of approximation  
 587 included within engineering practice tolerance.

588 A further limitation on the use of isotropic models is related to the difficulties in choosing suitable  
589 mechanical parameters for modeling the typical failure mechanisms of masonry panels. The  
590 definition of tensile and compressive strength in continuous isotropic models controls the collapse  
591 response of a masonry panel subjected to vertical and horizontal loads: a correct calibration of the  
592 flexural collapse mechanism, according to experimental results, does not guarantee a  
593 corresponding calibration of the shear failure, which is generally related to different values of  
594 tensile strength. This aspect is deeply discussed in D’Altri et al. (2021) along with more detailed  
595 practical approaches to solve the issue.



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597 Figure 9 – Anisotropic nature of the masonry tensile strength.  
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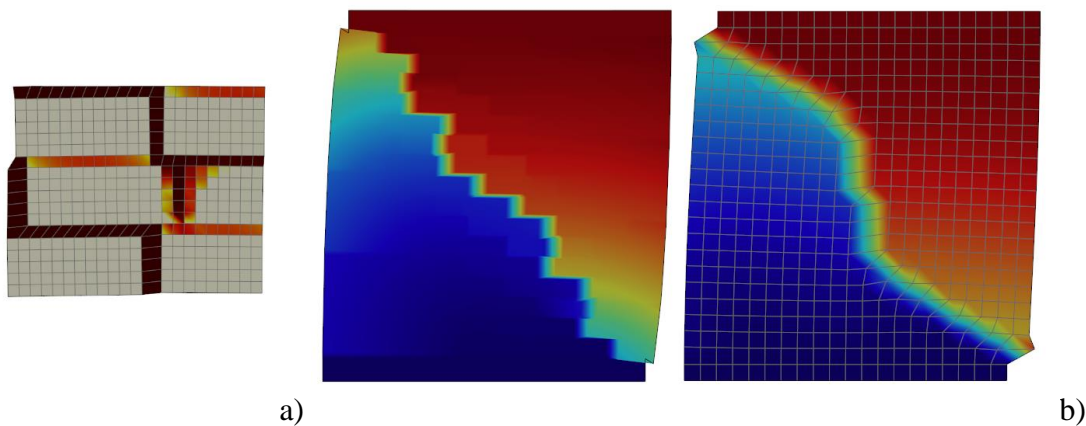
599 Concerning orthotropic continuum models, the orthotropy of masonry can be introduced either by  
600 using standard nonlinear isotropic models, but adopting a micro-modeling approach (also called  
601 textured continuum models, D’Altri et al. 2020b) or by using equivalent homogenous orthotropic  
602 nonlinear models.

603 The first approach directly leads to an orthotropic behavior without involving the use of orthotropic  
604 constitutive laws; however, it does not immediately provide the mechanical behavior of masonry  
605 at the macro scale. The orthotropy is naturally introduced by the explicit modeling of the texture  
606 (Berto et al. 2004, Petracca et al. 2016 and 2017a). This approach, however, leads to very  
607 demanding models, and consequently to high computational costs.

608 The second approach, instead, is much faster, but it requires the use of an orthotropic nonlinear  
609 constitutive model, that cannot be easily found in most finite element modeling codes.  
610 Additionally, it would require the introduction of further material parameters that are not always  
611 available to the analyst. An example is represented by the orthotropic damage model proposed by  
612 Berto et al. (2002) where the material parameters can be calibrated from a suitable set of  
613 experimental tests on full-scale masonry panels or from the mechanical behavior of the  
614 components (brick and mortar) through homogenization techniques or micro-modeling analysis.

615 A detailed state-of-art on the equivalent homogenization approach is out of the scope of the paper  
616 and the interested reader may refer to (Belytschko et al. 2008, Bosco et a. 2015, Cavalagli et al.  
617 2013, Kouznetsova et al.2004, De Bellis et al. 2011, Zucchini and Lourenco 2009, Berke et al.  
618 2014, Sacco 2009, Mercatoris and Massart 2011, Massart et al. 2007), to name a few.

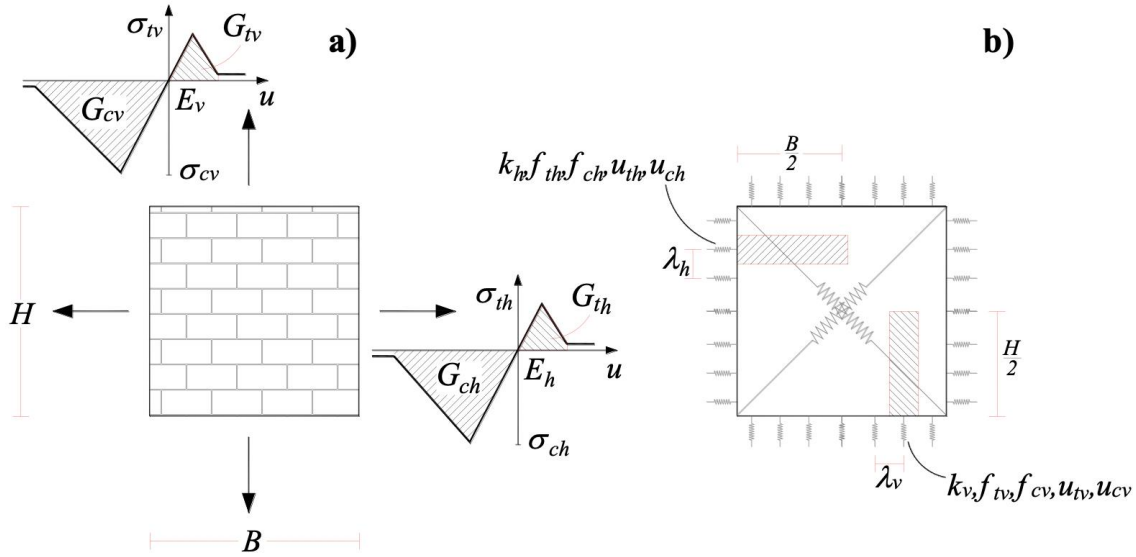
619 An alternative approach is to use an orthotropic mapper. It is a simple material wrapper that  
620 converts a fictitious isotropic material into an equivalent orthotropic one, based on the orthotropic  
621 elastic tensor and the ratios of the isotropic and orthotropic strengths (implemented in OpenSees,  
622 see for instance Pelà et al. 2013). Furthermore, the mapping parameters can be easily obtained by  
623 an initial analysis of an RVE (Representative Volume Element) of the masonry material (see  
624 Figure 10).



625 a) b)  
626 Figure 10 – (a) RVE response for calibration; (b) Reference micro-model; (c) Equivalent homogenous  
627 orthotropic macro-model.  
628

629 In the DMEM model, the mechanical calibration of the interface is based on a fiber discretization  
630 (Caliò and Pantò 2014) that allows for the definition of different mechanical properties in the  
631 vertical and horizontal directions of the panel, as summarized in Figure 11. As reported in the  
632 figure, for each direction it is needed to define the elastic modulus  $E$ , the tangential modulus  $G$ ,  
633 the tensile and compressive strength  $\sigma_t$  and  $\sigma_c$  and the tensile and compressive fracture energies  
634  $G_t$  and  $G_c$ . The pedeces  $h$  and  $v$  in each symbol identify the horizontal and vertical directions  
635 respectively. Once these parameters have been assigned, the calibration of the nonlinear  
636 orthogonal links of the interfaces follows a straightforward procedure related to the number of links  
637 and to the panel geometry. In Table 1, the mechanical characterization of the orthogonal nonlinear  
638 links of the interfaces (called *N-Links*) is expressed as a function of the main material parameters  
639 of the masonry wall.

640 This model allows for the separate calibration of the shear and flexural behaviors, which are not  
641 based on continuum mechanics.



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Figure 11 –Mechanical characterization of an orthotropic masonry panel: (a) constitutive laws and (b) calibration of the orthogonal Nlinks (adapted from Pantò et al., 2017a).

646

In general, it should be highlighted that continuum orthotropic models are less sensitive than isotropic models to the range of variation of the normal stress for the calibration.

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Finally, in the case of continuum constitutive models with strain-softening, the most problematic issue is related to the strain localization and mesh dependency and is worth mentioning.

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Table 1. Mechanical calibration of the orthogonal *N-links* for a rectangular panel with depth *s*

Direction	<i>elastic stiffness</i> horizontal $K_h$ vertical $K_v$	<i>compressive strength</i> horizontal $f_{ch}$ vertical $f_{cv}$	<i>tensile strength</i> horizontal $f_{th}$ vertical $f_{tv}$	<i>ultimate compressive displacement</i> horizontal $u_{ch}$ vertical $u_{cv}$	<i>ultimate tensile displacement</i> horizontal $u_{th}$ vertical $u_{tv}$
horizontal	$K_h = 2 \frac{E_h \lambda_h s}{B}$	$f_{ch} = \sigma_{ch} \lambda_h s$	$f_{th} = \sigma_{th} \lambda_h s$	$u_{ch} = 2 \frac{G_{ch}}{\sigma_{ch}}$	$u_{th} = 2 \frac{G_{th}}{\sigma_{th}}$
vertical	$K_v = 2 \frac{E_v \lambda_v s}{H}$	$f_{cv} = \sigma_{cv} \lambda_v s$	$f_{tv} = \sigma_{tv} \lambda_v s$	$u_{cv} = 2 \frac{G_{cv}}{\sigma_{cv}}$	$u_{tv} = 2 \frac{G_{tv}}{\sigma_{tv}}$

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653

The first dependency concerns the amount of dissipated energy. Upon the onset of strain localization, the strain localizes into a narrow band of elements. In the numerical model the width of such band is not ruled by parameters with clear physical meaning and is related to the characteristic size of the finite element. The amount of dissipated energy will therefore change as the mesh size changes. The second dependency concerns the direction of the crack. When the strain localizes into a band of finite elements, the boundaries of those elements will act (erroneously) as real boundaries, trapping the strain inside them, thus forcing the crack to follow the direction of the mesh rather than the real expected crack direction.

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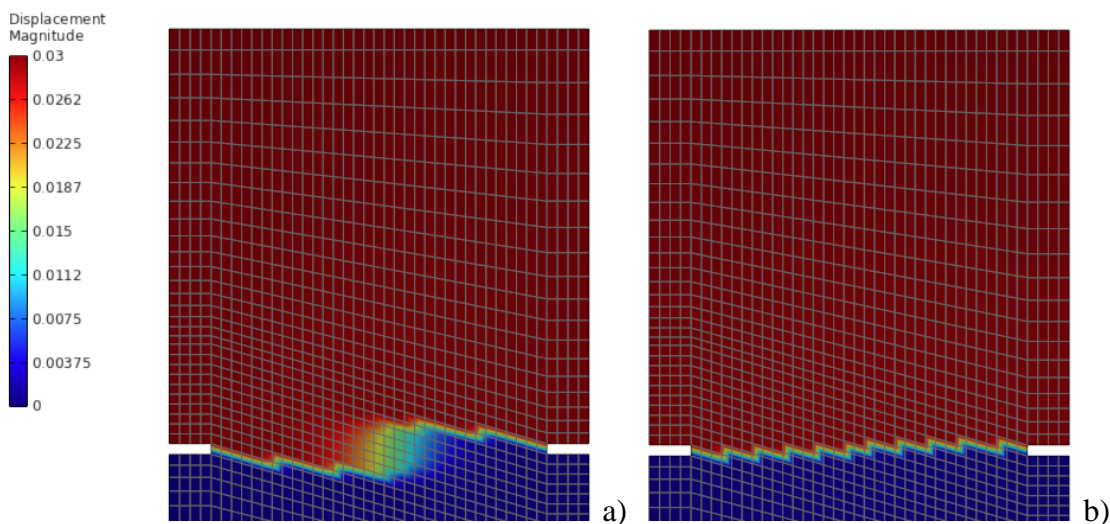
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661 To overcome the aforementioned problems, many algorithms were proposed in the literature.  
 662 Some effective strategies to tackle strain-localization issues, at present quite widespread and  
 663 nowadays considered classic, belong to the wide family of Non-Local and Gradient-Enhanced  
 664 models. They are relatively popular in masonry applications, but not necessarily the most effective  
 665 ones, especially when compared with other emerging numerical techniques. For instance, notable  
 666 examples with great potential to capture strong discontinuities are Finite Elements with elemental  
 667 (E-FEM) or nodal enrichments (X-FEM), see for instance Oliver et al. (2006). Their application  
 668 to masonry is subjected to continuous evolution to increase robustness and their progressive  
 669 diffusion in the scientific community is expected to grow rapidly. A variety of other specialized  
 670 techniques exist, e.g. Enhanced Assumed Strain (EAS) and mixed FEM. Among the others, crack  
 671 tracking algorithms deserve to be acknowledged, because of their promising efficiency and stability  
 672 in the analysis of real scale structural elements up to collapse (see Saloustros et al. 2018, 2019).  
 673 The use of tracking algorithms in combination with smeared crack approaches constitutes probably  
 674 the simplest solution to improve remarkably the prediction of the crack propagation direction,  
 675 avoiding the well know drawback of obtaining mesh-biased results (see Figure 12).



676  
 677 Figure 12 – a) Standard compatible FEM, the crack follows the mesh direction. b) Incompatible-modes  
 678 FEM, the crack follows the expected direction (based on the work of Simo and Rifai 1990)  
 679

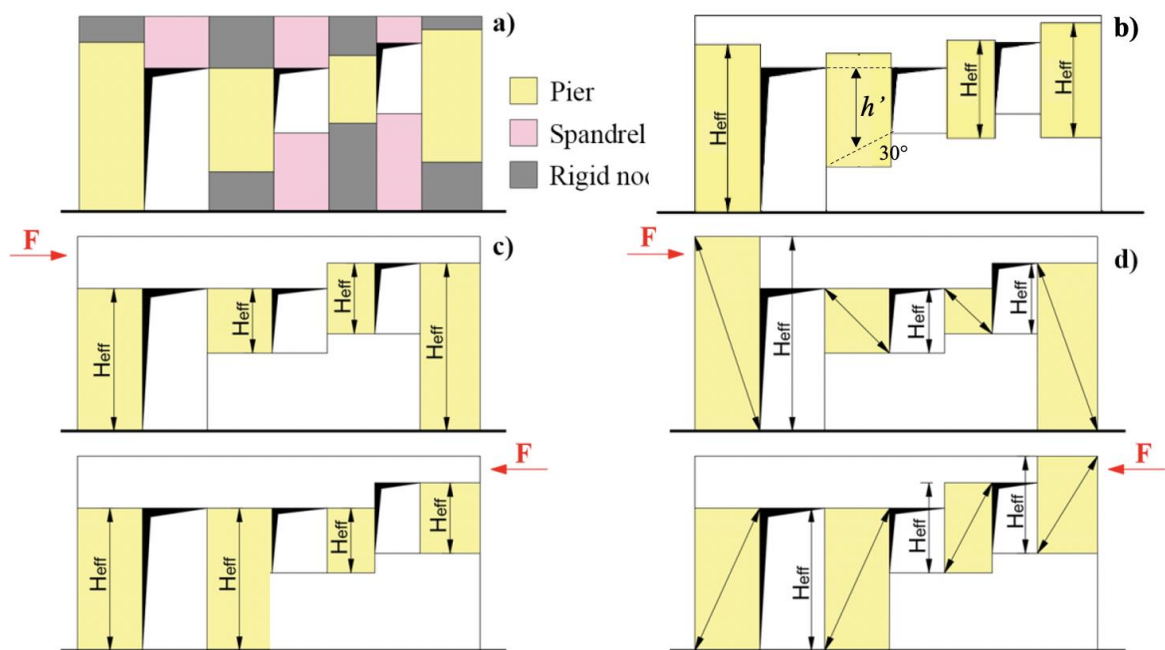
### 680 2.3 Critical issues in the idealization criteria of masonry walls according to EF approach

681 The rules adopted for the identification of the geometry of pier and spandrel elements are empirical  
 682 or based on limited experimentations and/or few numerical simulations. At present, an exhaustive  
 683 (*i.e.*, conclusive in providing unanimous scientific agreement) validation of their reliability is  
 684 missing in the literature, and codes also do not usually provide specific indications about the  
 685 criteria to use, leaving the analyst free to choose.

686 Most available rules are focused on piers and, specifically, to the definition of their effective height  
 687 ( $H_{eff}$ ), which represents a key modeling parameter in the EF approach. Figure 13 illustrates the



688 most popular approaches available in the literature; some of them are nowadays available in  
 689 commercial software packages specifically oriented to the seismic assessment of URM buildings.  
 690 Dolce (1991) proposed a simplified formula based on a series of linear elastic finite element  
 691 analyses on 20 different pier–spandrel systems, derived by considering a principle of statistic  
 692 equivalence between the elastic stiffness of EF and FE models. Moreover, a limit inclination of  
 693  $30^\circ$  was introduced for the cracks that start at the right or left corner of the openings and propagate  
 694 towards the opposite pier edges. The proposed formula is a function of a geometrical parameter  $h'$   
 695 (indicated in Figure 13b by way of example for one of piers), defined as the distance between the  
 696 midpoints of the lines connecting the vertices of two consecutive openings; according to Dolce  
 697 these lines have a limit inclination of  $30^\circ$ . The final effective height of the pier ( $H_{eff}$ ) is then  
 698 obtained by properly incrementing  $h'$ , accounting for the deformability of masonry above and  
 699 below the pier through an analytical expression that includes also the interstorey height.



700

701

702 Figure 13. (a) EF idealization of a masonry wall - pier's effective height according to Lagomarsino et al.  
 703 (2013); different criteria for the pier's effective height: (b) Dolce (1991), (c) Augenti (2006), (d) Moon  
 704 (2006)  
 705

706 The Tremuri software has implemented (Lagomarsino et al. 2013) a criterion similar to that of  
 707 Dolce's proposal but without the limitation of the maximum inclination of cracks (Figure 13a).  
 708 According to this criterion, pier elements are defined starting from the height of adjacent openings.  
 709 When these latter are perfectly aligned, the height is assumed equal to that of the openings.  
 710 Conversely, in presence of openings with different heights or external piers, the height is assumed  
 711 as the average of those of the adjacent openings or as the average between the inter-storey height

712 and the height of the opening. Furthermore, some works adopt a criterion similar to that proposed  
713 in Tremuri integrating the limitation of  $30^\circ$  proposed by Dolce (Bracchi et al. 2015, Rota et al.  
714 2014).

715 Other rules, available in the literature, take into account that the cyclic nature of the earthquake  
716 motion can induce a different failure pattern depending on the direction of the seismic forces, thus  
717 leading to a pier geometry that changes with the loading orientation. In particular, in Augenti  
718 (2006), on the basis of the damage observed in residential and school buildings after past  
719 earthquakes (Augenti and Parisi 2010), it is proposed to assume the pier effective height equal to  
720 the height of the opening that follows the pier in the direction of the seismic load (Figure 13c).  
721 Considering the results of quasi-static lateral loading tests on a full-scale 2-story URM building  
722 (Yi et al 2006), Moon et al (2006) proposed a pier effective height equal to the height over which  
723 a compression strut is likely to develop (Figure 13d). The compression strut is assumed to develop  
724 at the steepest possible angle joining the opposite free vertical edges of the pier; that is, the likely  
725 strut is that, among the others, which offers the minimum lateral resistance. This criterion is  
726 explicitly recommended in (NZSEE 2017). However, it is useful to highlight that the adoption of  
727 these two latter criteria requires alternative models for monotonic analyses along the X and Y  
728 directions, with a significant increase in the computational effort. The alternative could be the  
729 adoption of structural elements able to adapt the height according to the direction (positive or  
730 negative) of the horizontal loads; however, this option, at least to the Authors' knowledge, is not  
731 currently implemented in any commercial software package operating under the EF approach.

732 The analysis of the literature highlights that, while several criteria are proposed for the definition  
733 of the geometry of the piers, few indications are available for spandrels. While they can be easily  
734 obtained by considering the portions of masonry included between two vertically aligned openings  
735 for regular walls, their definition may become problematic in irregular layouts. An empirical  
736 criterion that provides indications also in the presence of irregularities is proposed in Lagomarsino  
737 et al. (2013). According to this rule, the idea is to conventionally assume a mean value for the  
738 effective length of spandrel elements as a function of the overlapping part between the openings  
739 at the two levels; when no overlap is present, or there are no openings at all, it is suggested to  
740 assume that portion of masonry as a rigid area.

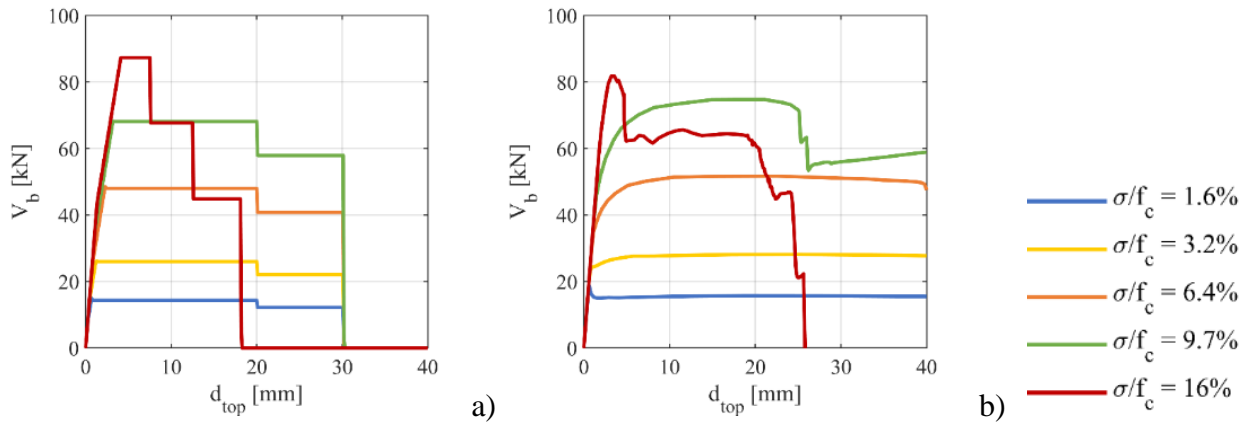
741 Various studies demonstrate how the criteria adopted for the geometry of the equivalent frame  
742 idealization of walls may represent, for analysts, one of the most significant uncertainties in the  
743 seismic assessment of complex existing buildings, corresponding to a high potential source of  
744 dispersion on the achievable results (see for example Bracchi et al. 2015, Manzini et al. 2021,  
745 Ottonelli et al. 2021).

746 The issue of defining the effective height of piers is also particularly challenging when the layout  
747 of the openings is irregular (*i.e.*, in the presence of openings with different heights on the same  
748 story or with a different number/position varying the level), which unfortunately represents a  
749 recurrent situation often observed in real cases. In recent years, many researchers have focused  
750 their attention on exploring the issue. Parisi and Augenti's (2013) work is one of the first to tackle  
751 such problem, proposing a classification of the different types of irregularities recurrent in  
752 masonry buildings and some irregularity indexes aimed at quantifying them. Other works (Berti  
753 et al. 2017, Siano et al. 2017, Siano et al. 2018, Camilletti et al. 2018, Camilletti 2019) aim to  
754 evaluate the accuracy of the EF approach when applied to irregular masonry walls by means of  
755 comparisons with refined modeling techniques in linear and nonlinear fields. In Calderoni et al.  
756 (2017) some suggestions for the application of the EF model to specific cases of irregular masonry  
757 walls are provided, even if they are not exhaustive. Despite that, the issue is still debated and open  
758 in the literature.

759 Adopting only the actual damage observed (*i.e.*, a purely empirical approach) as a reference for  
760 the definition and validation of these rules is complicated due to the huge variety of possible  
761 configurations. Thus, the numerical results carried out through analyses performed with refined  
762 models may be very useful and an essential support for validation aims. Some examples are  
763 discussed in the following by showing the results of a very detailed comparison between a refined  
764 model based on the use of the isotropic plastic-damaging 3D model implemented in ABAQUS  
765 and the EF model implemented in Tremuri software (Lagomarsino et al. 2013) by using the  
766 multilinear constitutive law proposed by Cattari and Lagomarsino (2013). The calibration  
767 procedure illustrated in Figure 7 and discussed in section 2.2 has been adopted to guarantee cross-  
768 consistency between the two approaches. Figure 14 presents the outcome of such a calibration for  
769 a panel characterized by a slenderness ratio equal to 2 and a fixed-fixed static scheme. For further  
770 details, interested readers may refer to (Camilletti 2019, Cattari et al. 2021a).

771 In particular, pushover analyses have been carried out on 2D models of various URM walls  
772 characterized by different opening layouts, very regular (Figure 15) or aimed at introducing  
773 different types of irregularity (Figure 16, named as B1 and BC).

774 For each wall, four EF models have been set by adopting the equivalent frame idealization rules  
775 proposed in Augenti (2006), Dolce (1991), Lagomarsino et al. (2013), and Moon et al. (2006). The  
776 comparison between the two modeling strategies is made at different levels in terms of: (i) overall  
777 pushover curve, (ii) damage pattern, (iii) generalized forces (either single values corresponding to  
778 specific steps of the analysis or their full variation during the analysis).

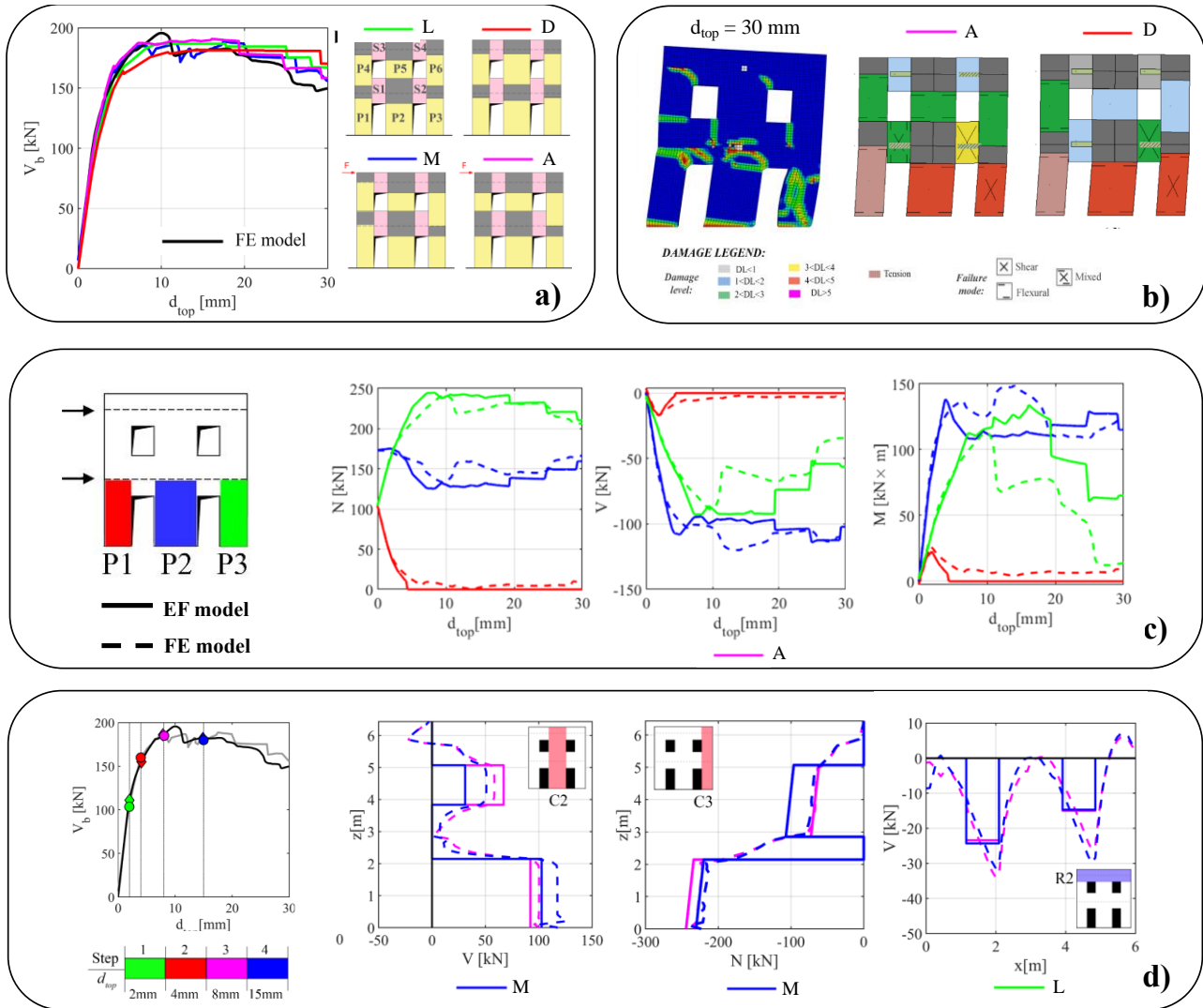


779  
 780 Figure 14 – Base shear-top displacement curves obtained on a single panel for different values of the  
 781 applied axial load with the EF model, based on the multilinear constitutive law implemented in Tremuri  
 782 program, (a) and with the FE model, based on the isotropic plastic-damaging 3D model implemented in  
 783 ABAQUS (b). These models are those used for the results presented in Figures 15 and 16.  
 784

785 Results on the very regular wall (Figure 15), whose geometry, load, and masonry type are inspired  
 786 by the “Door wall” tested by Magenes et al. (1995), show a very good agreement on all the  
 787 parameters checked and confirm that in this case, the dispersion of results in adopting different  
 788 rules is almost negligible (see also Cattari et al. 2021a).

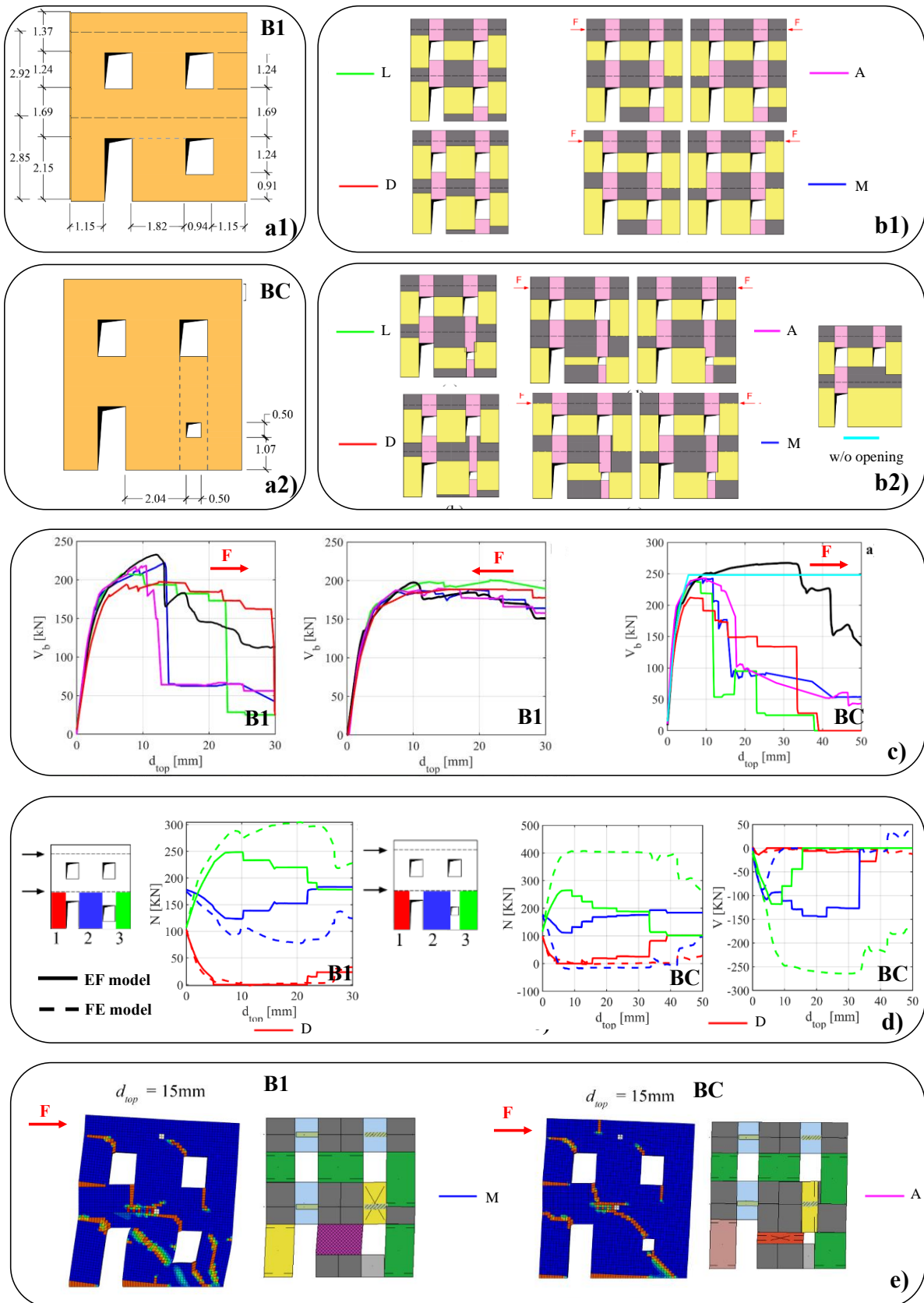
789 Concerning the other two cases investigated, at ground level, they introduce: (i) piers characterized  
 790 by a different height (case B1, Figure 16a1) and (ii) a very small opening (case BC, Figure 16a2).  
 791 The equivalent frame idealization of the wall B1 highlights that the application of the above-  
 792 mentioned rules is already capable of producing in such an apparently simple case a higher  
 793 dispersion in the effective height of piers than the very regular case of Figure 15. Wall BC  
 794 benchmark gives rise to the issue that accounting for all the openings present, independently by  
 795 their dimensions, may produce very squat piers (see Figure 16b2). Since the evaluation of the drift  
 796 is greatly influenced by the height, an acritical application of such rules may potentially result in  
 797 a significant underestimation of the ultimate displacement capacity of the structure; that is shown  
 798 in Figure 16c through the comparison of the pushover curves obtained by the EF models compared  
 799 with the target defined by the FE model (results refer to the positive direction). The same risk may  
 800 also occur in the B1 case when the rules proposed by Moon et al. (2006) and Augenti (2006) are  
 801 adopted (Figure 16b1). The comparison in terms of evolution of generalized force progressively  
 802 worsens, moving from case B1 to BC (Figure 16d). Finally, the analysis of the damage pattern,  
 803 particularly in case BC, confirms how the sudden drop off in the overall base shear is due to the  
 804 premature attainment of the collapse condition in the central squat pier. The diagonal crack  
 805 developed in the FE model suggests that, in this case, the very small opening is not able to  
 806 significantly affect the stress distribution in this masonry portion that, instead, behaves like a single  
 807 pier. In order to confirm such an outcome, a more reliable result is obtained in the EF model when  
 808 the small opening is completely neglected (see the light-blue curve in Figure 16c of case BC).

809 Even these results cannot conclusively define standardized rules to adopt, they only highlight the  
 810 potential of a cross-use of refined and EF models to this purpose in the future (see also Cattari et  
 811 al. 2021a).  
 812



813  
 814 Figure 15 – Comparison between EF and EF models for a regular wall: a) pushover curves and frame  
 815 idealization according to various rules (L, D, M, and A stand respectively for Lagomarsino et al. 2013,  
 816 Dolce 1991, Moon et al. 2006 and Augenti 2006); b) damage pattern; c) evolution of the axial load in the  
 817 piers at ground level; d) generalized forces for given steps of the pushover curve (adapted from Cattari et  
 818 al. 2021a)  
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Figure 16 – Comparison between EF and FE models for a wall with different height piers at ground level (B1) and a wall with a very small opening at ground level (BC) : a1/a2) wall geometry; b1/b2) frame idealization according to various rules (L, D, M and A stand respectively for Lagomarsino et al. 2013, Dolce 1991, Moon et al. 2006 and Augenti 2006); c) pushover curves; d) evolution of the axial load in the piers at ground level; e) damage pattern. Figures adapted from Camilletti 2019.

## 828 **2.4 Critical issues in the modeling of wall-to-wall connections**

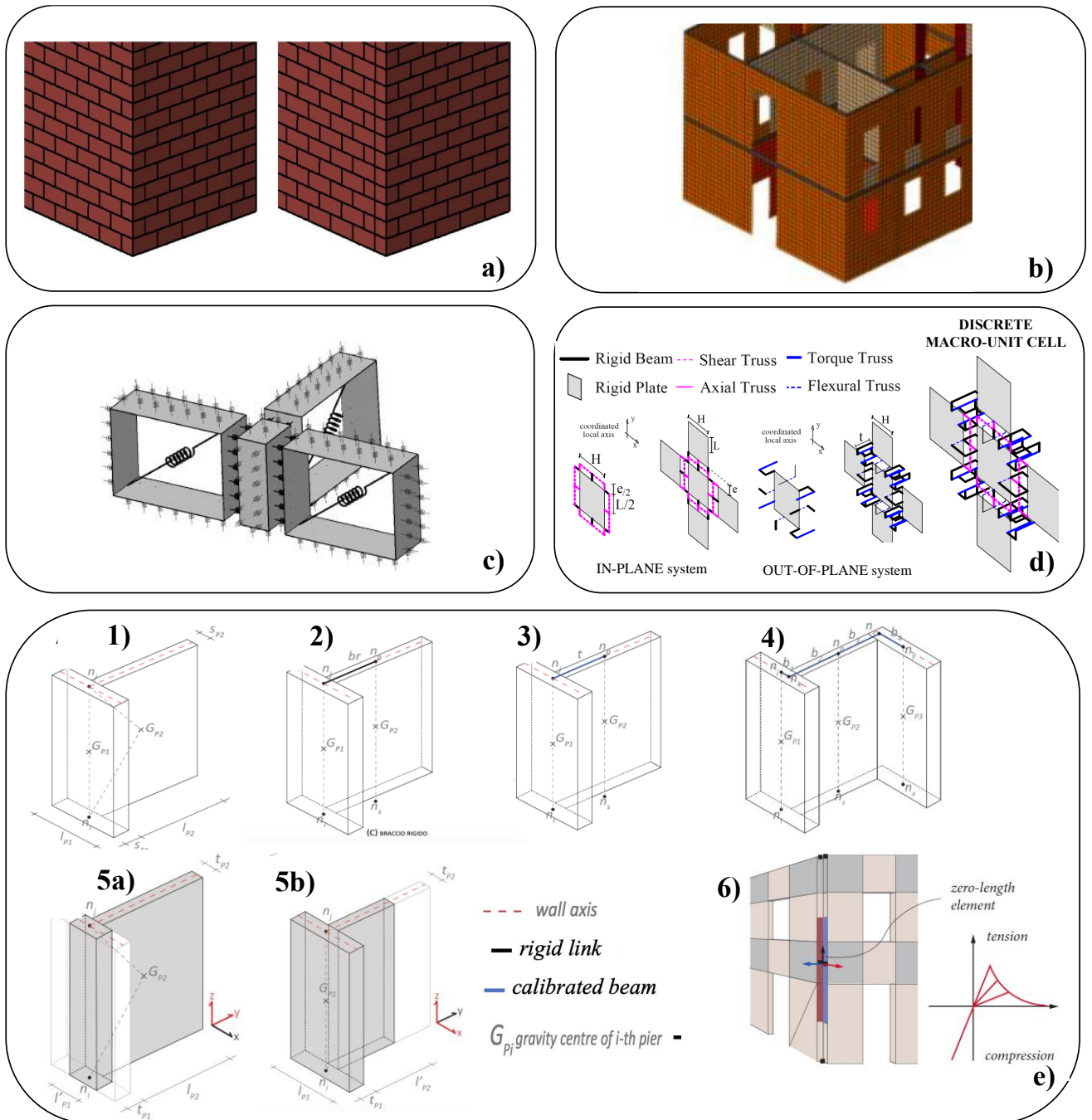
829 In order to build a 3D model and perform a global analysis, URM walls have to be properly  
830 connected, and different levels of effectiveness for the wall-to-wall connections should be  
831 accounted for to reproduce the large variety of existing buildings.

832 If the connection among walls is good, a possible redistribution of forces among intersecting piers  
833 may occur, generating the so-called “flange effect” (*i.e.*, forming piers with L-, C-, T-, or I-shaped  
834 cross-sections). The presence of such flanges can influence the in-plane response of the walls in  
835 terms of failure modes, maximum strength, and displacement capacity. As a consequence, the  
836 performance of the whole building, as highlighted, for instance, by experimental campaigns  
837 conducted on both single URM panels (Russell and Ingham 2010, Russell et al. 2014,  
838 Khanmohammadi et al. 2014, Sajid et al. 2018) and simple mock-ups (Costley and Abrams 1996,  
839 Paquette and Bruneau 2003, Moon et al. 2006) may vary too. Also, numerical simulations confirm  
840 experimental results (see Milosevic et al. 2020, Ottonelli et al. 2021, Tomic et al. 2021). For the  
841 aforementioned reasons, it is fundamental that the models employed for the design or the  
842 assessment can adequately account for these effects.

843 As depicted in Figure 17, such aspects may be accounted for in different ways depending on the  
844 modeling strategy.

845 While continuum models typically simulate a perfect connection (Figure 17b), only micromodels  
846 and block-based (*i.e.* a modelling approach where the masonry microstructure is explicitly  
847 modeled, and each microscopic behavior is described by its own nonlinear continuum constitutive  
848 model) models can consider the actual masonry texture, *i.e.*, the actual tothing between  
849 orthogonal walls (see Figure 17a). As an example, Figure 18 shows the numerical behavior of  
850 orthogonal walls with tothing obtained with a continuum micromodel considering bricks and  
851 mortar (Figure 18a) and a damaging block-based model (Figure 18b) separately.

852 In the plane discrete macro-element models (Caliò et al. 2012), the wall-to-wall connection is  
853 governed by corner elements that are endowed with interfaces and allow for the limitation of the  
854 coupling of orthogonal walls by adopting specific limit values, related to tensile and shear forces,  
855 according to specific constitutive laws. This approach can potentially grasps the typical nonlinear  
856 behaviour at the wall-to-wall connections with the development of a vertical crack due to  
857 detachment or sliding between the attached walls. In the three-dimensional macro-modeling  
858 (Figure 17c), this strategy also allows to consider the detachment between orthogonal walls when  
859 an out-of-plane mechanism is activated (Pantò et al. 2016, see also Section 2.6). The same strategy  
860 can be applied at the meso-scale: in this case, a block-based model can account for the actual  
861 masonry texture and, if efficiently calibrated, can provide a satisfactory prediction of the  
862 experimental response (Cannizzaro et al. 2017).



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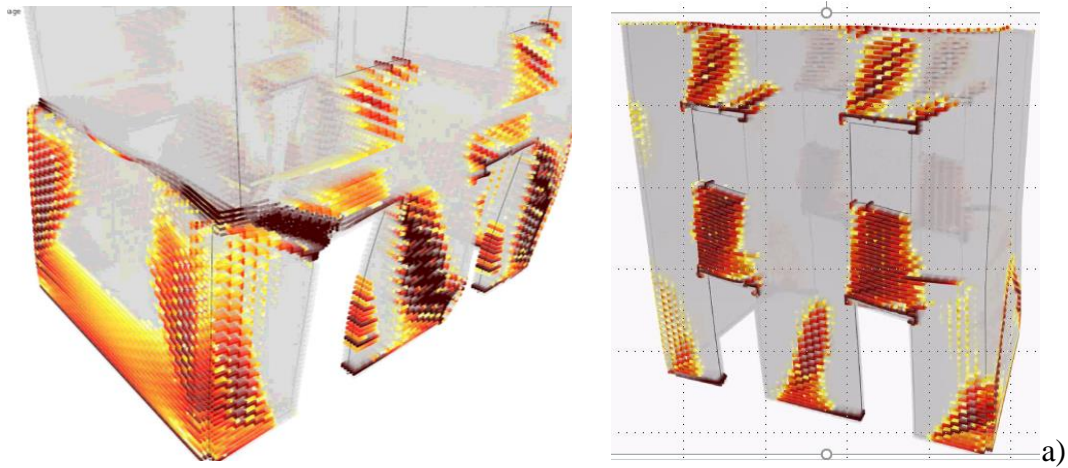
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Figure 17 – Possible strategies for accounting the walls-to-walls connection varying the modeling approach: a) micromodels and block-based models (from D’Altri et al. 2020b); b) continuum models (adapted from Castellazzi et al. 2021); c) corner elements connecting adjacent discrete-macro elements (adapted from Pantò et al. 2016); d) Rigid Body Spring Model, i.e., rigid elements joined by homogenized interfaces (adapted from Bertolesi et al. 2016); e) alternative options in EF models





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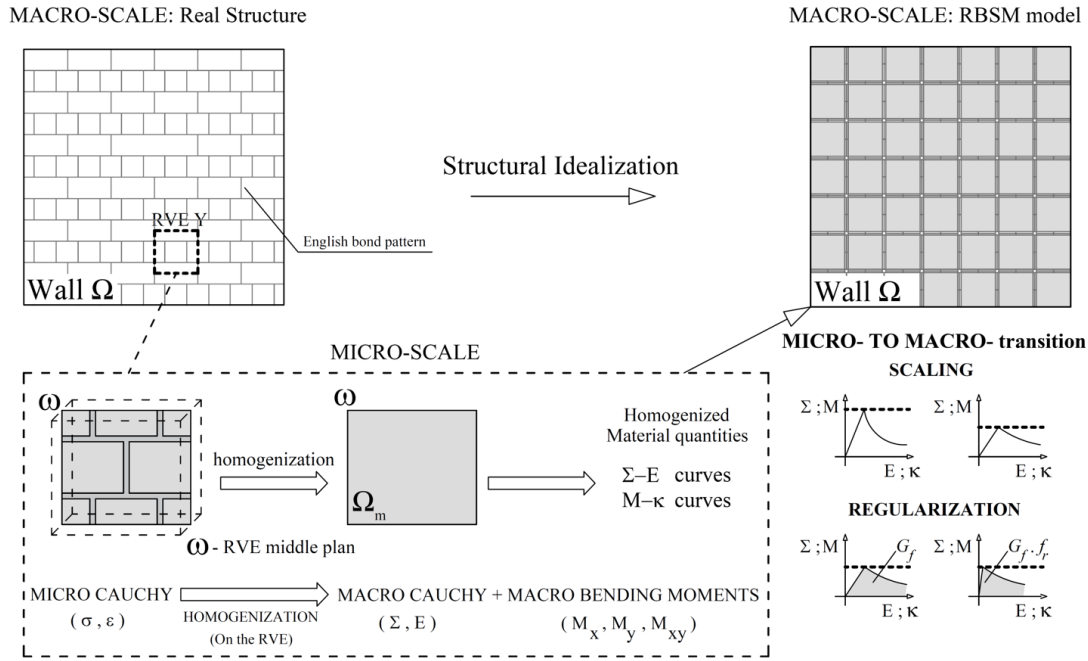
871

872 Figure 18 –Numerical examples of tothing behavior: (a) continuum micromodel considering the bricks  
 873 and mortar separately, and (b) damaging block-based model with cohesive-frictional contact (D’Altri et  
 874 al. 2019).

875 In the case of refined models, an alternative and effective approach to consider the degree of  
 876 interlocking between perpendicular walls is the combined use of homogenization and kinematic  
 877 limit analysis or an incremental approach based on the assumption of rigid elements joined by  
 878 homogenized interfaces (Figure 17d). The limit analysis case will be treated in detail later on in  
 879 the paper when dealing with the occurrence of local mechanisms (Section 2.6), whereas here the  
 880 attention is focused on the combination of homogenization and rigid elements interconnected by  
 881 nonlinear homogenized springs (Figure 19). At present, this procedure is conceived almost purely  
 882 for research purposes and is hardly utilizable immediately at professional level; however, efforts  
 883 are ongoing to automatize the required steps for making also practitioners able to use very  
 884 sophisticated approaches.

885 The strategy takes advantage of a classical first-order homogenization scheme, which relies on the  
 886 definition of a suitable boundary value problem able to provide the average (or homogenized)  
 887 nonlinear stress-strain behavior of a so-called unit cell that generates a masonry wall by repetition.  
 888 Interested readers should refer to Milani (2011) for further details on this theoretical topic applied

889 to masonry. The implementation at structural level occurs using a discrete element model,  
 890 hereafter designated as Rigid Body Spring Model (RBSM). RBSM is an assemblage of non-linear  
 891 homogenized springs and rigid bi-dimensional elements. The methodology and the main features  
 892 of the model at cell and structural level are briefly summarized in Figure 19.



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Legend of symbols:

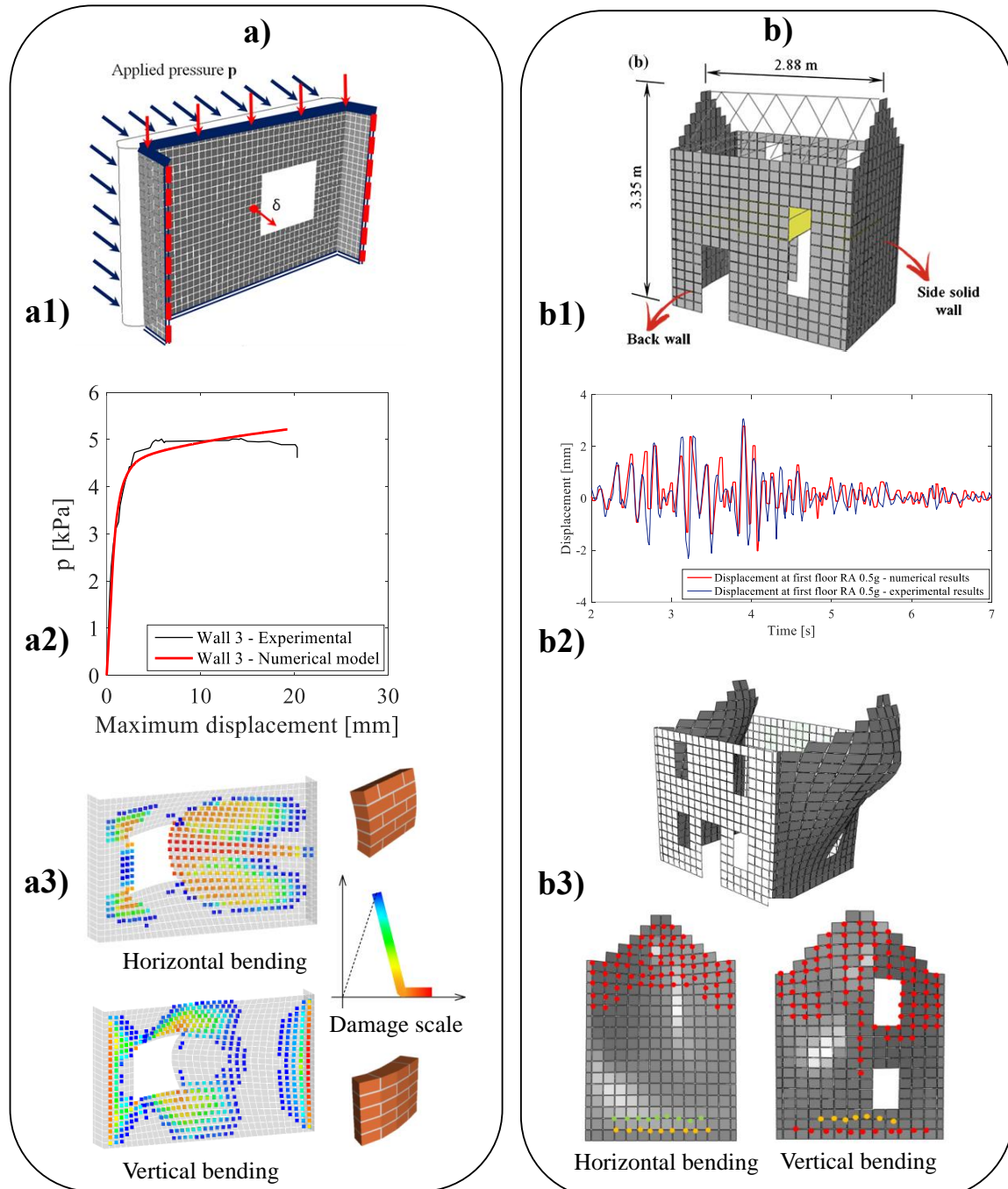
$E$	Macro-scale strain	$Y$	Boundary of the RVE
$f_r$	Flexural fracture energy coefficient	$e$	Micro-scale strain
$G_f$	Fracture energy in tension	$k$	Macro-scale curvature
$M$	Macro scale moment	$\sigma$	Micro-scale Cauchy stress
$M_x$	Horizontal macro scale bending moment	$w$	Unit cell volume
$M_y$	Vertical macro scale bending moment	$\Sigma$	Macro-scale Cauchy stress
$M_{xy}$	Macro-scale torsion	$W$	Macro-scale wall area
RVE	Representative Element of Volume	$W_m$	Micro-scale unit cell area

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Figure 19 –Methodology of a typical two-step numerical procedure to study 3D structures.  
Homogenization model and transition from micro to macro-scale.

899 Moving from micro-scale to macro-scale, some *ad-hoc* steps are developed for a correct upscaling,  
 900 *i.e.*, the scaling and regularization of the homogenized quantities. This is a critical step since it is  
 901 necessary for assuring that the macro-input is independent from the macro-mesh (Silva et al. 2020).  
 902 Indeed, accounting for the interlocking between perpendicular walls complicates the formulation  
 903 to a great extent. For illustrative purposes, Figure 20 shows how such approach has proven  
 904 effectiveness in the numerical simulation of experimental tests carried out under both static (Figure  
 905 20a) and dynamic conditions (Figure 20b). In particular, Figure 20a illustrates an application to a  
 906 wall specimen with openings tested by Griffith et al. (2007) that exhibited a good interlocking with  
 907 perpendicular walls; interested readers to more details on this numerical simulation should refer  
 908 to Bertolesi et al. (2016) and Bertolesi et al. (2019). Instead, Figure 20b shows an extract of some  
 909 numerical simulations performed by Bertolesi et al. (2018): it deals with a two-story masonry

910 building lab prototype tested in the nonlinear dynamic field by Bothara et al. (2010). In particular,  
 911 Figure 20b2 depicts a comparison between control point experimental and numerical  
 912 displacements during the application of a real accelerogram (Taft earthquake) with maximum  
 913 acceleration scaled to 0.5g. Figure 20b3 shows the damage cumulated on out-of-plane loaded walls  
 914 for horizontal/vertical bending at ground acceleration peak.  
 915



916  
 917 Figure 20 –Examples of application of homogenization combined with a Rigid Body and Spring Model  
 918 (RBSM) in: a) the non-linear static case by simulating the wall with opening tested by Griffith et al.  
 919 (2007); b) the non-linear dynamic case by simulating the two-story masonry building tested by Bothara et  
 920 al. (2010) (figures of the numerical simulation adapted from Bertolesi et al. 2016 and 2019 in case a) and  
 921 from Bertolesi et al. 2018 in case b). In both cases the damage for horizontal/vertical bending are reported  
 922 (a3 and b3) together with the comparison in terms of pushover curves (a2) or displacement at the first  
 923 floor (b2)

924  
925 Finally, it is worth mentioning that in the EF approach, the connection among intersecting walls  
926 is simulated through various implementation strategies varying with the software packages used  
927 (as depicted in Figure 17e).

928 Considering the general case of Figure 17e case 4), of which cases 1), 2) and 3) are particular  
929 cases, a kinematic coupling among incident walls can be simulated through different solutions  
930 such as the use of kinematic constraints and the consequent condensation of the degrees of freedom  
931 (Figure 17 case 1 or case 2). In general, this option, frequently adopted as default by commercial  
932 software packages (as discussed in Cattari and Magenes 2021), can be then edited, allowing: (i)  
933 the passage from the perfect kinematic coupling to the use of “equivalent” elastic beams of finite  
934 stiffness (Figure 17e3) and (ii) the deletion of the rigid link (thus downgrading the full coupling  
935 to a null wall-to-wall connection). When “equivalent elastic beams” are adopted, the effectiveness  
936 of wall-to-wall connections may be managed through a proper calibration made by the user of the  
937 stiffness parameters of the coupling beam (as discussed in more detail in Ottonelli et al. 2021). In  
938 the general case of Figure 17e case 4) where more than two panels are intersecting to form a  
939 complex section, a common option is to define one simple vertical element per each panel (three  
940 in the particular case of the figure). The nodes at the top of each element can be connected to each  
941 other via rigid links (kinematic constraints) or calibrated beams (links or elements  $b_1$ ,  $b_2$ ,  $b_3$  and  $b_4$   
942 in the figure) creating hence composite actions among the panels (the same can be applied for the  
943 bottom nodes of the elements). When such a solution is adopted, it is very important to consider  
944 that, unless rotational releases are introduced at nodes  $n_k$  between links  $b_1$  and  $b_2$ , and at node  $n_z$   
945 between links  $b_3$  and  $b_4$ , the free warping of the composite section would be restrained, increasing  
946 dramatically (and fictitiously) the stiffness of the composite wall (especially the torsional  
947 stiffness).

948 When using the EF approach in general-purpose software packages, the equivalent frame model  
949 is implemented directly by the user, and a common practical way to simulate the collaboration  
950 effect among intersecting walls must be defined. Depending on the direction of loading, an  
951 “effective flange width” must be considered collaborating with the “web” of the composite section.  
952 The section properties of the equivalent frame element are thus defined in terms of moment of  
953 inertia, cross-sectional area, effective shear area, as it would be done for T- or I-shaped sections.  
954 Such approach clearly produces different section properties depending on the direction of loading  
955 (as shown in Figure 17e, cases 5a for a shear force in the y-direction, and 5b for the x-direction).  
956 Finally, Figure 17e6 presents the approach adopted in the equivalent frame model by Vanin et al.  
957 (2020a, 2020b) and implemented in OpenSees (McKenna et al. 2000). In this case, the connection  
958 between orthogonal walls is modeled through zero-length elements. Such connection can also

959 exhibit a nonlinear behavior (thus giving the possibility to reproduce the development of a vertical  
960 crack as also discussed in section 2.6, Figure 23) and can be modeled either through point  
961 connections at the corner nodes (to which appropriate tensile properties are then assigned) or  
962 through the use of fiber sections.

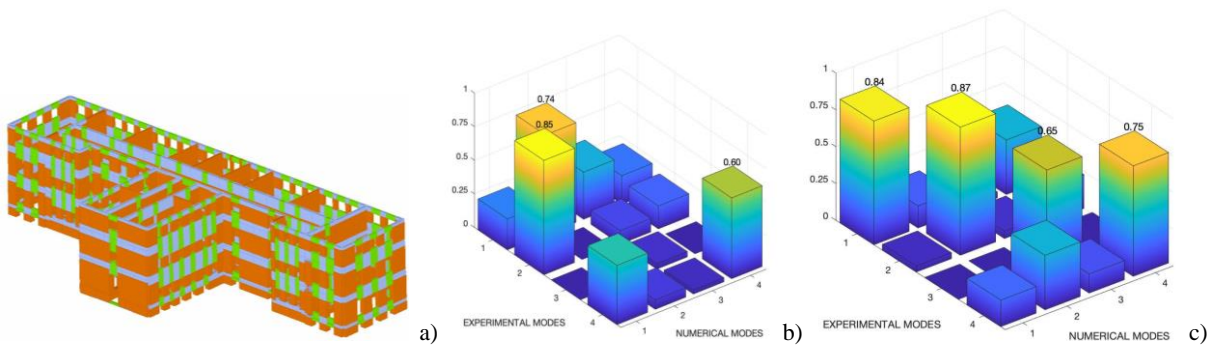
963 When the strategy presented in Figure 17e3 is adopted, the calibration of the equivalent beams  
964 used to simulate different degrees of effectiveness of the wall-to-wall connections becomes quite  
965 difficult since it should account for the geometry and properties of the incident piers and the  
966 masonry typology (*i.e.*, dimensions of block and bond type). Despite that, there are no specific  
967 indications in the literature to properly calibrate the stiffness of such beams, which is usually  
968 determined through empirical approaches. In Ottonelli et al. (2021), the “starting reference value”  
969 of the stiffness of the equivalent beams has been calibrated in order to reproduce the same solution  
970 obtained in the case of perfect coupling; then, from that value, a progressive reduction of the  
971 moments of inertia has been applied in order to reproduce various levels of effectiveness of the  
972 wall-to-wall connection (see also Milosevic et al. 2020).

973 The effective width to be considered for the flange (essential for applying the strategy of Figure  
974 17d5a/b and also for the equivalent beam calibration) still represents a critical open issue. Most  
975 works available in the literature focus on the determination of the effective flange width in  
976 reinforced concrete and reinforced masonry shear walls (*e.g.*, Priestley and He 1995, Hassan and  
977 EI-Tawil 2003, Shi and Wang 2016), but the rationales employed are different for different  
978 materials. For example, the effective flange width of a reinforced concrete shear wall increases  
979 due to the yielding of the reinforcement, and obviously this feature cannot be applied to URM  
980 walls. Some indications specifically conceived for URM walls have been proposed in (Yi 2004,  
981 EC6 -Part 1-1 CEN 2004, MSJC 2008, Mordant 2016, and Calderoni et al. 2019b), but research  
982 into this very specific topic is still ongoing.

983 Regardless of these difficulties, it is worth being aware of the high potential sensitivity of the  
984 achievable results, particularly in the case of EF models.

985 When the connection between the orthogonal walls is managed through the perfect coupling of the  
986 vertical component or with rigid links (*i.e.*, regardless of the dimension of the flange), this may  
987 lead to an overestimation of the flange collaboration in the presence of long intersecting walls  
988 which imply very long effective flange widths (*e.g.*, the case of very long and squat walls without  
989 openings), where it seems unrealistic to consider the whole width as effective. As an example,  
990 Figure 21 shows some results referred to the case of the Fabriano Courthouse (Italy), recently  
991 discussed in Cattari et al. 2021b. This building is permanently monitored by the Department of  
992 Civil Protection (DPC, Dolce et al. 2017) and the figure shows the comparison of the experimental

993 modes, as evaluated from the data provided by the DPC, and those numerically simulated by the  
994 EF frame model presented in Cattari et al. (2021b).



995  
996 Figure 21 – a) EF model of Fabriano Courthouse. Comparison of simulated and experimental modes in  
997 terms of MAC index in case of perfect (b) or calibrated (c) coupling among walls (adapted from Cattari et  
998 al. 2021b)

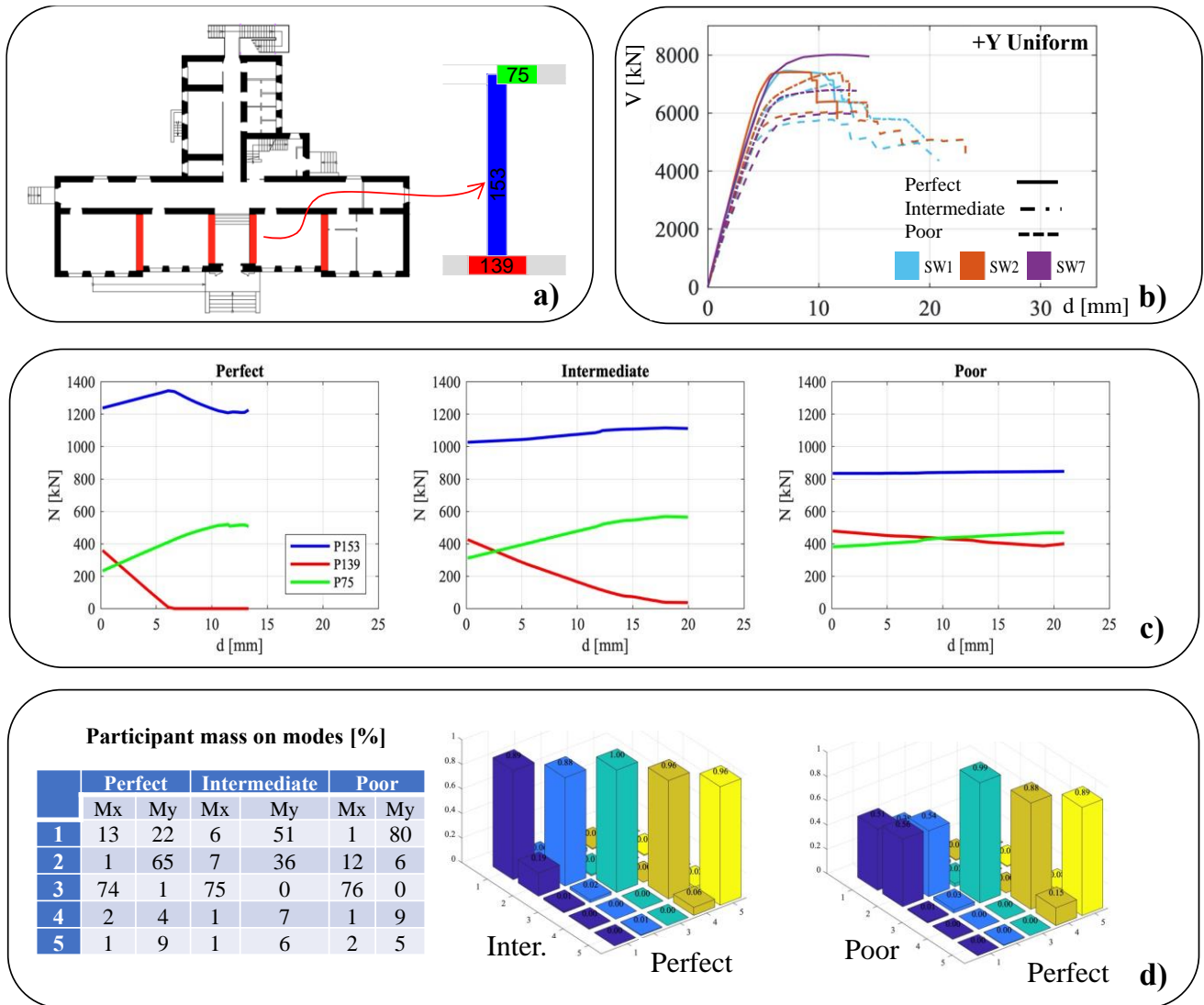
999 The comparison is made in terms of MAC (Modal Assurance Criterion) index (Allemange and  
1000 Brown 1982), where values close to 1 indicate a very good agreement between experimental and  
1001 numerical data, and the optimal result consists of having unitary values along the diagonal. It is  
1002 interesting to observe that the option associated with the full kinematic coupling (Figure 21b)  
1003 produces a very high discrepancy in terms of mode shapes. A correct prediction is obtained using  
1004 equivalent beams of finite stiffness calibrated to simulate a good quality of connection. This latter  
1005 hypothesis is in agreement with the actual configuration of the building, where strengthening  
1006 interventions were introduced to improve such specific structural aspect.

1007 Finally, Figure 22 provides a more comprehensive overview on the potential repercussions of  
1008 alternative assumptions on the quality of the wall connections by assuming as reference the  
1009 benchmark structure (BS5) inspired by the “P. Capuzi” school of Visso (MC, Italy) and analysed  
1010 in Ottonelli et al. (2021) within the scope of the *URM Nonlinear modeling*-Benchmark project.

1011 Passing from the perfect connection to the poor connection assumption among walls, the results  
1012 show a significant variation in terms of: (1) pushover curves (Figure 22b); (2) value of the axial  
1013 load on piers composing the flange system (Figure 22c), referring to both the initial value, after  
1014 the application of gravity loads, and its variation due to the application of horizontal forces; (3)  
1015 mode shapes and participant masses (Figure 22d, considering the first three modes). Results  
1016 presented in Figure 22b have been obtained using three different software packages (SWs) that  
1017 adopt different strategies to simulate the walls-to-walls connection. Such outcome also  
1018 demonstrates that the numerical procedures depicted in Figure 17e3 (used in SW1 and SW2) and  
1019 in Figure 17e5 (used in SW7) may produce analogous predictions, if consistently implemented.  
1020 Further details on this issue may be found in Ottonelli et al. (2021). Of course, it is worth  
1021 specifying that the actual variation in the results depends on the specific structural configuration  
1022 of each building.

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Figure 22 – Overview of potential repercussions of alternative assumptions on the quality of wall connections for BS5 (figures adapted and integrated from Ottonelli et al. 2021): a) in plan view of the BS5 and identification of piers analysed in c); b) pushover curves; c) axial load variation on piers; d) mode shape variation.

## 2.5 Critical issues in modeling the out-of-plane response of masonry walls

Modeling the out-of-plane response of URM walls in full 3D seismic analyses involves two main issues. The first deals with the contribution of the out-of-plane stiffness and pier strength to the overall horizontal force equilibrium. The second deals with properly accounting for the possible activation of out-of-plane failure mechanisms of walls (usually called “local mechanisms”, for whose an overview of the most recurring ones is provided in D’Ayala and Speranza 2003), which, at collapse, are ruled more by the loss of equilibrium rather than the attainment of the material strength limits. Particularly in the second case, the difficulty lies in modeling both the in-plane and

1039 out-of-plane responses in an integrated, efficient, and accurate way to also account for their  
1040 possible combined effects.

1041 In the following, these issues are first discussed for EF models.

1042 The pier out-of-plane stiffness and strength are included in most commercial software packages  
1043 based on this modeling approach (see also Cattari and Magenes 2021). For the strength, similarly  
1044 to the in-plane flexural response, the ultimate bending out-of-plane capacity is usually computed  
1045 based on the axial load applied to the panel, neglecting the masonry tensile strength and assuming  
1046 an equivalent rectangular stress block distribution of stresses. Since this contribution depends on  
1047 the axial load, it has to be highlighted that the latter is also dependent on the modeling of the flange  
1048 effect. As also discussed in Ottonelli et al. (2021), for traditional buildings and for wall thicknesses  
1049 lower than 0.40 m, the incidence of such out-of-plane contribution can be considered as not  
1050 particularly significant, and it is conservative to ignore it. Conversely, for thicknesses greater than  
1051 0.40 m, the contribution of the out-of-plane response becomes progressively more significant, and  
1052 neglecting it could lead to appreciable variations not only on the overall base shear capacity but  
1053 also on the initial stiffness and ultimate displacement capacity. In Ottonelli et al. 2021, for the  
1054 complex URM building of Figure 22, the incidence of both the out-of-plane contribution and the  
1055 flange effect is discussed by considering alternative options for each single modeling hypothesis  
1056 and by considering their combination.

1057 Passing to the second issue (*i.e.*, the simulation of “local mechanisms”), according to the most  
1058 common procedures accepted by the scientific community, EF approaches are considered  
1059 unsuitable for directly dealing with the partial collapses of out-of-plane loaded masonry walls and  
1060 for the hybrid in- and out-of-plane failures of portions of entire buildings. In general, it is assumed  
1061 that the EF approach alone can provide reliable and exhaustive information only when the out-of-  
1062 plane response is not critical, for example, if the walls are tied together at all story levels.  
1063 Conversely, if local mechanisms are likely to occur, a common approach is to adopt the method  
1064 of rigid body limit analysis with pre-assigned failure mechanisms (as discussed in the following).  
1065 Then the results of the global in-plane response made by the EF model and those provided by the  
1066 rigid body limit analysis have to be combined. The simplest approach is to consider the final safety  
1067 index as the worst between the two ones (see for example Simoes et al. 2014), but in the context  
1068 of developing fragility curves some attempts to integrate the two failures modes in a more accurate  
1069 way – even if still analysing them in a separate way - have been also developed (see for example  
1070 Angiolilli et al. 2021).

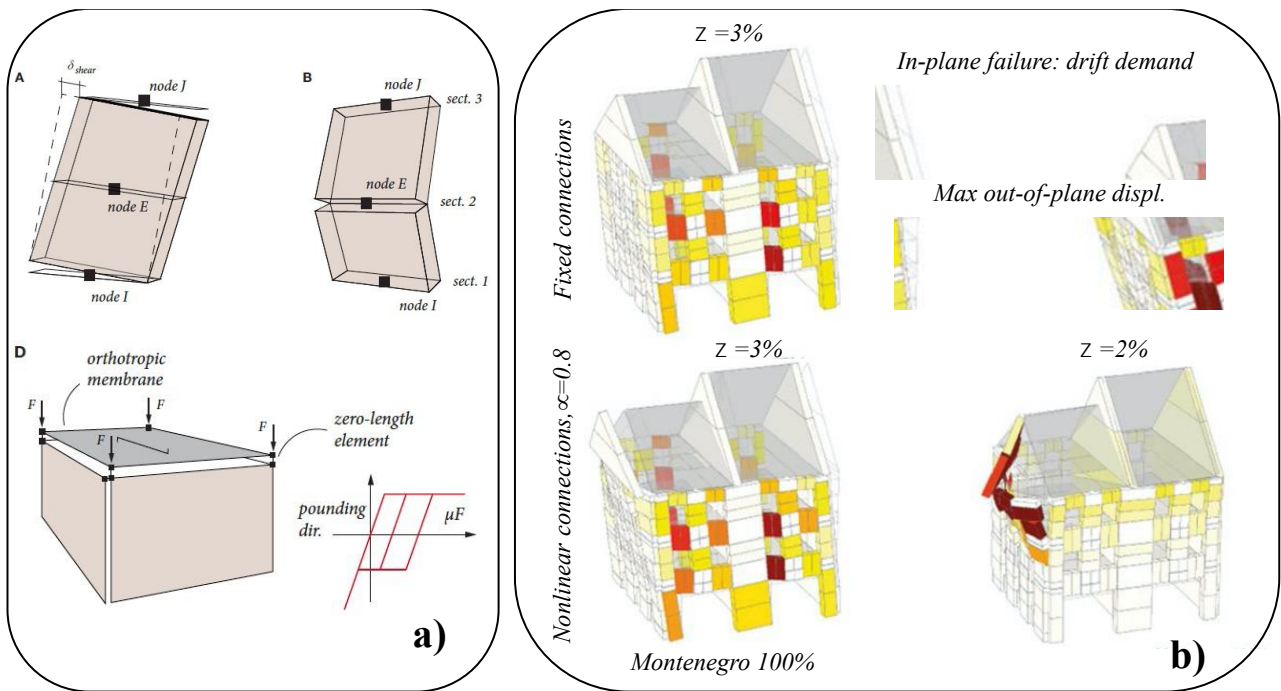
1071 A preliminary attempt to fill the gap between EF limitations and the possibility to predict out-of-  
1072 plane collapses (albeit in an approximative way) has been recently provided in the approach  
1073 implemented by Vanin et al. (2020a, 2020b) and already introduced in Section 2.5. This approach



1074 couples equivalent frames with possible out-of-plane failures of single elements. Figure 23  
 1075 illustrates the basis of this method with an application that highlights the sensitivity of the out-of-  
 1076 plane response to alternative hypotheses relative to the wall-to-wall connection. Unfortunately,  
 1077 this approach still does not account for very complex partial failure mechanisms observed in post-  
 1078 earthquake surveys, despite steps forward have been done to deal with the task in its full  
 1079 complexity.

1080 After a thorough revision of the literature available, it is the Authors' opinion that the possibility  
 1081 to include out-of-plane failures in EF models still needs substantial improvements.

1082



1083 Figure 23 – a) Basics of the equivalent frame approach implemented by Vanin et al. (2020); b) sensitivity  
 1084 of the in-plane and out-of-plane damage simulated to the effectiveness of walls connections and damping  
 1085 ratio by performing NLTHA (adapted from Vanin et al. 2020b)  
 1086

1087 The main reason for these difficulties stands in the awareness that experimental tests carried out  
 1088 since the 70's on laterally loaded brick masonry walls show that failure occurs along crack lines,  
 1089 whose patterns are only in some cases consistent with the simple overturning of the walls or the  
 1090 formation of a horizontal cylindrical hinge (as in simple one-way vertical or horizontal bending).  
 1091 In the past, these results inspired the utilization of approximate analytical solutions, based on the  
 1092 classic yield line theory, that in practice is an application of the classic kinematic theorem of limit  
 1093 analysis. Interested readers may refer to Lagomarsino (2015), Giresini et al. (2015), Abrams et al.  
 1094 (2017), Casapulla et al. (2017), Sorrentino et al. (2017) and Degli Abati et al. (2021), for a review  
 1095 of the methods proposed in the literature and Codes belonging to this approach and for some  
 1096 scientific studies addressed to validate them. The Italian Code (MIT 2019), for instance, explicitly  
 1097 requires the examination of such collapse modes and suggests the kinematic limit analysis as a  
 1098

1099 robust verification tool where masonry can be modeled as a no-tension material. Unfortunately,  
1100 this last assumption on the material strength (no-tension) does not take into account a number of  
1101 mechanical features of masonry that can influence the out-of-plane response. First of all, masonry  
1102 is a heterogeneous composite material (made of bricks or blocks and mortar) that, according to  
1103 experimental evidence, may exhibit in several cases a non-isotropic behavior both in the pseudo-  
1104 elastic field (service conditions) and at failure. Additionally, masonry tensile strength is not strictly  
1105 zero, and despite being low, highly scattered and with a quasi-brittle post-peak behavior, it can  
1106 affect significantly the results in several cases. In addition, the infinite compression strength  
1107 assumption made for rigid blocks is, in such cases, questionable, and currently, there are some  
1108 approaches available that can account for these features in a reasonable manner, without excessive  
1109 computational burden. In any case, brittle crushing phenomena are typically considered to be of  
1110 minor importance in an out-of-plane analysis of walls at failure. Another crucial aspect that  
1111 requires particular attention is the role played by the friction coefficient  $\mu$  of mortar joints.  
1112 According to several experiments (*e.g.* Atkinson et al. 1989 and Andreotti et al. 2019), the friction  
1113 coefficient appears relatively high, in general above 0.4 (with the exception of particular cases  
1114 such as in presence of damp proof courses) thus excluding sliding phenomena (at least out-of-  
1115 plane) in the majority of the cases, favoring the formation of torsional and flexural hinges. In such  
1116 case classic limit analysis can be applied without the risk to obtain inaccurate results from an  
1117 engineering standpoint. The role of the stabilizing role of the friction between interlocked walls  
1118 compared to other extrinsic or intrinsic loading capacity (*e.g.* the effect of tie-rods and simply  
1119 supported horizontal diaphragms with frictional resistance) has been discussed in Casapulla and  
1120 Argiento (2016). Moreover, in Casapulla et al. (2021) a macro-block model accounting for  
1121 frictional resistance has been used to calculate the onset load factors in multi-storey buildings for  
1122 two classes of local mechanisms which are particularly useful in engineering applications, like as  
1123 the rocking-sliding and the flexure mechanisms.

1124 Finally, the use of heterogeneous approaches or macroscopic nonlinear orthotropic models is at  
1125 present too demanding at professional level due to the excessive computational effort needed and  
1126 the expected weak background of the typical user in the definition of the many material parameters  
1127 needed for blocks, mortar or the macroscopic materials representing masonry to apply in the non-  
1128 linear range.

1129 Despite the aforementioned approximations and the awareness that masonry does not behave as a  
1130 rigid-plastic material, limit analysis remains a possible tool that is fast and simple enough for  
1131 assessing both the ultimate load capacity of masonry walls and the corresponding active failure  
1132 mechanisms. Thanks to its simplicity and the very limited number of parameters required, limit

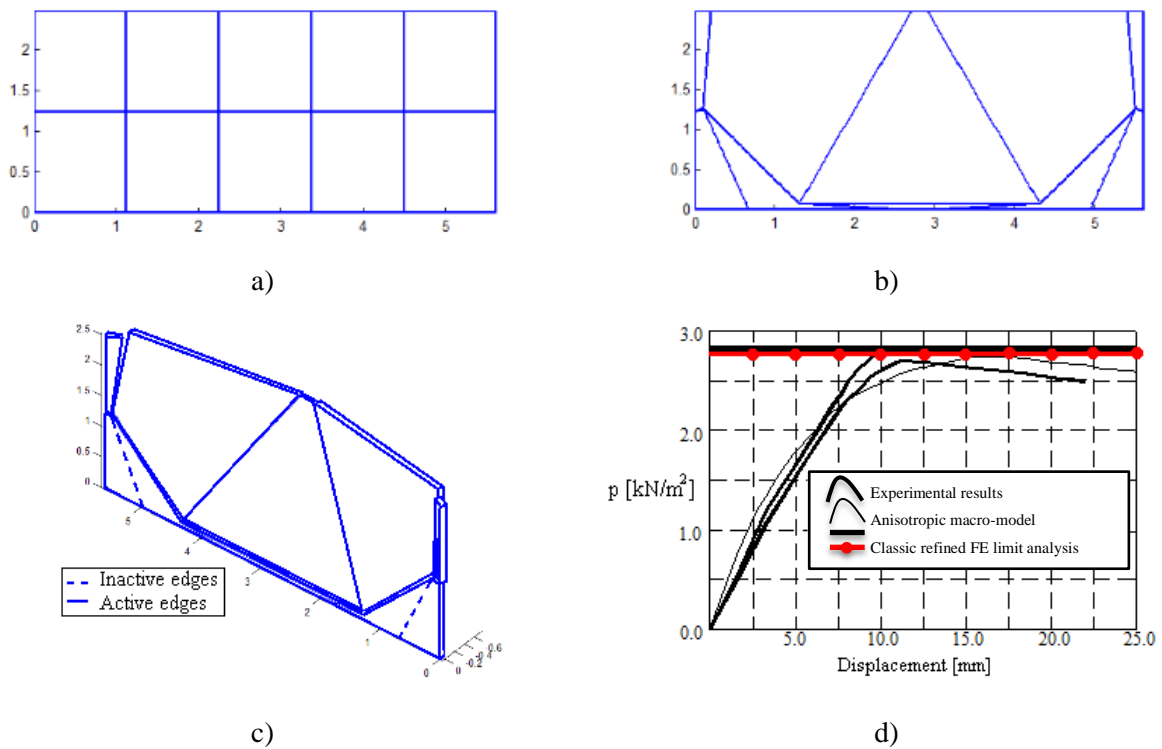
1133 analysis was selected over the last few decades as a standard procedure for evaluating failure  
1134 modes and ultimate bearing capacity of masonry in bending.

1135 According to the Authors' opinion, the main issues to investigate are the following: (1) to maintain  
1136 the analysis with a limited computational burden, (2) to accurately reproduce the crack pattern  
1137 developing during the formation of an out-of-plane failure mechanism, (3) to use a model that is  
1138 easily understandable by practitioners and finally (4) to take into account the most important  
1139 features of the masonry material under consideration, such as orthotropy, irregular assemblages of  
1140 stones/blocks, and the possible presence of multi-leaf walls.

1141 Recently, a first attempt has been made to define the pre-assigned failure mechanisms by drawing  
1142 the mesh with CAD. The mesh is imported in a standard FEM software using refined discretization  
1143 and recursively applying the principle of virtual works on a number of different mechanisms to  
1144 closely approximate the actual failure mechanism. It is important also to account for the real  
1145 geometry, load distribution and internal dissipation of the masonry material (Milani 2019). The  
1146 approach proved to be efficient for masonry towers, but additional work is needed to deal with  
1147 more complex historical structures.

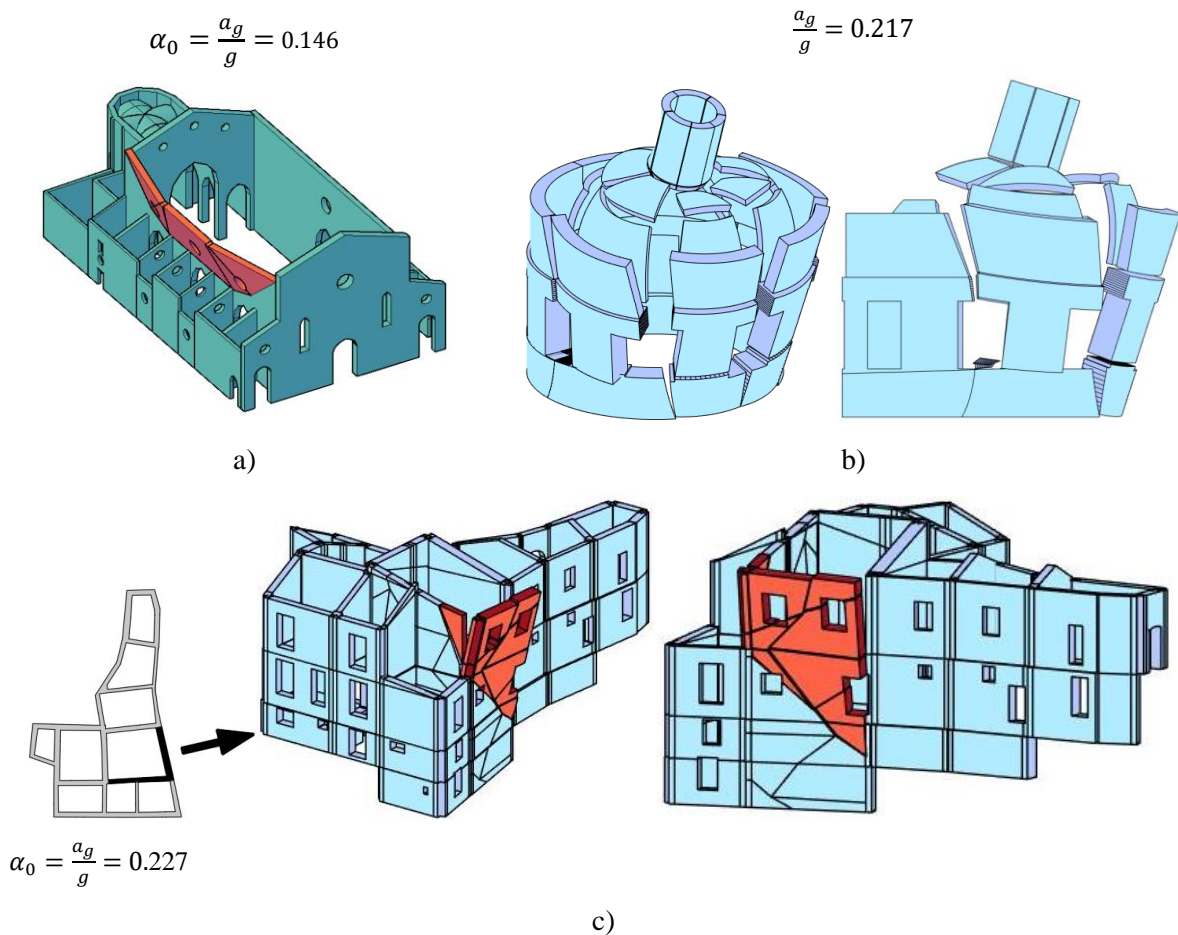
1148 The most recent literature in the general field of existing structures moves in another direction. It  
1149 suggests using numerical models conceived *ad-hoc*, where masonry is assumed as rigid plastic, so  
1150 the classic theorems of limit analyses still hold, *i.e.*, choosing to delegate the detection of the failure  
1151 mechanism to a numerical algorithm (Chiozzi et al. 2017). To circumvent the important drawback  
1152 of applying the kinematic approach of limit analysis, which needs to assign *a-priori* a failure  
1153 mechanism, recent literature proposes using very rough Finite Element FE meshes of the particular  
1154 portion of the structure that needs to be investigated. It also suggests a possible progressive  
1155 adjustment of the nodes in order to overlap step-by-step the plastic dissipation of the numerical  
1156 model with the actual one, thus accurately reproducing the actual failure mechanism with a  
1157 realistic assumption of the material properties to assign for masonry, deduced from heuristic,  
1158 compatible, or rigorous homogenization techniques. Examples can be found in some seminal  
1159 papers in the field (see, *e.g.*, Chiozzi et al. 2019a and 2019b), where adaptive curved/NURBS  
1160 finite elements with orthotropic behavior have been used to reproduce with a good level of  
1161 accuracy the activation of two way bending mechanisms for walls as in Figure 24 (Chiozzi 2019b),  
1162 churches as in Figure 25a (Chiozzi et al. 2019a), domes with complex geometries as in Figure 25b  
1163 (Grillanda et al. 2019), and masonry aggregates as in Figure 25c (Grillanda et al. 2020a). The  
1164 adaptation has been carried out either with Sequential Linear Programming or with non-standard  
1165 Genetic Algorithms and Meta-Heuristic approaches in general. The interested reader should refer  
1166 to (Milani 2015) and (Grillanda et al. 2020b) for further details. This approach could also be

1167 coupled with displacement-based evolutive analysis strategies, as successfully carried out in  
 1168 (D'Altri et al. 2020b), to perform pushover analysis.  
 1169 Passing to other refined approaches (see Figure 26), like as FE models, masonry walls are often  
 1170 modeled with shell elements to reduce the computational cost (*e.g.*, Petracca et al. 2017b).  
 1171 This seems to be a reasonable simplification when the wall is not extremely thick. However, most  
 1172 FEM codes adopt a plane-stress formulation for the nonlinear material in each layer (through-the-  
 1173 thickness integration point) of the shell. For thick shells (shear-deformable), the out-of-plane shear  
 1174 is modeled elastically (and un-coupled from the in-plane and bending response). The transverse  
 1175 shear strain can be considered negligible up to the onset of strain location and only if the model is  
 1176 homogeneous. However, in micro-models, even if the overall wall can be considered as thin, the  
 1177 mortar joints are not. The overall crack in the out-of-plane failure can be seen as a rotational hinge  
 1178 at the global level, but locally (in a micro-model), there is a high concentration of transverse shear  
 1179 deformation in the joints. Therefore, it is mandatory to use a full 3D constitutive model (except  
 1180 when the normal stress is equal to zero) in each layer of the shell.

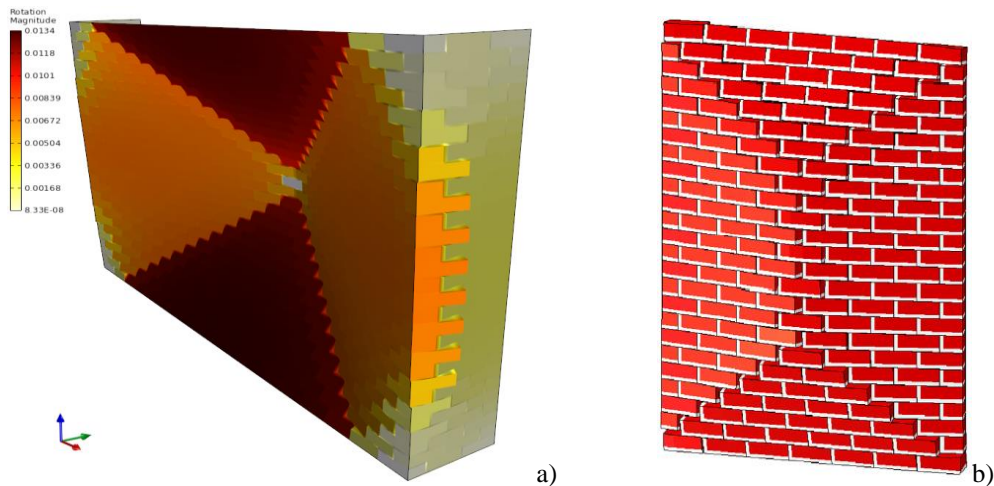


1181 Figure 24. Masonry panel without openings in two way bending: a) initial rough mesh and b) final mesh  
 1182 obtained with a Genetic Algorithm optimization approach. c) Failure mechanism with overturning  
 1183 obtained at the optimization procedure. d) Comparison among adaptive limit analysis (red horizontal  
 1184 line), standard refined limit analysis (black horizontal line), and a variety of non-linear force-  
 1185 displacement approaches available in the literature  
 1186

1187



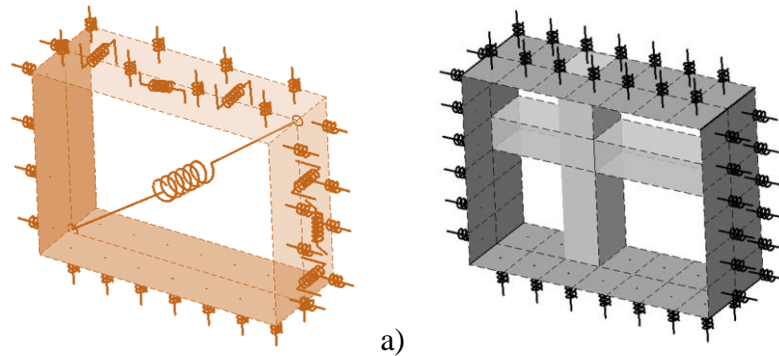
1188 Figure 25. Examples of optimized failure mechanisms found with an adaptive kinematic limit analysis: a)  
 1189 a medium-scale masonry church in Italy; b) a masonry dome subjected to horizontal load up to failure; c)  
 1190 a masonry aggregate with good interlocking between perpendicular walls  
 1191



1192  
 1193 Figure 26 –Numerical modeling of the out-of-plane failure of masonry walls: (a) using layered shell  
 1194 elements with a full 3D constitutive model (Petracca et al. 2017b), (b) using a 3D detailed micro-model  
 1195 (D’Altri et al. 2018).  
 1196

1197 In the discrete macro-element approach (DMEM), the 2D macro-element allows for the simulation  
 1198 of a masonry wall in its own plane but ignores the out-of-plane response (Caliò et al. 2012). To  
 1199 overcome this significant restriction, a third dimension and the relevant needed additional degrees

1200 of freedom have been introduced in a 3D macro-element (Panto et al. 2016, Pantò et al. 2017;  
1201 Chácará et al. 2018). Figure 27 reports the 3D macro-element obtained as the extension to the  
1202 space of the plane element first introduced in (Caliò et al. 2005).



1203 Figure 27 - 3D macro-element. (a) simplified mechanical scheme; (b) a typical fiber discretization of the  
1204 element.  
1205  
1206

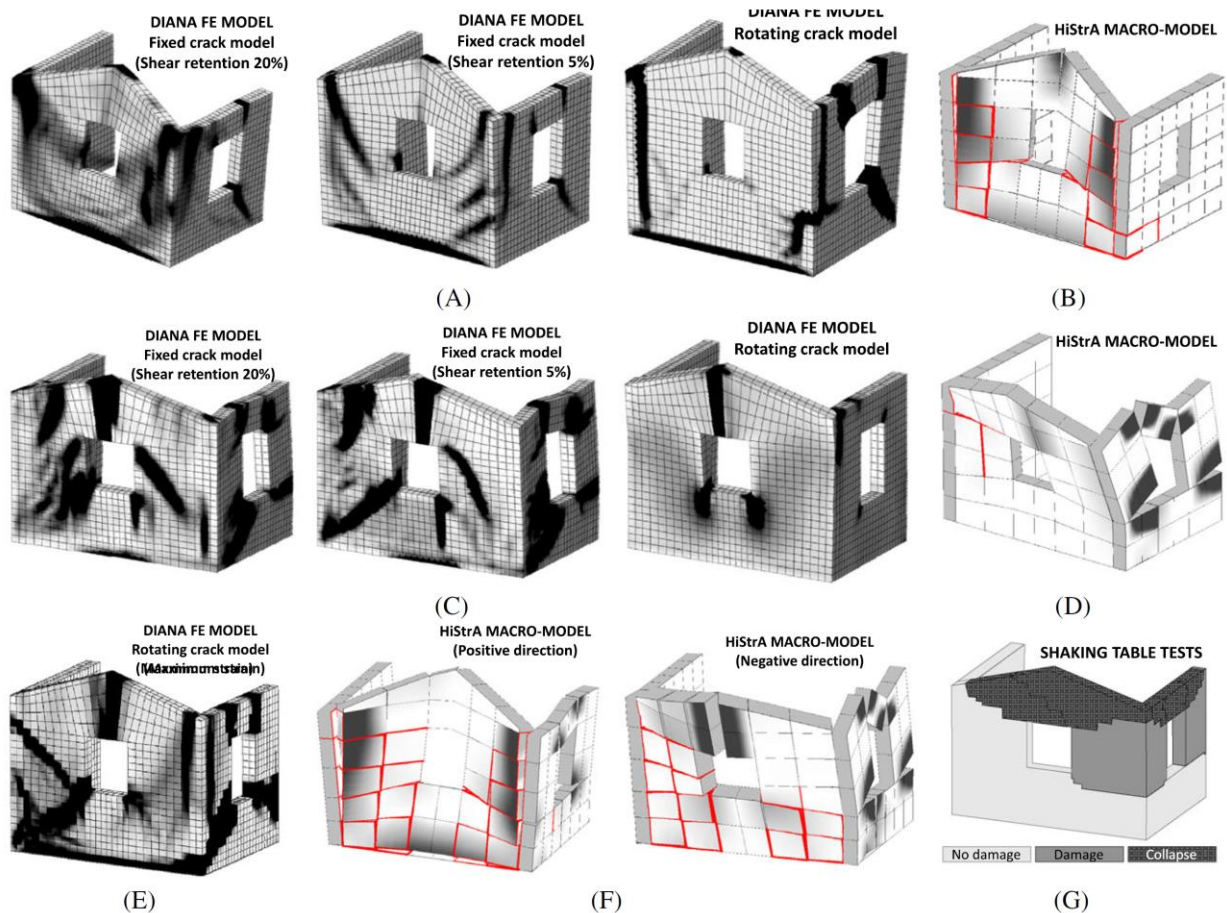
1207 The kinematics of the spatial macro-element is governed by just seven degrees-of-freedom and  
1208 can describe the in- and out-of-plane rigid body motions of the quadrilateral as well as the in-plane  
1209 shear deformability. The interaction of the spatial macro-element with the adjacent elements or  
1210 the external supports is ruled by 3D-interfaces. Each 3D-interface possesses  $m$  rows of  $n$   
1211 orthogonal (*i.e.*, perpendicular to the planes of the interface) nonlinear links. Consequently, each  
1212 interface is discretized in  $m \times n$  sub-areas (Figure 27b), similarly to what is done in classical fibre  
1213 models. The 3D interfaces are endowed with additional shear-sliding springs, required to control  
1214 the in-plane and out-of-plane sliding mechanisms and the torsion around the axis perpendicular to  
1215 the plane of the interface. The number of Nlinks adopted in the 3D-interfaces is selected according  
1216 to the desired level of accuracy of the nonlinear response. A detailed description of the spatial  
1217 macro-element's mechanical calibration and its numerical and experimental validation is reported  
1218 in (Pantò et al. 2017). This model has been also applied for the simulation of infilled frame  
1219 structures accounting for the in- and the out-of-plane behaviour of infills (Pantò et al. 2018).

1220 Figure 28 reports a recent numerical and experimental validation of the DMEM with reference to  
1221 a shaking table test carried out on a U-shaped prototype and discussed in Mendes et al. (2017). In  
1222 the figure, the gray color map represents the activation of plastic deformations in the nonlinear  
1223 links orthogonal to the interfaces and the red lines indicate the sliding motions along the in-plane  
1224 and out-of-plane directions.

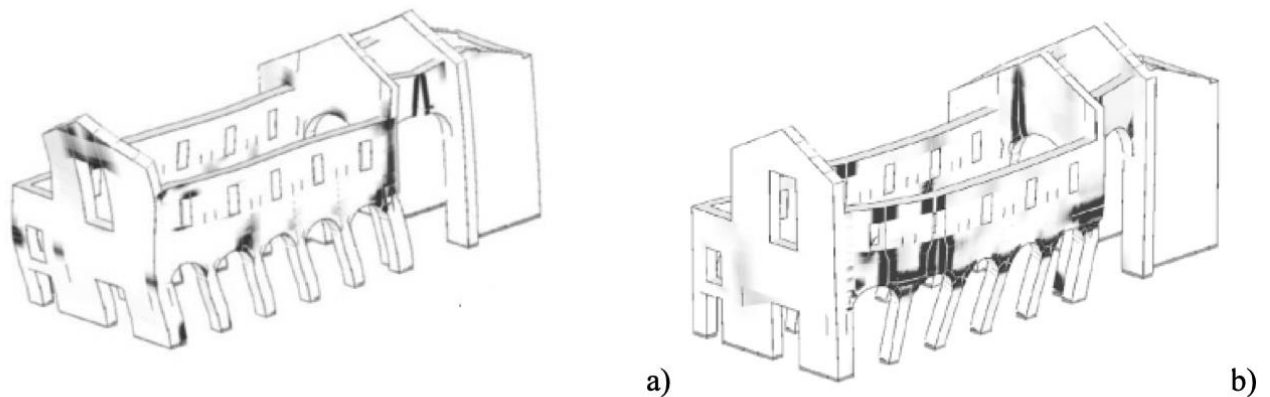
1225 Although the DMEM possesses a spatial element able to account for the in-plane and the out-of-  
1226 plane behaviour of masonry walls, its use in engineering practice still presents some limitations  
1227 due to the following issues. Unlike the in-plane behaviour, the simulation of the out-of-plane  
1228 mechanisms requires the use of a mesh of macro-elements that increases the computational burden  
1229 for large models. The a priori definition of the interfaces does not allow for the accurate  
1230 reproduction of the crack pattern developed during the formation of an out-of-plane failure

1231 mechanism. Aiming at improving the kinematics of the spatial element in (Minga et al. 2020), a  
 1232 further degree of freedom for simulating the out-of-plane diagonal cracking modes has been  
 1233 introduced to represent, in a phenomenological approach, the respective deformation modes of the  
 1234 inner block.

1235 Figure 29 reports a simulation of the nonlinear behaviour of a masonry church, via push-over  
 1236 analyses obtained with the DMEM.



1237  
 1238 Figure 28 - Compilation of collapse mechanisms of brick masonry prototype: positive (pushing) direction  
 1239 of (A) finite element (FE) models and (B) DMEM macromodel; negative (pulling) direction of (C) FE  
 1240 models and (D) DMEM macromodel; and dynamic response analysis of (E) FE, (F) DMEM  
 1241 macromodels, and (G) experimental campaign (adapted from Chácara et al. 2018).



1242  
 1243 Figure 29 –Failure mechanism and damage distribution obtained with the DMEM for longitudinal (a) and  
 1244 transversal (b) directions of a basilica plan church subjected to mass proportional push-over analyses  
 1245 (adapted from Pantò et al. 2016).

## 1246 **2.6 Critical issues in diaphragm modeling**

1247 Diaphragm stiffness and strength, as well as its connections to the masonry walls, have a crucial  
1248 role in the seismic behavior of masonry structures.

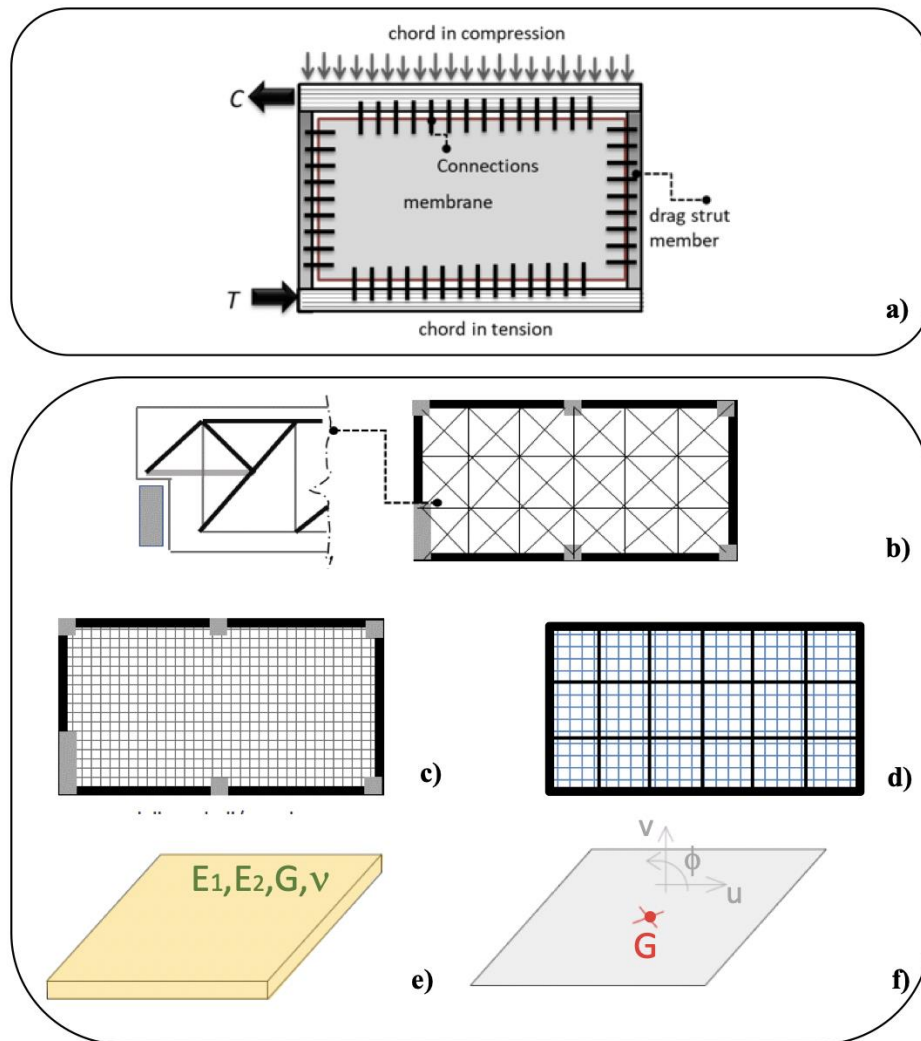
1249 As introduced in Section 1, statistical studies (Del Gaudio et al. 2019, Rosti et al. 2020), based on  
1250 a large amount of data on the seismic damage caused by the 2009 L’Aquila earthquake, pointed  
1251 out the importance of the in-plane deformability of floor diaphragms in the seismic response of  
1252 URM buildings. These research efforts have highlighted the greater seismic vulnerability of  
1253 buildings (worsened by out-of-plane actions in the absence of retaining steel tie-rods) with vaults  
1254 and, in general, deformable floors. On the contrary, it is well established that the in-plane stiffness  
1255 of floor diaphragms positively influences the global dynamic behavior of the structure, ensuring  
1256 the lateral load redistribution, restraining out-of-plane mechanisms and promoting a box-like  
1257 behavior. Diaphragms transfer horizontal loads to the vertical elements depending on their in-  
1258 plane stiffness. The latter should be considered in the model because it affects the distribution of  
1259 horizontal forces on the vertical elements. Numerical studies, based on nonlinear static and  
1260 dynamic analyses, have also proved that the seismic assessment is considerably sensitive to the  
1261 variation of the diaphragm stiffness (Nakamura et al. 2017), with further potential repercussions  
1262 on the definition of the damage limit states on the pushover curve (Cattari et al. 2015b, Marino et  
1263 al. 2019). The assumption made on the in-plane stiffness assigned to the floor diaphragms can  
1264 significantly influence the safety assessment evaluations, also according to the recommendations  
1265 of Codes. Extremely flexible diaphragms are not able to transfer in-plane shear forces, so that  
1266 lateral forces acting on each vertical element (wall) mainly depend on the corresponding tributary  
1267 area. Therefore, the element capacity can be assessed independently and the building could be  
1268 modeled as a set of independent subsystems.

1269 Figure 30a illustrates the main components of the diaphragms influencing their response:  
1270 membrane (or horizontal trusses), which carry the in-plane shear; drag strut members that transfer  
1271 the load to the masonry walls; chords which resist the compression and tensile forces that develop  
1272 in the diaphragm; and connections between the membrane, the chords, and masonry walls (a  
1273 review on the typologies that may characterize masonry buildings is presented in Solarino et al.  
1274 2019).

1275 Various modeling strategies can be adopted, and the elements should reproduce both the  
1276 diaphragm capacity and stiffness. Diaphragm stiffness and strength can be modeled with 1D  
1277 elements, beams, and/or trusses (Figure 30b), or 2D elements, shell or membrane (Figure 30c)  
1278 combined, if necessary, with beam elements (Figure 30d). If the diaphragm is very stiff, it is  
1279 reasonable to adopt kinematic constraints to reproduce its effect, thus reducing the number of  
1280 degrees of freedom of the model (Figure 30f). Even when simplified, a quite effective strategy



1281 often adopted in the case of EF models is to simulate the diaphragm in terms of an equivalent  
1282 orthotropic membrane (Figure 30e).



1283  
1284 Figure 30 – Diaphragm components a) and possible modeling strategies: b) 1D element model (with  
1285 springs and struts); c) 2D element model (with shell/membrane elements); d) with combined 2D element  
1286 models and beams; e) through equivalent orthotropic membranes; f) through a simple kinematic  
1287 constraint (rigid floor).  
1288

1289 According to this strategy, the crucial point becomes the assignment of equivalent elastic moduli  
1290 that are as reliable as possible. To this end, the support of structural diagnostic investigations  
1291 becomes very useful. However, even if several in situ tests are available to investigate the material  
1292 properties (as for example discussed in Krzan et al. (2015)), fewer experimental techniques are  
1293 available to evaluate the stiffness of floor diaphragms and the attention to this issue is quite recent.  
1294 For further details, interested readers may refer to: Giongo et al. (2015), Rizzi et al. (2020) and  
1295 NZSEE 2017, more specifically for timber floors; Rossi et al. (2016, 2017) and Cattari et al.  
1296 (2008b), more specifically for vaults; Sivori et al. (2021), in general for the use of ambient  
1297 vibration measurements to address the hypothesis on diaphragm stiffness.

1298 The model of the diaphragm can be linear or nonlinear. When a linear behavior is assumed, an  
1299 effective reduced stiffness should be considered depending on the expected damage of the  
1300 diaphragm (*i.e.*, cracking or slip among the elements). Moreover, in a linear hypothesis, the shear  
1301 capacity of the diaphragm must be checked after the analysis to ensure that it is higher than the  
1302 corresponding demand. The nonlinear behavior can be modeled with zero-length or fiber elements  
1303 in the case of 1D models and with continuum nonlinear models for 2D models. The models should  
1304 be conceived to correctly predict the in-plane stiffness and shear capacity of the membrane as well  
1305 as the chord and drag strut member capacities.

1306 The connections can be modeled explicitly with 1D elements or springs or, if not modeled, their  
1307 capacity should be checked against the in-plane shear demand.

1308 Finally, it is worth noting that, while in theory, the problem appears simple to solve, it is actually  
1309 unrealistic to assume it is always possible to reliably define the stiffness and the resistance capacity  
1310 of a real floor of an existing building. For this reason, in professional applications, it is sometimes  
1311 advisable to consider both the limit conditions of a completely flexible floor and, if the case may  
1312 be, of a completely rigid floor.

1313

### 1314 **3 Challenging issues in the use of URM nonlinear modeling for the seismic** 1315 **assessment based on nonlinear static analyses**

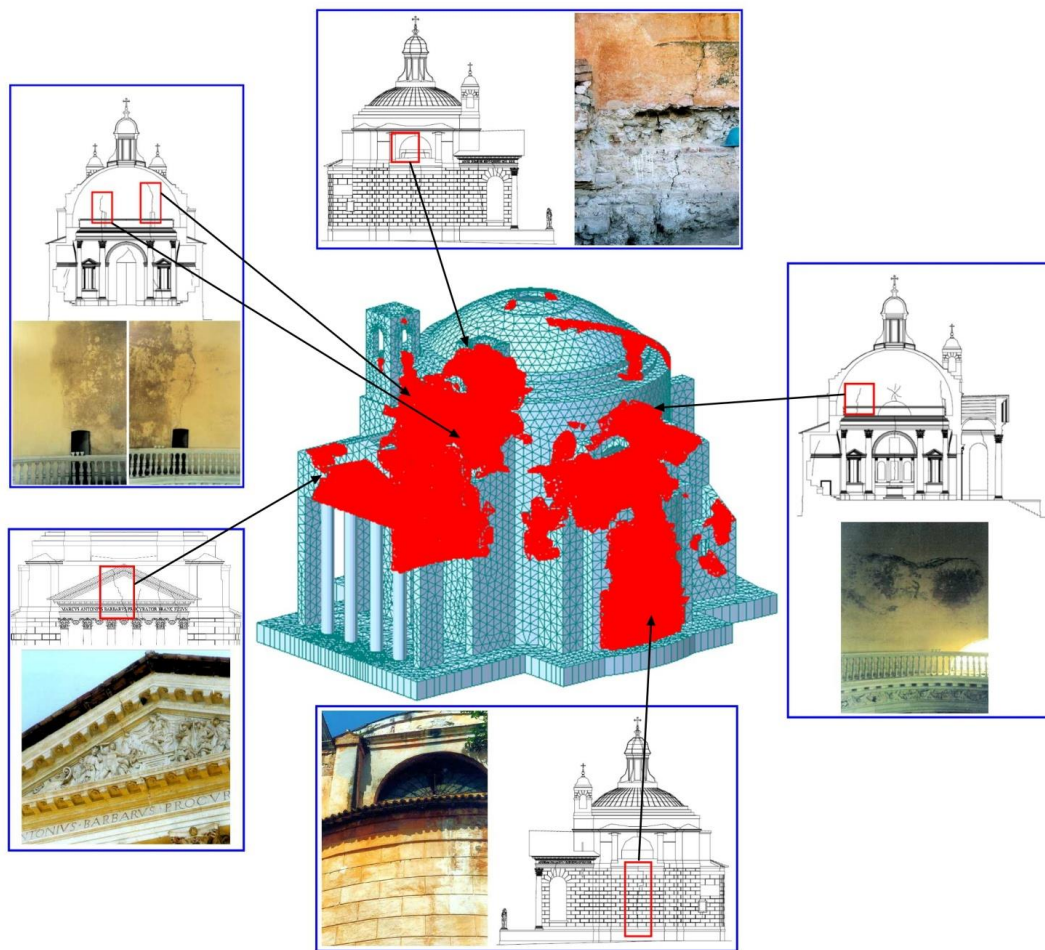
#### 1316 **3.1 Convergence issues in highly nonlinear analyses and algorithmic aspects**

1317 It is well known that the numerical modeling of brittle materials, such as masonry, is often very  
1318 challenging. The softening nature of masonry leads to highly nonlinear responses with many  
1319 instabilities, making the rigorous numerical simulation of such responses very difficult, mostly  
1320 due to convergence problems.

1321 It is mandatory to use methods, such as Displacement-Control or Arc-Length, to overcome the  
1322 local and global instabilities that can be found during the analysis in order to trace the correct  
1323 equilibrium path.

1324 At the same time, convergence problems should be mitigated in order to trace the equilibrium path  
1325 beyond the peak resistance point. Constitutive models are often equipped with algorithms that aim  
1326 to reduce the strong nonlinearity of the problem to achieve a better convergence. These kinds of  
1327 algorithms, such as viscoplastic regularization (see the Concrete Damage Plasticity CDP model in  
1328 ABAQUS) or IMPL-EX Mixed Implicit-Explicit integration (Oliver et al. 2008), are however  
1329 artificial, and their algorithmic parameters are non-physical, thus hard to calibrate. If not calibrated  
1330 properly, they can lead to highly different results. It is therefore necessary to adopt procedures  
1331 where the error is maintained under a reasonable threshold.

1332 Another possible approach, particularly suitable for overcoming convergence difficulties, is the  
 1333 so-called “Sequentially linear analysis” (*e.g.*, Rots 2001), which adopts a simplified saw-tooth  
 1334 stress-strain softening law for masonry. In detail, a series of linear analyses is performed where  
 1335 the elastic modulus of the elements reaching the tensile strength is reduced following the saw-  
 1336 tooth stress-strain law adopted. It is worth noting that the sequentially linear analysis, due to its  
 1337 intrinsic simplicity, can be used in most commercial FE programs even without specific  
 1338 customization capabilities, making it particularly interesting for practitioners. As an example,  
 1339 Figure 31 shows the results obtained with a sequentially linear analysis applied to the Palladio’s  
 1340 Tempietto Barbaro, where the elements reaching threshold values in tension are marked in red,  
 1341 indicating potential cracking areas (see Berto et al. 2017).



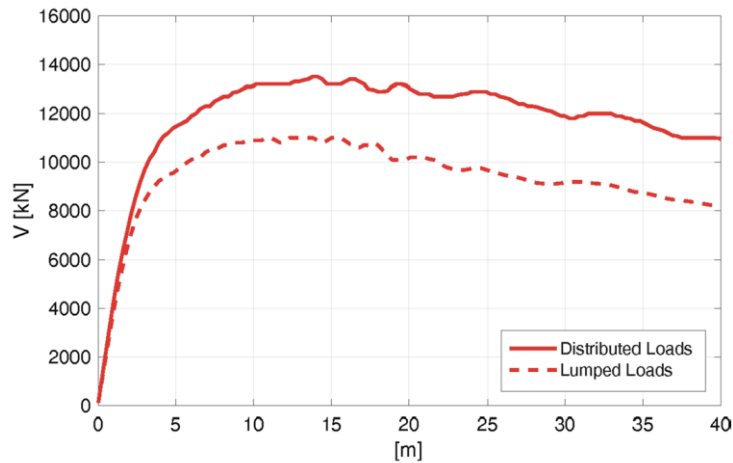
1342 Figure 31 – Palladio’s Tempietto Barbaro: results of sequentially linear analysis compared with crack  
 1343 pattern (Berto et al. 2017)  
 1344  
 1345

### 1346 3.2 Horizontal load application in pushover analysis

1347 The application of the horizontal load pattern in pushover analyses represents a conventional  
 1348 aspect that may induce additional sources of differences in results passing from EF to refined  
 1349 models.

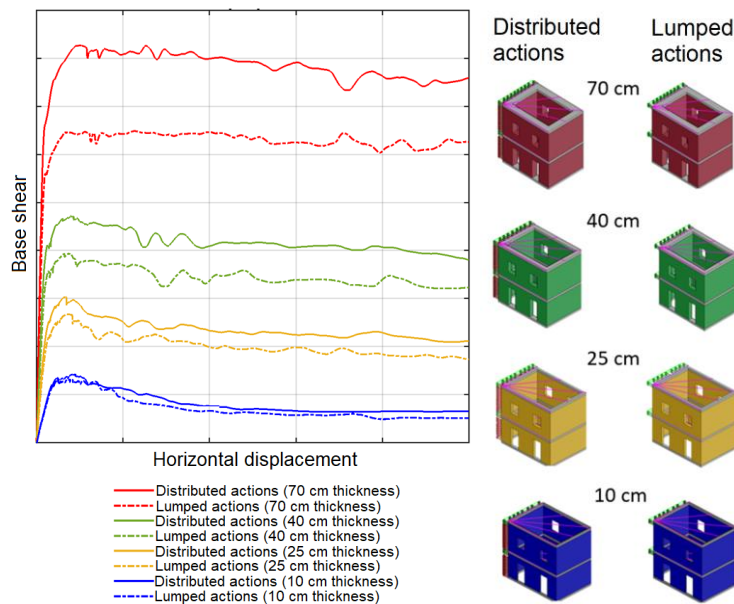
1350 <https://www.saxion.edu/programmes/exchange-programme/industrial-sustainable-building>A first  
1351 issue, indeed common to both modeling approaches, concerns the choice of the load pattern  
1352 simulating the seismic action through static incremental horizontal forces. Possible options for  
1353 the distribution to apply are the following (see Aydinoglu and Onem 2010 for an overall  
1354 overview): (1) proportional to masses (obtained from a uniform displacement shape); (2) obtained  
1355 from a triangular displacement shape (pseudo-triangular) (3) proportional to the fundamental  
1356 modal shape (modal); (4) given by a proper combination of different modes (e.g. SRSS-based or  
1357 as proposed in Reyes and Chopra 2013 or Azizi-Bondarabi et al. 2019, among others); (5) load  
1358 pattern adapted to the current displacement shape (adaptive, e.g. as originally proposed in  
1359 Antoniou and Pinho 2004). The first three distributions correspond to the ones most frequently  
1360 adopted in engineering practice, but depending on the geometric and mass features of the building  
1361 to analyze, the typologies of diaphragms (if rigid or not), the in plan and in elevation irregularities,  
1362 etc., the most reliable choice deserves careful attention. A critical discussion on the use of these  
1363 various possibilities in the case of URM buildings is reported in Lagomarsino and Cattari (2015b).  
1364 Once a certain load pattern is assumed (e.g., proportional to masses), the horizontal load can be  
1365 considered lumped at the floor level (lumped actions) or distributed along the entire height of the  
1366 building (distributed actions). Although this last scheme better represents the inertial forces that  
1367 can actually arise in the structure, the simplified use of lumped actions at the floor level is generally  
1368 assumed in EF models.

1369 The distribution of horizontal loads could have a non-negligible impact on the pushover curve and  
1370 on the peak base shear. Lumped actions are expected to produce a lower peak base shear than  
1371 distributed actions, as shown in Figure 32, where the pushover curves referred to a benchmark  
1372 structure, namely the Benchmark Structure BS5 inspired by “P. Capuzi” school in Visso (already  
1373 discussed in Figure 22, see Castellazzi et al. 2021 for further details), are comparatively  
1374 represented. The curves have been obtained by using a continuum constitutive law model and a  
1375 general increase of up to 22% of the maximum load when moving from lumped actions to  
1376 distributed actions at floor level has been registered. To further confirm this potential effect in  
1377 Figure 33 the results of the 3D two-story single unit building (Benchmark Structure 4 in the  
1378 *Nonlinear URM modeling – Benchmark* project), discussed in Cannizzaro et al. 2021, are  
1379 presented. As it could be expected, the larger the wall thickness, the bigger the differences are in  
1380 terms of pushover curves between lumped and distributed actions.



1381

1382 Figure 32 –Influence of the type of horizontal load application (distributed actions versus lumped actions)  
 1383 on the BS5 analyzed in Nonlinear URM modeling – *Benchmark project* reproduced by using a continuum  
 1384 model (from Castellazzi et al. 2021)  
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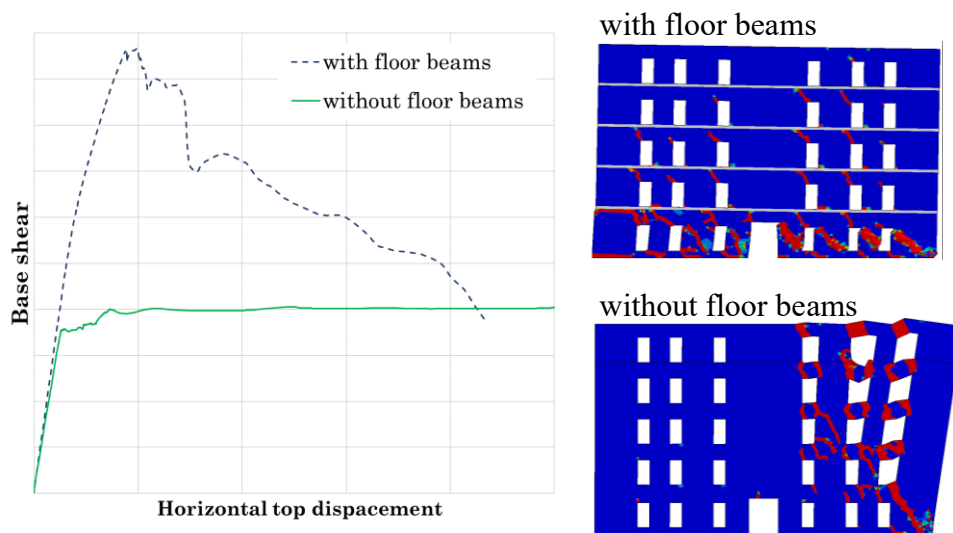
1387 Figure 33 –Influence of the type of horizontal load application (distributed actions versus lumped actions)  
 1388 depending on the thickness of the walls for the BS4 analyzed in *Nonlinear URM modeling* – *Benchmark*  
 1389 *project* (adapted from Canizzaro et al. 2021)  
 1390

1391 **3.3 Definition of displacement thresholds in the pushover curve associated to ultimate limit**  
 1392 **states**

1393 Seismic verification, according to the performance-based approach, requires to associate given  
 1394 limit states (LS) to specific performance conditions characterizing the structure under  
 1395 consideration. Usually, LSs do not refer only to the structural damage but also to other  
 1396 performances (such as reusability, immediate occupancy, operational functions, economic issues,  
 1397 etc.). The quantification of the performance requirements usually needs the introduction of  
 1398 conventional criteria to correlate them to proper engineering demand parameters that can be  
 1399 estimated and monitored by the numerical models. To this aim, different criteria are proposed in

1400 the literature and codes. In some cases, the definition of LSs is treated by checking the attainment  
1401 of corresponding damage levels in each single element (ASCE 41-17 2017) or by considering  
1402 inter-storey drift thresholds and/or heuristic criteria on the stiffness and strength degradation of  
1403 the pushover curve (EC8-3 2005, MIT 2019). More specifically, and referring to ultimate limit  
1404 states, in (MIT 2019), various criteria are integrated with those based on the strength degradation,  
1405 and for example, consist of defining the attainment of the ultimate drift in all the masonry piers of  
1406 one level of the building as the ultimate condition or, for very deformable floors, of one level of  
1407 one masonry wall. Further limitations are put in terms of maximum ductility demand (expressed  
1408 equivalently through a maximum force reduction ratio) differentiated for Life Safety and for Near  
1409 Collapse limit state. The logic and benefit of such a multi-criteria approach are also discussed in  
1410 (Lagomarsino and Cattari 2015a). An in-depth review of these criteria is out of the scope of this  
1411 paper, and those interested may refer to (Marino et al. 2019). Whatever the rule adopted, the  
1412 definition of an ultimate condition of the structure (in terms of ultimate displacement) is  
1413 fundamental, also because in most nonlinear static procedures, like for instance the N2 Method  
1414 proposed by (Fajfar 2000) and recommended in both (EC8-3 2005) and (MIT 2019), it also affects  
1415 the derivation of the bilinear equivalent curve that is at the basis of the assessment procedure.  
1416 Ideally, the adoption of coherent criteria and rules should be required for all the various modeling  
1417 strategies discussed in the paper.

1418 Despite that, heuristic criteria based exclusively on the strength degradation rule (*i.e.*, when the  
1419 base shear decreases at 80% of the maximum base shear) are not always easy to apply in the case  
1420 of refined models. In fact, while in EF models the collapse condition at element scale is usually  
1421 automatically implemented in terms of attainment of given drift thresholds and, consequently, the  
1422 softening phase on the overall pushover curve is detected thanks to the progressive failure of piers,  
1423 in more refined models such a pronounced softening cannot be reached systematically. This may  
1424 be the consequence of either convergence issues in a strong nonlinear phase or damaging and  
1425 failure modes of the structure that are not always associated with a significant softening behavior  
1426 (*e.g.*, when dominated by the flexural response). Moreover, as already introduced in Section 2.2,  
1427 it is useful to recall that the above-mentioned checks at element scale are not directly managed by  
1428 refined models and require some post-processing of the results. As an example, Figure 34 shows  
1429 two pushover curves obtained for the masonry wall of Figure 5 by (Occhipinti et al. 2021). In such  
1430 analyses, two different configurations have been considered, namely with or without floor  
1431 reinforced concrete beams. The case with floor beams shows a significant softening after the  
1432 diagonal shear failure of the masonry panel at the bottom level. Conversely, the case without floor  
1433 beams shows the formation of an overturning mechanism of the right part, which is associated  
1434 with a long plateau in the pushover curve (*i.e.*, no softening behaviour is observed).



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Figure 34 – Example of a masonry structure that shows softening behavior when floor beams are present, while no softening is observed without floor beams (adapted from Occhipinti et al. 2021)

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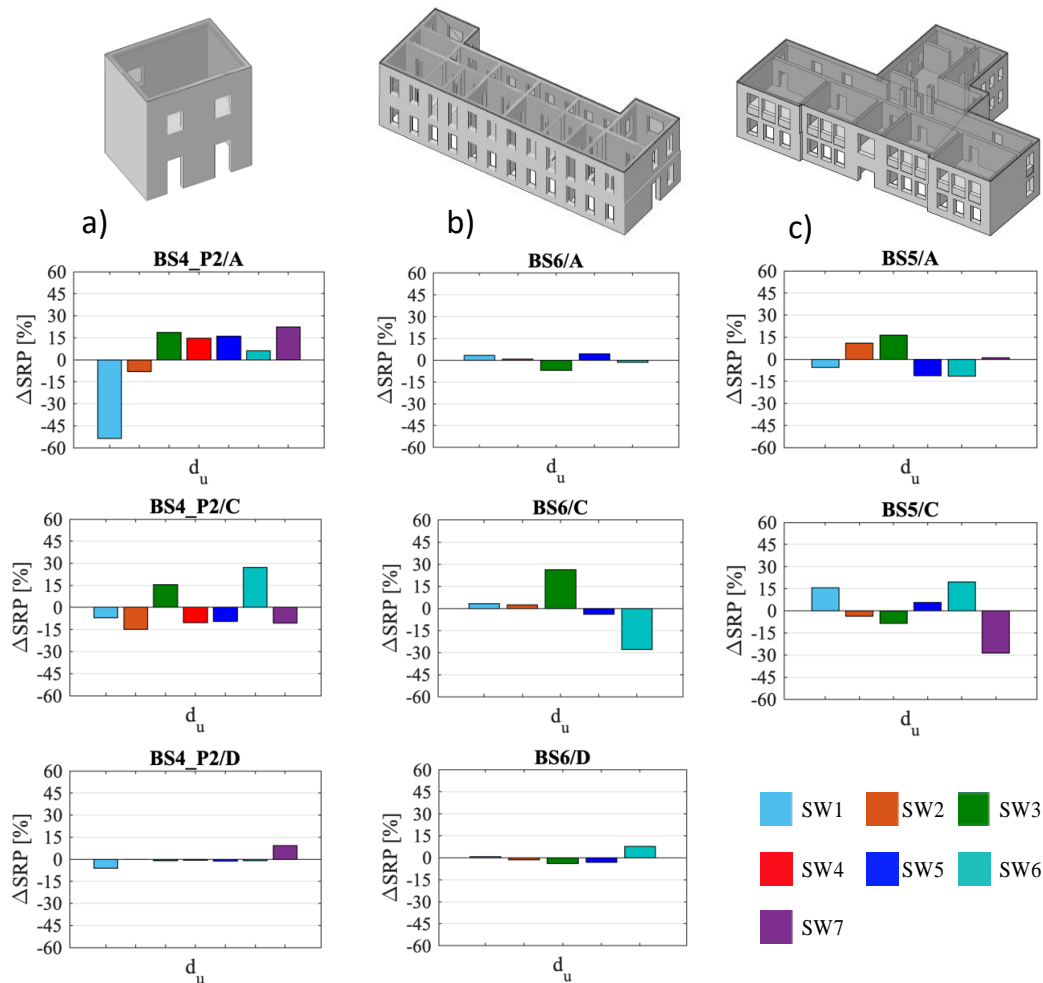
1461

When no significant softening is observed in the pushover curve, the alternative above-mentioned solutions, that refer to performing checks on single elements or sets of elements (e.g., those at the same level) are very effective and useful, see also for instance Figure 32 for model B.

However, as already briefly mentioned in Section 2.2, such an approach implies various steps to be done *ex-post* like identifying the most representative structural elements, interpreting their failure modes, and computing derived parameters on selected sections through integration operations (e.g., computing the drift). Some examples are illustrated in (Castellazzi et al. 2021).

Various uncertainties characterize the conventional ultimate drift and influence the results achievable with EF models. As also discussed in (Cattari and Magenes 2021), the main issue is whether rigid body motion is detracted from the angular deformation demand on the pier. The result is that different variables may be associated with the same general concept of “drift,” like the angular deformation, the simple ratio between the difference of horizontal displacements at the end sections and the panel height, the chord rotation, and its equivalence with the plastic component of the rotation. This arbitrariness follows the fact that most codes recommend thresholds for the “drift” of a wall element to check the attainment of the collapse without clarifying the criteria adopted to compute it, and still, there are no unanimous criteria shared in the scientific community. Since these criteria are then autonomously defined by the software packages without allowing users to change them, this may result in a scatter of the ultimate displacement capacity on pushover curves obtained by different software. As shown in Figure 35, this has been highlighted in all the benchmark structures examined in the *URM nonlinear modeling*-Benchmark project, somehow independently from their complexity as documented in (Manzini et al. 2021) for BS4, (Ottonelli et al. 2021) for BS5, and (Degli Abbati et al. 2021) for BS6. More specifically, the variation illustrated in the figure ( $\Delta SRP$ ) refers to the difference between the value computed for

1462 a given software package and a reference value equal to the average estimated from the numerical  
 1463 predictions made by the entire SW set considered. The results demonstrate that this variation  
 1464 systematically decreases only for specific idealized conditions (*e.g.*, in the shear-type idealization,  
 1465 named /D in Figure 35), which makes the rotation component at the end sections of elements  
 1466 irrelevant. Conversely, in structural configurations associated with weak spandrel behavior  
 1467 (named /A in Figure 35) or even with the systematic presence of r.c. beams at floor level (named  
 1468 /C in Figure 35), the dispersion on the ultimate displacement capacity is substantially comparable.



1469 Figure 35 – Variation of the ultimate displacement estimated by various software packages used in the  
 1470 *URM nonlinear modeling*-Benchmark project across various benchmark structures (adapted from Manzini  
 1471 et al. 2021 for BS4, from Ottonelli et al. 2021 for BS5, and Degli Abbati et al. 2021 for BS6)  
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## 1474 4 Conclusions

1475 The overview outlined in the paper on the nonlinear seismic modeling of URM structures has  
 1476 revealed the complexity of the topic and has shown the significant scientific advancements  
 1477 obtained in the last years. Considering the first applications of non-linear seismic analysis on  
 1478 masonry buildings dating back to the 1970s (Tomazevic 1978), today the available numerical tools  
 1479 really have the potential to simulate much more accurately the actual seismic response. This allows  
 1480 to overcome the numerous simplifying hypotheses of models used in the past, that in many cases



1481 did not reflect the actual behaviour of existing buildings. Furthermore, significant progress has  
1482 been made in transferring many of the research advances into commercial software packages,  
1483 making them available to professionals. However, given the variety of available choices, the  
1484 analysts, when using the software packages, still need a solid expertise regarding the seismic  
1485 response of the buildings, and need to be aware of what the available software is able to simulate  
1486 and how. To provide a contribution in this direction, in this paper the advantages, limitations and  
1487 open problems related to the use of different modeling techniques of URM buildings in  
1488 engineering practice have been discussed, with particular regard to the nonlinear static seismic  
1489 analysis. Possible criteria and methods for the critical comparison of different models/software  
1490 packages have been suggested, in particular when simplified equivalent frame model results are  
1491 compared with more refined models, with reference also to code-prescribed performance limit  
1492 states.

1493 Despite the current significant level of progress of the available nonlinear modeling techniques for  
1494 masonry structures, the review presented in the paper has pointed out several open issues in which  
1495 scientific advancements are needed.

1496 Regarding equivalent frame models, the areas of future development are primarily the following:

- 1497 - The integration of the in-plane response with the out-of-plane one (mainly referring to the  
1498 activation of local mechanisms) into the same equivalent frame 3D representation of a  
1499 building.
- 1500 - The availability of more standardized and sound rules to account for the epistemic  
1501 uncertainties involved in the modeling process. Among the others, the two most important  
1502 issues are the following: (1) the idealization of URM walls into equivalent frames,  
1503 especially in the case of irregular opening layouts (since results may be very sensitive to  
1504 the alternative options currently available in the literature); (2) the development of  
1505 sufficiently reliable criteria to account for the wall-to-wall connections and composite  
1506 sections, accounting, if possible, for different degrees of effectiveness. This would be very  
1507 useful, particularly for professionals, to reduce the significant scatter of results generated  
1508 by such epistemic uncertainties and, moreover, to avoid inappropriate choices, potentially  
1509 leading to strong underestimation/overestimation of the actual capacity of existing  
1510 buildings.

1511 As regards refined models, the paper has highlighted how they can be very effective also in guiding  
1512 the calibration of simplified models. In principle, they have the advantage of allowing the  
1513 modeling of any type of geometry without the approximations implied in simplified models such  
1514 EF. In theory, virtually every detail of the structure, including connections and interfaces, could  
1515 be modeled, nonlinearly when needed, with a suitably calibrated constitutive law. However, in

1516 engineering practice, some recommendations and guidelines are necessary and needed since their  
1517 use requires a very high level of expertise and a proper calibration of the parameters of the models.  
1518 Nowadays, their use in engineering practice is thinkable only when the resources needed (human  
1519 and computational) to produce reliable results justify their cost.

1520 Thus, as regards refined models, the outlined areas of future development are primarily:

- 1521 - To increase their robustness in allowing practitioners to use very sophisticated  
1522 approaches reducing as much as possible the risk of incurring in large errors due to the  
1523 lack of knowledge about how all the parameters should be calibrated, allowing thus the  
1524 professional to focus on the knowledge of the physical object that is studied (the  
1525 building), which is always and anyway an essential step of the assessment procedure.
- 1526 - To improve their reliability, addressing efforts towards the following tasks: (1) in  
1527 calibrating the constitutive model parameters to match code-defined macromechanical  
1528 properties, an issue which can affect tremendously the results (and the convergence of  
1529 the solution); (2) in properly describing the orthotropic nature of masonry; (3) in relating  
1530 damage patterns to code-defined performance limits (these last difficulties in good part  
1531 are also related to the oversimplification implied in code-prescribed limits).

1532 Regardless of the modeling approach adopted, it has been noted how criteria to guide the nonlinear  
1533 modeling of floor and roof diaphragms are very scarce both in the literature and in  
1534 design/assessment guidelines. Moreover, the potential repercussions of improvements in the  
1535 modeling of diaphragms on performance assessments based on nonlinear static or dynamic  
1536 analyses are still unclear (*e.g.*, criteria for the definition of limit states).

1537 Some issues more specifically related to nonlinear static modeling of URM structures have also  
1538 been discussed in the paper, given the increasing use of this method for seismic assessment in  
1539 everyday engineering practice. Among these, the paper has highlighted the issues related to the  
1540 representation of the seismic action in distributed mass systems (the effect of lumping the seismic  
1541 forces vs. using distributed load patterns) and to the definition of displacement/deformation based  
1542 criteria and thresholds for a performance based assessment depending on the type of model that is  
1543 being used (simplified or refined).

1544 Finally, some issues have not been covered by the review presented in the paper, and could be the  
1545 subject of future reviews. Among these are some complex issues typical of the execution of  
1546 nonlinear time history analyses. Just to mention a few of them: the simulation of the cyclic  
1547 response, which would give rise to additional issues such as the computational efficiency; the  
1548 capability of reproducing energy dissipation associated to different failure modes; and, last but not  
1549 least, the proper interpretation of the large and complex amount of data obtained from the analyses.  
1550 Surely, EF models have a great potential if extended to cyclic modeling, taking advantage of the

1551 reduced computational effort needed, with respect to refined nonlinear models. Some examples  
1552 already exist in the literature, but they need to be further consolidated and developed before they  
1553 can be accessible to professionals in commercial software packages.

1554

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1578

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1582

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1584 The benchmark structures mentioned in the paper can be replicated by other interested researchers and  
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1589

1590 **Authors' contributions**

1591 SC: conceptualization, methodology, original basis of scientific results presented in the examples discussed  
1592 in the paper, writing-original draft; BC/IC/GC/SDM/GM/GM/AS: conceptualization, methodology,  
1593 original basis of scientific results presented in the examples discussed in the paper, writing-review.

1594

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1596

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