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The role of reinforced concrete roofs in the seismic performance of masonry buildings

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Abstract

The 2016 Central Italy earthquake caused many collapses of existing masonry buildings that had previously been retrofitted with reinforced concrete roofs. The aim of this paper is to explore the role of these roofs in the seismic behaviour of masonry buildings. Simple analytical models are presented to illustrate two typical out-of-plane collapse mechanisms: wall overturning and vertical flexure. The models are based on linear kinematic analysis, which allows fast modelling and calculation of a coefficient that can be used to assess the safety level of a structure. Nonlinear kinematic analyses were also performed. Both methods were applied to two case studies taken from areas struck by the earthquake. Results show that linear analysis represents an effective tool for preliminary verifications that can allow one to understand whether retrofit interventions are needed.

Keywords: masonry, reinforced concrete roof, collapse, retrofit, kinematic analysis, Central Italy earthquake

1. Introduction

After the 24th August 2016 Central Italy earthquake, most of the buildings of small towns nearby the epicenter were declared unsafe and several structures collapsed completely. Poor material quality and scant building techniques were certainly the main reason of collapses. However, inadequate retrofit interventions also contributed to the disruptive effect of the seismic event. For instance, the replacement of the old wooden roofs with reinforced concrete roofs seemed to facilitate some mechanisms that led to severe damages and collapses. This type of retrofitting was broadly adopted in the 80s and 90s since it was believed to be effective against seismic actions. In fact, it was the Italian code itself to recommend it [1]. Moreover, at that period there was a massive use of concrete that led to a gradual abandon of research and experimental tests on masonry [2]. The overall idea was to put robust structures such as RC roofs and floors connected to perimetric walls by means of RC ring beams to avoid independent movements of masonry macro-elements. After Tolmezzo earthquake in 1976, this and other retrofitting techniques became part of technical codes, until Umbria and Marche earthquake in 1997 [3, 4].

40 This event pointed out the disadvantages of heavy and stiff roofs and floors. In fact, if vertical
41 structures are not robust enough, they are indeed the primary cause of collapses. The significant
42 stiffness and load increment at the top have led to the collapse of the walls, which were made of
43 poor materials and not strengthened. Conversely, there were also many cases of masonry
44 structures retrofitted with reinforced concrete roofs that withstood the earthquake with no
45 significant damages (Figure 1).



46
47 Figure 1. Masonry buildings retrofitted with concrete roof not collapsed after the earthquake in
48 Pescara del Tronto (a-b) and small villages near Accumoli (c-d).

49
50 However, there is no guarantee that those buildings are safe. Therefore, in this paper a simple
51 verification procedure that is able to estimate the level of safety of masonry buildings with
52 reinforced concrete roofs is implemented. The adopted approach is based on the linear kinematic
53 analysis, which is also described by Italian codes [5, 6]. Despite the method is well known in its
54 theoretical formulation, it is rarely used and usually the effect of the roof and the connection
55 among structural elements are neglected. This research contributes to the current literature with
56 practical applications of the kinematic analysis introducing simplified analytical models that
57 take into account the effect of reinforced concrete roofs. An additional advantage of the
58 proposed models is that the number of input parameters has been reduced as much as possible so
59 that the analysis does not require any particular investigation or survey to be carried out. A
60 safety factor was also defined to assess the safety level of the building towards different collapse

61 mechanisms. The choice of a simplified procedure has been made in order to have a fast tool
62 which could be used even by non-professional users. The method would allow property owners
63 to understand if they are in danger. For instance, if the obtained safety factor is low or close to
64 the unsafe threshold, further investigations should be conducted. More detailed methods have
65 been studied by many authors to describe masonry buildings behaviour, but they need to be
66 calibrated and the input data are often not accessible [7-10]. Obviously, results will not be as
67 accurate, and a certain margin of error should be taken into account in final considerations.
68 Nonetheless, they can provide relevant preliminary information about the structure. In addition,
69 in the literature there is a number of studies about masonry where analytical models turned out
70 to be highly effective and close to the real behaviour [11, 12].

71 After defining the formulation, the method is applied to different models describing the
72 overturning and the vertical flexural behaviour. The models derive from those commonly used
73 to study the out-of-plane mechanisms [13-15] and the arch rocking [16-18]. To analyse the
74 influence of the connection between the roof and the floor to the walls, a ring beam is also
75 considered. The presence of a reinforced concrete (RC) ring beam is dangerous if it is not well
76 connected to masonry walls and if the latter is not strengthened. Furthermore, the spread of
77 reinforced concrete in the construction sector, led to wrong applications in the interventions of
78 existing buildings. Nowadays there are many solutions to realize effective structural
79 connections, such as reinforced masonry ring beams [19]. The use of innovative composite
80 materials has become a common practice in retrofit strategies. Several studies have been carried
81 out in this field which has allowed to investigate the behavior of strengthened beams [20, 21]
82 and strengthened masonry walls through out-of-plane tests [22].

83 Two case studies taken from two towns struck by the abovementioned earthquake were
84 analyzed, but the method can be extended to any building by choosing appropriate parameters.
85 Both examples were selected by considering typical houses in the area, built with local materials
86 and poor construction techniques and retrofitted with reinforced concrete roofs. The first one is
87 1-storey building while the second one has two storeys and thus also the action of the inter-
88 storey floor is considered in the model. For each model, the linear kinematic analysis is repeated
89 for different values of the input parameters, as they could be affected by uncertainty. In this way
90 it is possible to see the influence of a single parameter and what happens if it is over-estimated
91 or under-estimated. Finally, nonlinear analyses are performed in order to compare the results and
92 understand if the additional computational effort of a more refined method is worth it.

93

94 **2. The linear kinematic analysis**

95 In existing masonry buildings there are often collapses due to a loss of equilibrium of some
96 portions of bearing structures. In general, these types of mechanisms happen when seismic
97 forces act in the out-of-plane direction. The linear kinematic analysis can be used to study
98 these phenomena and for the verification process. It is based on the choice of the possible
99 mechanisms that are most likely to happen. These ones are assumed by evaluating the current
100 cracking state and analyses performed on similar buildings. In the literature there are plenty
101 of studies on historical buildings, such as churches, which are helpful to clarify how the
102 collapse process activates and evolve [23]. The ability to detect the most probable
103 mechanisms is crucial to prevent local or global collapses, since it is possible to run specific
104 analyses and consequently suggest specific interventions.

105 The linear kinematic approach schematizes the building in a discrete number of macro-
 106 elements which move according to their boundary conditions. For this reason, the
 107 assumptions are that the material has no tensile strength and infinite compressive strength. In
 108 each rigid block, vertical loads (including dead and external loads) and a system of horizontal
 109 forces are applied. Horizontal forces are proportional to the vertical loads through a
 110 coefficient called load multiplier (α). Incrementing the load multiplier, it is possible to
 111 evaluate the horizontal force that activates a specific mechanism. α_C is named the collapse
 112 load multiplier, and it is calculated with the principle of virtual works. Therefore, the total
 113 work of the external forces (L_e) has to be equal to the total work of the internal forces (L_i)
 114 which in this case is null as shown in Eq. (1):

$$115 \quad L_e = \alpha_C \left(\sum_{i=1}^n W_i \cdot \delta_{x,i} + \sum_{j=n+1}^{n+m} W_j \cdot \delta_{x,j} \right) - \sum_{i=1}^n W_i \cdot \delta_{y,i} - \sum_{h=1}^o F_h \cdot \delta_h = L_i = 0 \quad (1)$$

116 where: n is the number of the weight forces applied to all macro-elements; m is the number of
 117 weight forces that generate horizontal forces upon macro-elements; o is the number of
 118 external forces; W_i is the generic weight force; W_j is the generic weight force that generates
 119 horizontal forces upon macro-elements; $\delta_{x,i}$ is the virtual horizontal displacement of the point
 120 where the i -th weight force is applied; $\delta_{y,i}$ is the virtual vertical displacement of the point
 121 where the i -th weight force is applied; F_h is the generic external force; δ_h is the virtual
 122 displacement of the point where the generic external force F_h is applied. Eq. (1) often
 123 becomes an equilibrium equation between a stabilizing moment and an overturning moment,
 124 so it is not necessary to calculate the virtual displacement. The method is also used to
 125 determine the most probable collapse mechanism which is the one that requires less energy to
 126 be activated (i.e. the one with the lower load multiplier). However, the decay conditions of
 127 masonry should never be neglected since they are able to reveal if a specific mechanism has
 128 already been activated. Once α_C is calculated, it is possible to obtain the acceleration that
 129 generates the mechanism (Eq. (2)).

$$130 \quad a_0^* = \frac{\alpha_C \cdot \sum_{i=1}^{n+m} P_i}{M^* \cdot FC} \quad (2)$$

131 where FC is a coefficient that depends on the level of knowledge about the masonry
 132 structure. The level of knowledge is based on information like geometry, construction details,
 133 and material properties. Such data can be acquired in different ways, from generic research
 134 and visual inspection to extensive tests and measurements. Since the material strength is not
 135 considered in this research, only basic information about geometric characteristics, type of
 136 masonry panels and construction details was collected. According to the Italian code [6],
 137 three level of knowledge can be identified: limited, extended and exhaustive. Due to the
 138 limited available data, the level of knowledge is limited. In this case, the code reports the FC
 139 coefficient has to be assumed equal to 1,35 which reduces the acceleration that generates the
 140 mechanism. M^* is the participating mass, calculated considering the virtual displacements of
 141 the points where the loads are applied, as shown in Eq. (3) [6]:

142

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \cdot \delta_{x,i} \right)^2}{g \cdot \sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}^2} \quad (3)$$

143

The acceleration given by Eq. (2) has now to be compared with an allowable acceleration.

144

This one is given by Eq. (4), which is valid when the analysed blocks are in contact with the

145

ground, meaning that the mechanism involves ground floor walls:

146

$$a^* = \frac{a_g \cdot S}{q} \quad (4)$$

147

where: a_g is the peak ground acceleration at the site determined, as indicated by Italian codes,

148

for a return period of 475 years; q is the reduction factor which can be assumed equal to 2 for

149

regular masonry structures; S is given by the product of two coefficients: S_s that depends on

150

the soil category and represents the stratigraphic amplification, and S_t which takes into

151

account the effects of the topographical amplification and depends on the surface

152

configuration of the soil.

153

It is clear that the acceleration that activates the mechanism should be greater than the

154

allowable one. To quickly verify this, it is possible to introduce a safety factor which is the

155

ratio between the two accelerations (Eq. (5)):

156

$$SF = \frac{a_0^*}{a^*} \geq 1 \quad (5)$$

157

158 3. Analytical models

159

In some cases, there are mechanisms that are suggested by the building itself just looking to

160

the geometry, the nature of the structural elements, the cracking state, the interventions

161

occurred over the years, etc. The linear kinematic analysis is a powerful tool that can be used

162

to describe local mechanisms that have been observed after disruptive earthquakes. If only

163

out-of-plane mechanisms are considered, then they can be basically grouped in three

164

categories: (i) overturning, (ii) vertical flexural behaviour and (iii) horizontal flexural

165

behaviour. In this paper only the first two categories are considered since they were the main

166

cause of collapses during the Central Italy earthquake. In addition, for these two mechanisms

167

the presence of a reinforced concrete roof is more crucial. For the overturning, two cases

168

were studied, a 1-storey and a 2-storey building, whereas for the vertical flexural behaviour

169

only the 2-storey building was studied, taking into account also the effect of the inter-storey.

170

The considered macro-elements are the walls, the floor, and the reinforced concrete roof. This

171

one is assumed as an element that transfers only vertical loads to the walls and no lateral

172

thrusts. Therefore, it is modelled as a rigid block with a large mass. The effects of

173

perpendicular walls are neglected, as in many real cases there are no connections. All models

174

consider alternatively the presence and the absence of connections between roof and walls

175

(by means of a ring beam) and between floor and walls. When there is a ring beam the roof is

176

fixed to the walls and moves with them, while when there is no connection the roof is

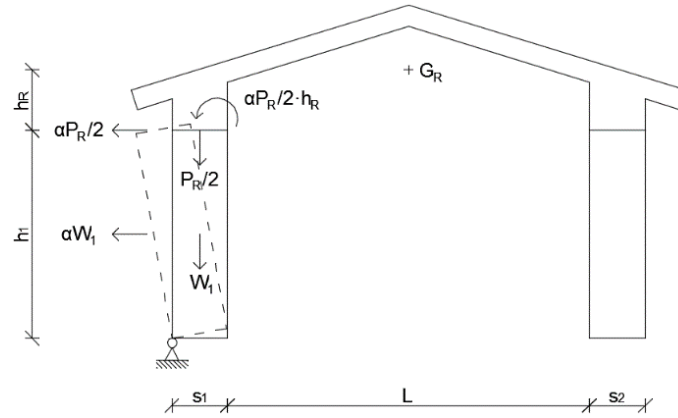
177

considered simply supported.

178

179 3.1 1-storey overturning without ring beam

180 This is the case of simple overturning where there is no ring beam, and therefore no
 181 connection between the top of the wall and the roof. For this reason, the macro-elements are
 182 independent one from another and only one wall is subjected to overturning. Figure 2
 183 illustrates the mechanism and the forces involved. W_l is the weight of the left wall, P_R is the
 184 weight of the roof applied in its center of mass G_R and split equally between the two walls.



185

186 Figure 2. Calculating scheme for 1-storey overturning without ring beam.

187

188 According to Eq. (1) the load multiplier that leads to collapse for this configuration is given
 189 by Eq. (6):

$$190 \quad \alpha_C = \frac{W_l \cdot \frac{s_1}{2} + \frac{P_R}{2} \cdot \frac{s_1}{2}}{W_l \cdot \frac{h_1}{2} + \frac{P_R}{2} \cdot (h_1 + h_R)} \quad (6)$$

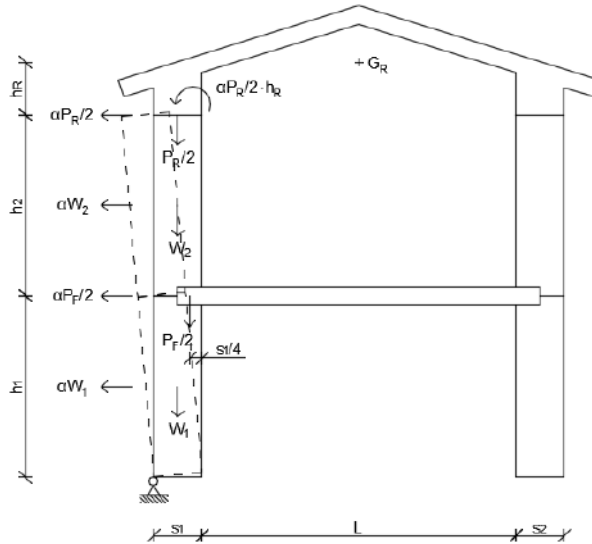
191 where s_l is the thickness of left wall; h_l is the height of walls; h_R is the distance between the
 192 top of walls and the center of mass (G_R) of the roof, and L is the span length.

193

194 3.2 1-storey overturning with ring beam

195 If there is a ring beam the two external walls are connected at the top to realize the so called
 196 “box-like” behaviour. Macro-elements are no more independent, so they move together until
 197 the loss of equilibrium. As shown in Figure 3 both walls rotate around the hinge at the bottom
 198 under a seismic action. The effect of the ring beam is modelled using a force acting in the
 199 opposite direction of the kinematic movement. To a first approximation, it can be calculated
 200 as the product of the friction coefficient μ and the weight of the roof P_R . For the estimation of
 201 the friction coefficient there are many experimental tests available in literature [24].

202 However, since the proposed model is simplified and no detailed information about the
 203 materials were available, the guidelines provided by national codes were followed [5, 6]. In
 204 particular, as a precautionary measure, a friction coefficient of 0.4 was used, which is the
 205 lowest among the suggested values.



225

226

Figure 4. Calculating scheme for 2-storey overturning without ring beam.

227

228 3.4 2-storey overturning with ring beam

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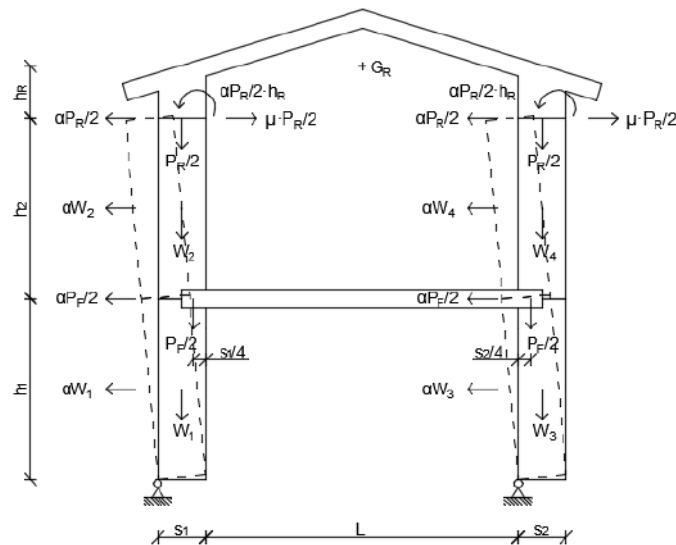
233

As already seen in the 1-storey case, the ring beam at the roof level connects the walls together, so they can overturn at the same time. Once again, the connection is modelled as a friction force proportional to the weight of the roof (Figure 5). In this case, the inter-storey floor is well connected to the walls and moves together with them. Eq. (9) provides the collapse load multiplier for this case:

$$234 \quad \alpha_c = \frac{\left(W_1 + W_2 + \frac{P_R}{2}\right) \cdot \frac{s_1}{2} + \frac{P_F}{2} \cdot \frac{3s_1}{4} + \left(W_3 + W_4 + \frac{P_R}{2}\right) \cdot \frac{s_2}{2} + \frac{P_F}{2} \cdot \frac{s_2}{4} + (\mu \cdot P_R) \cdot (h_1 + h_2)}{\left(W_1 + W_3\right) \cdot \frac{h_1}{2} + P_F \cdot h_1 + \left(W_2 + W_4\right) \cdot \left(h_1 + \frac{h_2}{2}\right) + P_R \cdot (h_1 + h_2) + P_R \cdot h_R} \quad (9)$$

235

where W_4 is the weight of the right masonry panel of the upper level.



236

237

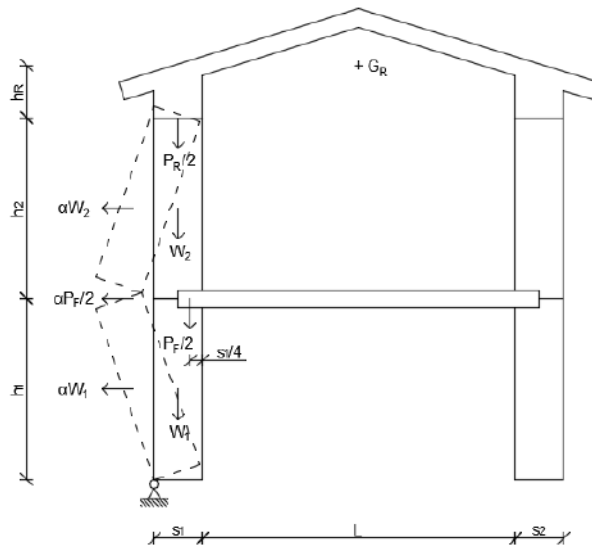
Figure 5. Calculating scheme for 2-storey overturning with ring beam.

238

239 **3.5 2-storey flexural behaviour without connection between floor and walls**

240 Vertical flexural behaviour can occur in any part of the wall. It can be seen as a triple-hinged
 241 arch where, in this specific case, the hinges are located at the bottom, at the top, and at the
 242 inter-storey level. This means that the mechanism is activated by the horizontal inertial force
 243 caused by the floor during the seismic action. The upper level wall is connected at the top of
 244 the roof, whereas the inter-storey floor is not connected to the walls (Figure 6). The collapse
 245 load multiplier α_C is given by Eq. (10):

246
$$\alpha_C = \frac{W_1 \cdot \frac{s_1}{2} + W_2 \cdot \left(s_1 + \frac{s_1}{2} \cdot \frac{h_1}{h_2} \right) + \frac{P_F}{2} \cdot \frac{3s_1}{4} + \frac{P_R}{2} \cdot \left(s_1 + \frac{s_1}{2} \cdot \frac{h_1}{h_2} \right)}{W_1 \cdot \frac{h_1}{2} + W_2 \cdot \left(h_1 \cdot \frac{h_1}{h_2} + \frac{h_2}{2} \cdot \frac{h_1}{h_2} \right) + \frac{P_F}{2} \cdot h_1} \quad (10)$$



247

248 Figure 6. Calculating scheme for vertical flexural behaviour without floor-walls connection.

249

250 **3.6 2-storey flexural behaviour with connection between floor and walls**

251 In this model the floor is well connected to the walls so that it can pull them together, but
 252 eventually it detaches. To represent this type of connection, a friction force proportional to
 253 the floor weight is considered at the inter-storey level (Figure 7).



Figure 8. View of the building used as case study before and after the earthquake.

272

273

274

275 As already mentioned, most of the data are geometrical so they can be easily collected. Only
 276 the PGA needs to be calculated referring to the information given by the Italian code [5]. In
 277 this case, the location is fixed, so the value can be determined analytically. The value used for
 278 S_s is referred to rock soil, while the one used for S_t is referred to the top of a hill as there is
 279 evidence of a slope behind the building.

280

281 4.1.1 Pescara del Tronto results

282 The results using the linear kinematic analysis and verification are reported in Table 1.

283

284 Table 1. Collapse load multipliers and safety factors for the Pescara del Tronto case study.

	1-storey overturning without ring beam	1-storey overturning with ring beam
α_c	0.10	0.26
SF	0.57	1.48

285

286 Comparing the load multipliers, it can be observed that the first mechanism is more likely to
 287 happen as the 1-storey overturning with ring beam case has a collapse load 2.6 times higher
 288 than the 1-storey overturning without ring beam. This is confirmed by the fact that the safety
 289 factor in the case without ring beam is lower and it is below the safety threshold. Indeed, as
 290 shown in Figure 8 there is no connection between roof and walls. If this kind of analysis was
 291 carried out in a pre-earthquake situation, it would have been possible to demonstrate how the
 292 building was unsafe towards a seismic event with a return period of 475 years. The actual
 293 demand of 2016 earthquake was larger than the one used as input for this verification
 294 procedure. However, the aim of the method is to report if retrofitting is needed to improve the
 295 level of safety. Validation analyses were not performed at this stage, but adequate
 296 interventions could have likely prevented the structure from a complete collapse.

297 A sensitivity analysis was carried out to assess the impact of the chosen input parameters on
 298 results. Most relevant outcomes were then plotted in graphs with the varying parameter on
 299 the x axis and the safety factor SF on the y axis. Figure 9(a) shows that when there is no

300 friction the two mechanisms are equivalent because of the symmetry of the systems and the
301 configuration is unsafe. The minimum friction coefficient to ensure a safety factor greater
302 than 1 is 0.2. As predictable, the impact of the friction is overall positive: it is enough to build
303 a ring beam able to ensure a friction coefficient of 0.4 and triple the safety factor. In Figure
304 9(b) the span length is varying, resulting in an increase or decrease in the weight of the roof.
305 This has almost no effects in the case without ring beam, which remains unsafe in the whole
306 range of variation. On the other hand, since the friction force is proportional to the weight of
307 the roof, increasing the span length leads to a significant increment of the safety factor. When
308 the masonry specific weight of the masonry varies, safety factor remains almost constant
309 (Figure 9(c)). The weight of the walls has indeed a twofold effect since it contributes to both
310 stabilizing and overturning forces. Also, looking at the equations that lead to determine SF (in
311 particular Eqs. (6-7) and Eq. (2)), it can be noticed how the weight appears always at both
312 nominator and denominator. Therefore, in the calculation steps its variation tends to have
313 negligible effects. Figure 9(d) shows the influence of the wall thickness. In the first case,
314 without ring beam, an increment of the wall thickness corresponds to a linear and
315 considerable increment of the SF . However, almost 70 cm walls are required to be in the safe
316 area. In the second case, there is an asymptotic trend. It means that for low thicknesses the
317 presence of the ring beam is effective (e.g. for $s = 20$ cm the SF is two times the SF in the
318 case with no ring beam), but for high thicknesses the structure is so massive that the presence
319 of the ring beam at the top is irrelevant. Finally, in Figure 9(e) different soil categories are
320 taken into account by varying the abovementioned parameter S_s . Following the guidelines
321 provided by the Italian seismic codes, the possible values S_s can assume were calculated
322 (Table 2). A rock soil allows to have higher safety factors, whereas if the quality gets worse
323 there is a decreasing trend, except for category E (coarse soil upon a stiff or soft soil).

324

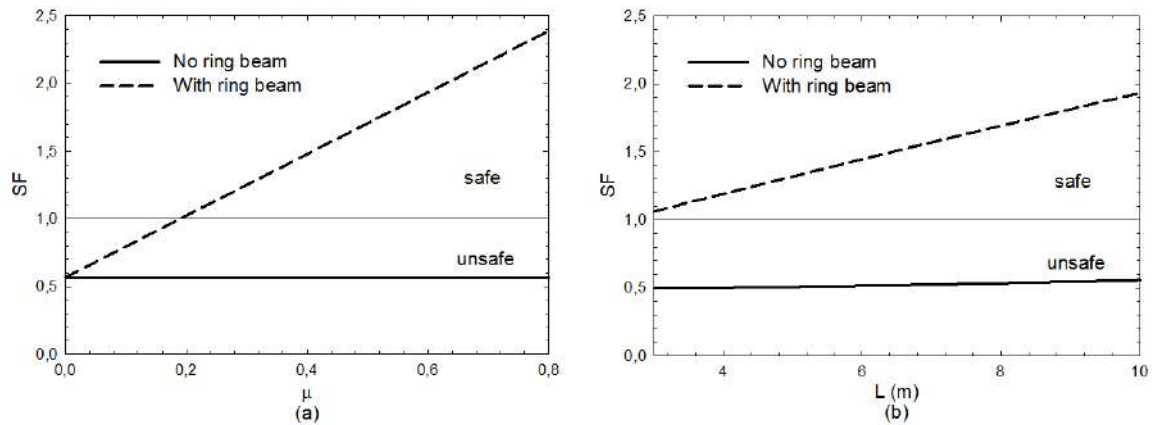
325 Table 2. Values of the parameter S_s for different soil categories.

Soil category	Description	S_s
A	Rock soil ($V_{s30} > 800$ m/s)	1.00
B	Soft rock and very dense soil (360 m/s $< V_{s30} < 800$ m/s)	1.16
C	Stiff soil (180 m/s $< V_{s30} < 360$ m/s)	1.33
D	Soft soil ($V_{s30} < 180$ m/s)	1.48
E	Coarse soil upon stiff or soft soil	1.33

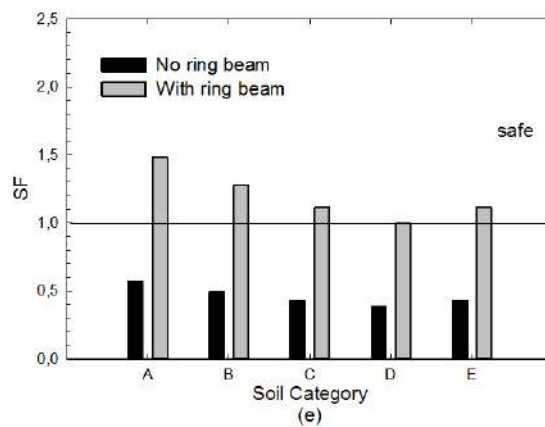
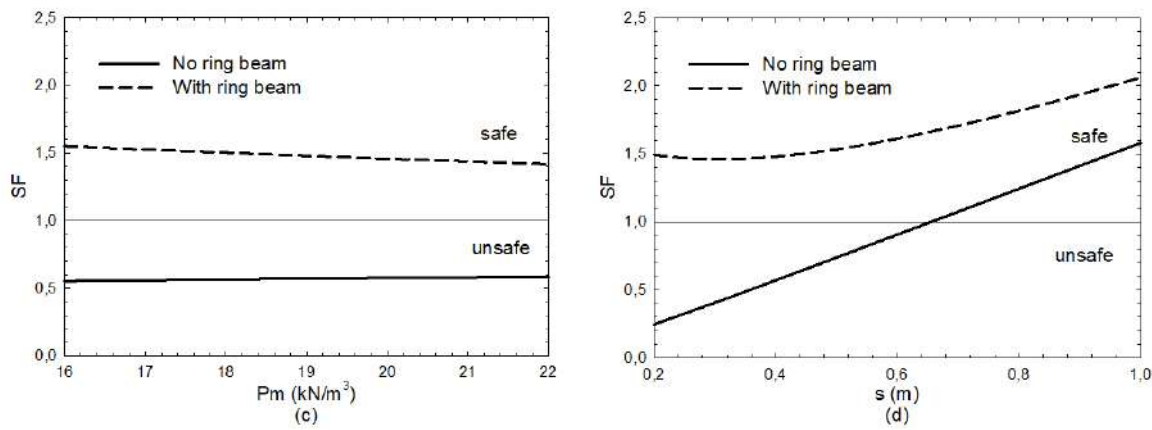
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329



330

331 Figure 9. (a) Friction coefficient vs. safety factor, (b) span length vs. safety factor, (c)
 332 masonry specific weight vs. safety factor, (d) wall thickness vs. safety factor, (e) soil category
 333 vs. safety factor.

334

335 4.2 Amatrice case study

336 The 2-floor models were tested through the building in Figure 10 which was located in
 337 Amatrice. The data used as input for the analysis are the following: wall thickness ($s_1 = s_2$):
 338 0.4 m; roof thickness (s_R): 0.15 m; wall height ($h_1 = h_2$): 3 m; wall length (l_w): 11 m; span
 339 length (L): 6.5 m; distance between the center of mass of the roof and the top of the wall (h_R):

340 0.4 m; specific weight of the masonry (P_m): 19 kN/m³; specific weight of the reinforced
 341 concrete (P_c): 21 kN/m³; floor weight (P_f): 4 kN/m²; friction coefficient roof – wall (μ): 0.4;
 342 friction coefficient floor – wall (μ): 0.5; PGA for a return period of 475 years (a_g): 2.538
 343 m/s²; S_s : 1 (rock soil); S_i : 1 (flat area).



344
 345 Figure 10. View of the building used as case study before and after the earthquake.

346
 347 *4.2.1 Amatrice results*

348 The results obtained using the linear kinematic analysis are summarized in Table 3:

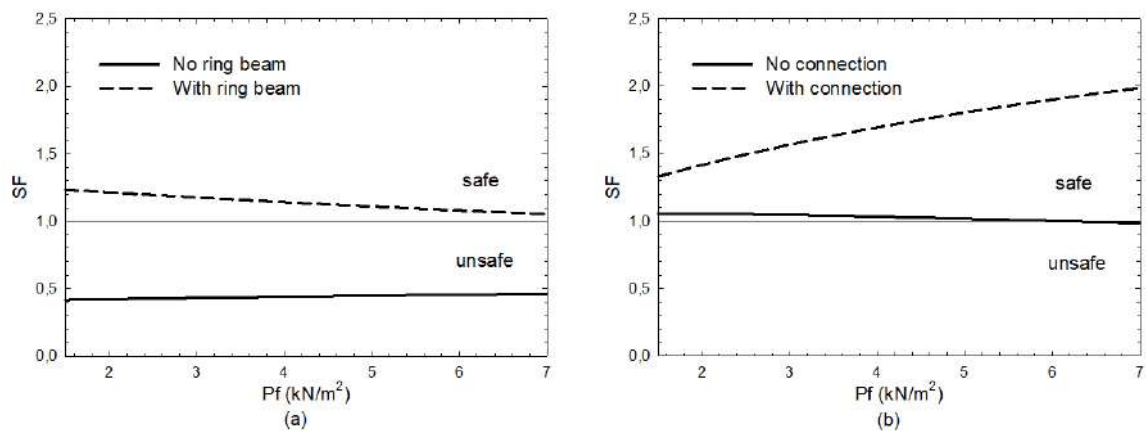
349
 350 Table 3. Collapse load multipliers and safety factors for the Amatrice case study.

	2-storey overturning without ring beam	2-storey overturning with ring beam	2-storey vertical flexural behaviour without connection between floor and walls	2-storey vertical flexural behaviour with connection between floor and walls
α_C	0.06	0.16	0.16	0.27
SF	0.44	1.14	1.03	1.70

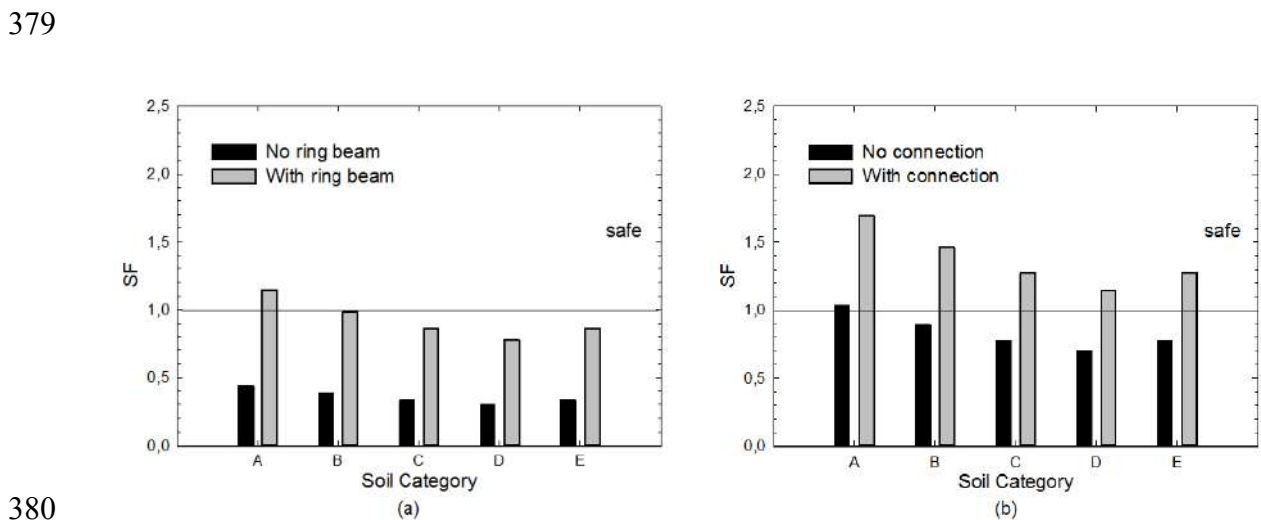
351
 352 Comparing these results, it is clear that the configuration of overturning with no ring beam is
 353 the less safe and the most probable. The presence of a ring beam increases the collapse load
 354 multiplier of about 2.7 times for the overturning mechanism. On the other hand, the model
 355 with connection at the top of the walls and at floor level is the safest and less probable case.
 356 For the vertical flexural behaviour, the case with connection between floor and walls has 1.7
 357 times as high of a collapse load as the case without connection has. Finally, it is possible to
 358 observe that the mechanisms of overturning with ring beam and vertical flexural behaviour
 359 without connection at floor level are almost equivalent.

360 Also for this case study, sensitivity analyses were performed for all parameters. The trends of
 361 variation of other parameters are really similar to the previous ones, thus the considerations
 362 done for the 1-storey model are still valid. The substantial difference between 2-storey
 363 overturning and vertical flexural behaviour is that in the second case the safety factors are
 364 higher. Omitting obvious and well-known results, it is worth to comment what happens when
 365 the floor weight is changing (Figure 11). In both cases of overturning and in the case of

366 vertical flexural behaviour without connection, when the floor weight varies the safety factor
 367 is not subject to significant changes. A slight reduction of SF can be observed in the
 368 overturning scenario for the model with ring beam as the floor weight increases. A heavier
 369 floor leads to greater horizontal inertial forces which contribute to the overturning. Instead,
 370 for the mechanism of vertical flexural behaviour with connection, a heavy floor has a positive
 371 impact since the stabilizing force increases. As far as the variation of soil category is
 372 concerned, the values of the S_s coefficient that were used in the previous case study are still
 373 valid (Table 2). It is interesting to notice that if the category of soil was not “A” (rock soil),
 374 then only the mechanism of vertical flexural behaviour with connection would be safe
 375 (Figure 12).



376
 377 Figure 11. Floor weight vs. safety factor for the (a) overturning mechanism and the (b)
 378 vertical flexural behaviour.



380
 381 Figure 12. Soil category vs. safety factor for the (a) overturning mechanism and the (b)
 382 vertical flexural behaviour mechanism.

383
 384 **5. Nonlinear kinematic analysis**

385 Nonlinear kinematic analyses were carried out with the aim of supporting the results obtained
 386 from linear analyses. The whole procedure follows the method proposed by Italian codes [5,

387 6], which is presented hereafter just in its main steps. The underlying idea of the method is to
 388 determine the trend of the horizontal action that the structure is progressively able to
 389 withstand during the collapse mechanism's evolution. This can be seen as the capacity curve
 390 of an equivalent single degree of freedom system. The ultimate displacement capacity of the
 391 local mechanism is then defined and compared with the seismic demand. Similarly to the
 392 linear case, a multiplier α is introduced and defined as the ratio between the applied
 393 horizontal forces and the displacement d_k of a control point. The horizontal multiplier of
 394 loads is evaluated at various configurations of the kinematic chain until reaching the collapse
 395 condition, which is identified by a null multiplier α , and a displacement $d_{k,0}$. Assuming that
 396 involved actions (i.e. weights, external and internal forces) are constant during the evolution
 397 of the mechanism, the curve is almost linear. In this case, only the evaluation of the
 398 displacement $d_{k,0}$ is required, and the curve is described by Eq. (12):

$$399 \quad \alpha = \alpha_0(1 - d_k / d_{k,0}) \quad (12)$$

400 where α_0 denotes the value of the multiplier capable of activating the analysed mechanism.
 401 The problem can be solved considering a configuration varied from the static condition and
 402 calculating the induced finite rotation θ_k by means of virtual work principle. Reaching the
 403 collapse situation, the overturning moment equals the stabilizing moment, and the resulting
 404 nonlinear equation gives the final rotation $\theta_{k,0}$. Once the latter is determined, the
 405 corresponding displacement $d_{k,0}$ can be obtained. Let the control point be the center of gravity
 406 of vertical forces, and h_{bar} be its distance from the base hinge. Eq. (13) expresses the relation
 407 between the rotation angle and the displacement of the control point related to ultimate
 408 capacity towards horizontal actions.

$$409 \quad d_{k,0} = h_{bar} \sin \theta_{k,0} \quad (13)$$

410 At this point, it is possible to define the $\alpha - d_k$ curve according to Eq. (12). The equivalent
 411 capacity curve should now be determined. It describes the relation between the acceleration
 412 a^* that activates the mechanism and displacement d^* . The first is obtained through Eq. (2) as
 413 in the linear case, whereas d^* is given by Eq. (14).

$$414 \quad d^* = d_{k,0} \frac{\sum_{i=1}^{n+m} P_i \delta_{x,i}^2}{\delta_{x,k} \sum_{i=1}^{n+m} P_i \delta_{x,i}} \quad (14)$$

415 where: $n+m$ is the number of weight forces that generates horizontal forces upon the macro-
 416 elements; P_i is the generic weight force; $\delta_{x,i}$ is the virtual horizontal displacement of the
 417 application point of P_i ; $\delta_{x,k}$ is the horizontal virtual displacement of the control point.

418 Making the same assumption done for the $\alpha - d_k$ curve, the capacity curve can be derived from
 419 Eq. (15).

$$420 \quad a^* = a_0^*(1 - d^* / d_0^*) \quad (15)$$

421 where d_0^* is the equivalent displacement corresponding to $d_{k,0}$.

422 The verification for the life safety limit state consists in a comparison between the ultimate
 423 displacement capacity d_u^* of the local mechanism and the spectral displacement evaluated at
 424 the period T_s (Eq. (16)):

$$425 \quad T_s = 2\pi \sqrt{\frac{d_s^*}{a_s^*}} \quad (16)$$

426 where a_s^* , is the acceleration correspondent to the displacement d_s^* , which is equal to $0.4d_u^*$,
 427 and $d_u^* = 0.4d_0^*$. Therefore, if d_u^* is greater than the spectral displacement $S_{De}(T_s)$, the
 428 verification is fulfilled (Eq. (17)).

$$429 \quad SF = \frac{d_u^*}{S_{De}(T_s)} \geq 1 \quad (17)$$

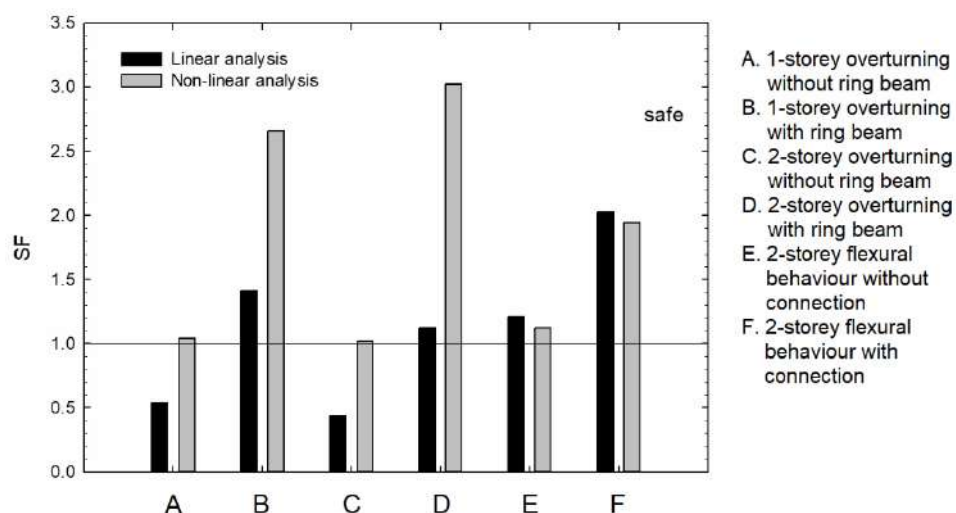
430 Table 4 shows obtained results for all analysed models. As expected, a lack of connection is
 431 responsible of a small safety factor, always around 1 for the three studied cases. On the other
 432 hand, the presence of a ring beam or floor-wall connections ensures much more capacity to
 433 withstand larger ultimate displacements, which also means a higher safety level. The
 434 difference is less exaggerated in the case of vertical flexural behaviour since the connection
 435 between the floor and the wall makes the structure more rigid.

436 Comparing the linear with the nonlinear kinematic analysis in terms of safety factors, the bar
 437 chart of Figure 13 can be plotted. The nonlinear kinematic analysis usually provides safety
 438 coefficients that are two times or more the coefficient obtained with the linear analysis. This
 439 confirms that a fast and preliminary analysis, such as the linear one, provides precautionary
 440 results and it is recommended to understand if further investigations are needed. The
 441 described trend is inverted in the cases of vertical flexural behaviour, although the nonlinear
 442 safety factors are just slightly lower than linear ones. This phenomenon is due to the fact that
 443 the weight of a reinforced concrete roof has a particularly positive effect for these two
 444 models. In addition, the same analyses were repeated several times increasing the weight of
 445 the roof for each iteration. In all cases the safety factor increased, but the increment in the
 446 linear analysis was greater than in the nonlinear one.

447 Table 4. Results of the nonlinear kinematic analysis.

	Pescara del Tronto		Amatrice			
	1-storey overturning without ring beam	1-storey overturning with ring beam	2-storey overturning without ring beam	2-storey overturning with ring beam	2-storey flexural behaviour without connection	2-storey flexural behaviour with connection
d_u^* [m]	0.089	0.227	0.093	0.326	0.041	0.074
SF	1.04	2.66	1.02	3.02	1.12	1.95

448



449

450 Figure 13. Comparison between safety factors resulting from linear and non-linear analyses.

451

452 6. Conclusions

453 The 2016 Central Italy earthquake caused damages and collapses of many masonry buildings,
 454 including those retrofitted with reinforced concrete roofs. This type of retrofit intervention
 455 seems to facilitate some collapse mechanisms if not properly executed. The paper presents a
 456 simplified procedure to evaluate the seismic performance of masonry buildings retrofitted
 457 with reinforced concrete roofs based on the linear kinematic analysis. Despite the analysis
 458 method is well known, the proposed models have the advantage of explicitly considering the
 459 interaction between the roof and the walls. Moreover, they require only few input parameters
 460 that can be easily obtainable. The collapse load multiplier obtained as output of the linear
 461 kinematic analysis was used to define a safety factor to have an idea whether a structure can
 462 be considered safe towards a certain collapse mechanism.

463 The introduced analytical models are based on the overturning and vertical flexural behaviour
 464 collapse mechanisms. In particular, the overturning scenario was analysed both for a 1-storey
 465 and a 2-storey building, while the vertical flexural behaviour was considered for a 2-storey
 466 building. Each scenario is studied twice: one configuration assuming that there is no
 467 connection among the involved macro-elements and another one that provides for an
 468 effective connection. The resulting six models were applied to two case studies, a 1-storey
 469 and a 2-storey building both collapsed during the 2016 Central Italy earthquake. Results
 470 highlight the importance of an efficient connection between the reinforced concrete roof and
 471 masonry walls and that some configurations are unsafe, which means additional
 472 investigations and possibly retrofit intervention are required. To account for the uncertainties
 473 introduced by the simplified models, sensitivity analyses were performed. These allowed to
 474 evaluate the influence of each input parameter to the overall safety level of the structure,
 475 highlighting peculiar aspects in each mechanism. Nonlinear kinematic analyses were also
 476 carried out and results were compared to those obtained from linear analyses. It was possible
 477 to observe that the linear analysis is faster and more precautionary as it provides smaller
 478 safety factors with respect to the nonlinear one.

479 Overall, the proposed analytical models represent an effective yet simple tool that can serve
480 as a preliminary evaluation of the safety level of masonry structures retrofitted with
481 reinforced concrete roofs. The method is not meant to be an alternative to other more refined
482 analyses, such as finite element methods, that would allow for a more accurate safety
483 verification. However, due to its simplicity, the procedure could be recommended when only
484 few input data are available and be applied even by non-professional users to understand if
485 further investigations and/or retrofit interventions are needed.

486

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Highlights

- The paper presents a preliminary simplified procedure to evaluate the seismic safety of masonry buildings retrofitted with reinforced concrete roofs.
- Effective connection between RC roof and walls is crucial to be in a safe condition
- Sensitivity analyses performed to evaluate the influence of each input parameter
- In most cases nonlinear kinematic analyses provide larger safety factors

The role of reinforced concrete roofs in the seismic performance of masonry buildings

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Abstract

The 2016 Central Italy earthquake caused many collapses of existing masonry buildings that had previously been retrofitted with reinforced concrete roofs. The aim of this paper is to explore the role of these roofs in the seismic behaviour of masonry buildings. Simple analytical models are presented to illustrate two typical out-of-plane collapse mechanisms: wall overturning and vertical flexure. The models are based on linear kinematic analysis, which allows fast modelling and calculation of a coefficient that can be used to assess the safety level of a structure. Nonlinear kinematic analyses were also performed. Both methods were applied to two case studies taken from areas struck by the earthquake. Results show that linear analysis represents an effective tool for preliminary verifications that can allow one to understand whether retrofit interventions are needed.

Keywords: masonry, reinforced concrete roof, collapse, retrofit, kinematic analysis, Central Italy earthquake

1. Introduction

After the 24th August 2016 Central Italy earthquake, most of the buildings of small towns nearby the epicenter were declared unsafe and several structures collapsed completely. Poor material quality and scant building techniques were certainly the main reason of collapses. However, inadequate retrofit interventions also contributed to the disruptive effect of the seismic event. For instance, the replacement of the old wooden roofs with reinforced concrete roofs seemed to facilitate some mechanisms that led to severe damages and collapses. This type of retrofitting was broadly adopted in the 80s and 90s since it was believed to be effective against seismic actions. In fact, it was the Italian code itself to recommend it [1]. Moreover, at that period there was a massive use of concrete that led to a gradual abandon of research and experimental tests on masonry [2]. The overall idea was to put robust structures such as RC roofs and floors connected to perimetric walls by means of RC ring beams to avoid independent movements of masonry macro-elements. After Tolmezzo earthquake in 1976, this and other retrofitting techniques became part of technical codes, until Umbria and Marche earthquake in 1997 [3, 4].

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63 40 This event pointed out the disadvantages of heavy and stiff roofs and floors. In fact, if vertical
64 41 structures are not robust enough, they are indeed the primary cause of collapses. The significant
65 42 stiffness and load increment at the top have led to the collapse of the walls, which were made of
66 43 poor materials and not strengthened. Conversely, there were also many cases of masonry
67 44 structures retrofitted with reinforced concrete roofs that withstood the earthquake with no
68 45 significant damages (Figure 1).



46
47 Figure 1. Masonry buildings retrofitted with concrete roof not collapsed after the earthquake in
48 Pescara del Tronto (a-b) and small villages near Accumoli (c-d).

50 However, there is no guarantee that those buildings are safe. Therefore, in this paper a simple
51 verification procedure that is able to estimate the level of safety of masonry buildings with
52 reinforced concrete roofs is implemented. The adopted approach is based on the linear kinematic
53 analysis, which is also described by Italian codes [5, 6]. Despite the method is well known in its
54 theoretical formulation, it is rarely used and usually the effect of the roof and the connection
55 among structural elements are neglected. This research contributes to the current literature with
56 practical applications of the kinematic analysis introducing simplified analytical models that
57 take into account the effect of reinforced concrete roofs. An additional advantage of the
58 proposed models is that the number of input parameters has been reduced as much as possible so
59 that the analysis does not require any particular investigation or survey to be carried out. A
60 safety factor was also defined to assess the safety level of the building towards different collapse

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121 61 mechanisms. The choice of a simplified procedure has been made in order to have a fast tool
122 62 which could be used even by non-professional users. The method would allow property owners
123 63 to understand if they are in danger. For instance, if the obtained safety factor is low or close to
124 64 the unsafe threshold, further investigations should be conducted. More detailed methods have
125 65 been studied by many authors to describe masonry buildings behaviour, but they need to be
126 66 calibrated and the input data are often not accessible [7-10]. Obviously, results will not be as
127 67 accurate, and a certain margin of error should be taken into account in final considerations.
128 68 Nonetheless, they can provide relevant preliminary information about the structure. In addition,
129 69 in the literature there is a number of studies about masonry where analytical models turned out
130 70 to be highly effective and close to the real behaviour [11, 12].

133 71 After defining the formulation, the method is applied to different models describing the
134 72 overturning and the vertical flexural behaviour. The models derive from those commonly used
135 73 to study the out-of-plane mechanisms [13-15] and the arch rocking [16-18]. To analyse the
136 74 influence of the connection between the roof and the floor to the walls, a ring beam is also
137 75 considered. The presence of a reinforced concrete (RC) ring beam is dangerous if it is not well
138 76 connected to masonry walls and if the latter is not strengthened. Furthermore, the spread of
139 77 reinforced concrete in the construction sector, led to wrong applications in the interventions of
140 78 existing buildings. Nowadays there are many solutions to realize effective structural
141 79 connections, such as reinforced masonry ring beams [19]. The use of innovative composite
142 80 materials has become a common practice in retrofit strategies. Several studies have been carried
143 81 out in this field which has allowed to investigate the behavior of strengthened beams [20, 21]
144 82 and strengthened masonry walls through out-of-plane tests [22].

148 83 Two case studies taken from two towns struck by the abovementioned earthquake were
149 84 analyzed, but the method can be extended to any building by choosing appropriate parameters.
150 85 Both examples were selected by considering typical houses in the area, built with local materials
151 86 and poor construction techniques and retrofitted with reinforced concrete roofs. The first one is
152 87 1-storey building while the second one has two storeys and thus also the action of the inter-
153 88 storey floor is considered in the model. For each model, the linear kinematic analysis is repeated
154 89 for different values of the input parameters, as they could be affected by uncertainty. In this way
155 90 it is possible to see the influence of a single parameter and what happens if it is over-estimated
156 91 or under-estimated. Finally, nonlinear analyses are performed in order to compare the results and
157 92 understand if the additional computational effort of a more refined method is worth it.

160 93 161 94 **2. The linear kinematic analysis**

163 95 In existing masonry buildings there are often collapses due to a loss of equilibrium of some
164 96 portions of bearing structures. In general, these types of mechanisms happen when seismic
165 97 forces act in the out-of-plane direction. The linear kinematic analysis can be used to study
166 98 these phenomena and for the verification process. It is based on the choice of the possible
167 99 mechanisms that are most likely to happen. These ones are assumed by evaluating the current
168 100 cracking state and analyses performed on similar buildings. In the literature there are plenty
169 101 of studies on historical buildings, such as churches, which are helpful to clarify how the
170 102 collapse process activates and evolve [23]. The ability to detect the most probable
171 103 mechanisms is crucial to prevent local or global collapses, since it is possible to run specific
172 104 analyses and consequently suggest specific interventions.

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181 105 The linear kinematic approach schematizes the building in a discrete number of macro-
182 106 elements which move according to their boundary conditions. For this reason, the
183 107 assumptions are that the material has no tensile strength and infinite compressive strength. In
184 108 each rigid block, vertical loads (including dead and external loads) and a system of horizontal
185 109 forces are applied. Horizontal forces are proportional to the vertical loads through a
186 110 coefficient called load multiplier (α). Incrementing the load multiplier, it is possible to
187 111 evaluate the horizontal force that activates a specific mechanism. α_C is named the collapse
188 112 load multiplier, and it is calculated with the principle of virtual works. Therefore, the total
189 113 work of the external forces (L_e) has to be equal to the total work of the internal forces (L_i)
190 114 which in this case is null as shown in Eq. (1):

$$191 \quad 114 \quad L_e = \alpha_C \left(\sum_{i=1}^n W_i \cdot \delta_{x,i} + \sum_{j=n+1}^{n+m} W_j \cdot \delta_{x,j} \right) - \sum_{i=1}^n W_i \cdot \delta_{y,i} - \sum_{h=1}^o F_h \cdot \delta_h = L_i = 0 \quad (1)$$

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196 116 where: n is the number of the weight forces applied to all macro-elements; m is the number of
197 117 weight forces that generate horizontal forces upon macro-elements; o is the number of
198 118 external forces; W_i is the generic weight force; W_j is the generic weight force that generates
199 119 horizontal forces upon macro-elements; $\delta_{x,i}$ is the virtual horizontal displacement of the point
200 120 where the i -th weight force is applied; $\delta_{y,i}$ is the virtual vertical displacement of the point
201 121 where the i -th weight force is applied; F_h is the generic external force; δ_h is the virtual
202 122 displacement of the point where the generic external force F_h is applied. Eq. (1) often
203 123 becomes an equilibrium equation between a stabilizing moment and an overturning moment,
204 124 so it is not necessary to calculate the virtual displacement. The method is also used to
205 125 determine the most probable collapse mechanism which is the one that requires less energy to
206 126 be activated (i.e. the one with the lower load multiplier). However, the decay conditions of
207 127 masonry should never be neglected since they are able to reveal if a specific mechanism has
208 128 already been activated. Once α_C is calculated, it is possible to obtain the acceleration that
209 129 generates the mechanism (Eq. (2)).

$$210 \quad 129 \quad a_0^* = \frac{\alpha_C \cdot \sum_{i=1}^{n+m} P_i}{M^* \cdot FC} \quad (2)$$

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217 131 where FC is a coefficient that depends on the level of knowledge about the masonry
218 132 structure. The level of knowledge is based on information like geometry, construction details,
219 133 and material properties. Such data can be acquired in different ways, from generic research
220 134 and visual inspection to extensive tests and measurements. Since the material strength is not
221 135 considered in this research, only basic information about geometric characteristics, type of
222 136 masonry panels and construction details was collected. According to the Italian code [6],
223 137 three level of knowledge can be identified: limited, extended and exhaustive. Due to the
224 138 limited available data, the level of knowledge is limited. In this case, the code reports the FC
225 139 coefficient has to be assumed equal to 1,35 which reduces the acceleration that generates the
226 140 mechanism. M^* is the participating mass, calculated considering the virtual displacements of
227 141 the points where the loads are applied, as shown in Eq. (3) [6]:

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \cdot \delta_{x,i} \right)^2}{g \cdot \sum_{i=1}^{n+m} P_i \cdot \delta_{x,i}^2} \quad (3)$$

The acceleration given by Eq. (2) has now to be compared with an allowable acceleration. This one is given by Eq. (4), which is valid when the analysed blocks are in contact with the ground, meaning that the mechanism involves ground floor walls:

$$a^* = \frac{a_g \cdot S}{q} \quad (4)$$

where: a_g is the peak ground acceleration at the site determined, as indicated by Italian codes, for a return period of 475 years; q is the reduction factor which can be assumed equal to 2 for regular masonry structures; S is given by the product of two coefficients: S_s that depends on the soil category and represents the stratigraphic amplification, and S_t which takes into account the effects of the topographical amplification and depends on the surface configuration of the soil.

It is clear that the acceleration that activates the mechanism should be greater than the allowable one. To quickly verify this, it is possible to introduce a safety factor which is the ratio between the two accelerations (Eq. (5)):

$$SF = \frac{a_0^*}{a^*} \geq 1 \quad (5)$$

3. Analytical models

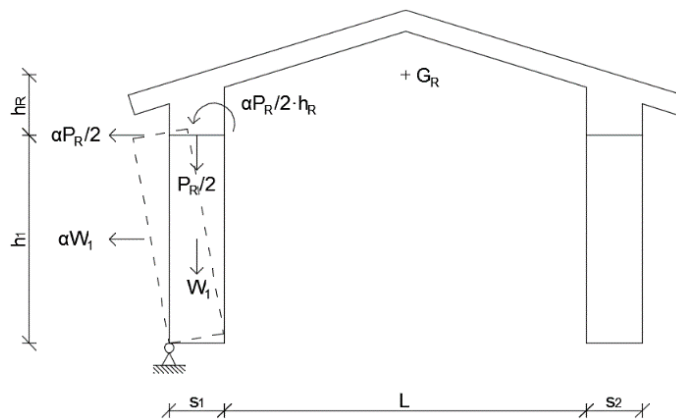
In some cases, there are mechanisms that are suggested by the building itself just looking to the geometry, the nature of the structural elements, the cracking state, the interventions occurred over the years, etc. The linear kinematic analysis is a powerful tool that can be used to describe local mechanisms that have been observed after disruptive earthquakes. If only out-of-plane mechanisms are considered, then they can be basically grouped in three categories: (i) overturning, (ii) vertical flexural behaviour and (iii) horizontal flexural behaviour. In this paper only the first two categories are considered since they were the main cause of collapses during the Central Italy earthquake. In addition, for these two mechanisms the presence of a reinforced concrete roof is more crucial. For the overturning, two cases were studied, a 1-storey and a 2-storey building, whereas for the vertical flexural behaviour only the 2-storey building was studied, taking into account also the effect of the inter-storey. The considered macro-elements are the walls, the floor, and the reinforced concrete roof. This one is assumed as an element that transfers only vertical loads to the walls and no lateral thrusts. Therefore, it is modelled as a rigid block with a large mass. The effects of perpendicular walls are neglected, as in many real cases there are no connections. All models consider alternatively the presence and the absence of connections between roof and walls (by means of a ring beam) and between floor and walls. When there is a ring beam the roof is fixed to the walls and moves with them, while when there is no connection the roof is considered simply supported.

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179 **3.1 1-storey overturning without ring beam**

180 This is the case of simple overturning where there is no ring beam, and therefore no
181 connection between the top of the wall and the roof. For this reason, the macro-elements are
182 independent one from another and only one wall is subjected to overturning. Figure 2
183 illustrates the mechanism and the forces involved. W_l is the weight of the left wall, P_R is the
184 weight of the roof applied in its center of mass G_R and split equally between the two walls.



185

186 Figure 2. Calculating scheme for 1-storey overturning without ring beam.

187

188 According to Eq. (1) the load multiplier that leads to collapse for this configuration is given
189 by Eq. (6):

190

$$\alpha_c = \frac{W_l \cdot \frac{s_1}{2} + \frac{P_R}{2} \cdot \frac{s_1}{2}}{W_l \cdot \frac{h_1}{2} + \frac{P_R}{2} \cdot (h_1 + h_R)} \quad (6)$$

191 where s_l is the thickness of left wall; h_l is the height of walls; h_R is the distance between the
192 top of walls and the center of mass (G_R) of the roof, and L is the span length.

193

194 **3.2 1-storey overturning with ring beam**

195 If there is a ring beam the two external walls are connected at the top to realize the so called
196 “box-like” behaviour. Macro-elements are no more independent, so they move together until
197 the loss of equilibrium. As shown in Figure 3 both walls rotate around the hinge at the bottom
198 under a seismic action. The effect of the ring beam is modelled using a force acting in the
199 opposite direction of the kinematic movement. To a first approximation, it can be calculated
200 as the product of the friction coefficient μ and the weight of the roof P_R . For the estimation of
201 the friction coefficient there are many experimental tests available in literature [24].
202 However, since the proposed model is simplified and no detailed information about the
203 materials were available, the guidelines provided by national codes were followed [5, 6]. In
204 particular, as a precautionary measure, a friction coefficient of 0.4 was used, which is the
205 lowest among the suggested values.

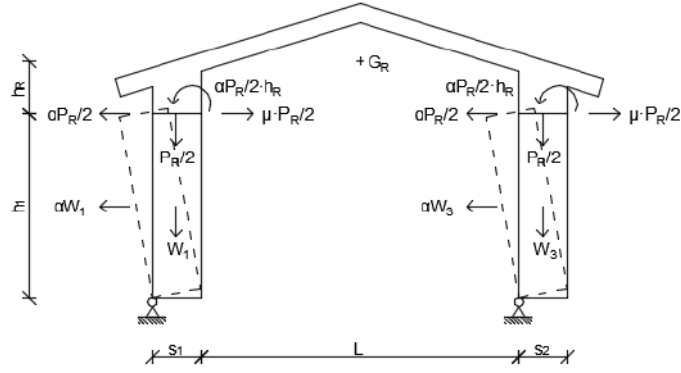


Figure 3. Calculating scheme for 1-storey overturning with ring beam.

Eq. (7) gives the collapse load multiplier in the case of overturning with ring beam:

$$\alpha_C = \frac{\left(W_1 + \frac{P_R}{2}\right) \cdot \frac{s_1}{2} + \left(W_3 + \frac{P_R}{2}\right) \cdot \frac{s_2}{2} + (\mu \cdot P_R) \cdot h_1}{(W_1 + W_3) \cdot \frac{h_1}{2} + P_R \cdot (h_1 + h_R)} \quad (7)$$

where s_1, s_2 are the thicknesses of left and right walls, respectively; W_1 is the weight of the left wall, while W_3 is the weight of the right one.

3.3 2-storey overturning without ring beam

This model is an extension of the 1-storey model, so the same assumptions can be made. However, in this model there is a new macro-element, the floor of weight P_F that is split equally between the left and the right wall. In a simplified manner, in the current model and the ones below, the contact area between the inter-storey floor and ground floor walls is assumed to be half the thickness of walls. To allow a complete rotation of the two masonry panels, the floor is considered disconnected to the walls and the connection to the ground is modelled as a hinge (Figure 4). The formulation of the load multiplier α_C is given by Eq. (8):

$$\alpha_C = \frac{\left(W_1 + W_2 + \frac{P_R}{2}\right) \cdot \frac{s_1}{2} + \frac{P_F}{2} \cdot \frac{3s_1}{4}}{W_1 \cdot \frac{h_1}{2} + \frac{P_F}{2} \cdot h_1 + W_2 \cdot \left(h_1 + \frac{h_2}{2}\right) + \frac{P_R}{2} \cdot (h_1 + h_2) + \frac{P_R}{2} \cdot h_R} \quad (8)$$

where W_2 is the weight of the left masonry panel of the upper level and h_2 is its height.

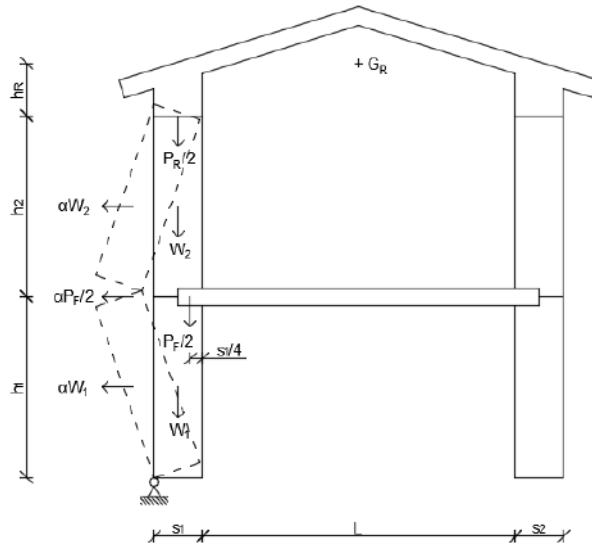
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239 **3.5 2-storey flexural behaviour without connection between floor and walls**

240 Vertical flexural behaviour can occur in any part of the wall. It can be seen as a triple-hinged
 241 arch where, in this specific case, the hinges are located at the bottom, at the top, and at the
 242 inter-storey level. This means that the mechanism is activated by the horizontal inertial force
 243 caused by the floor during the seismic action. The upper level wall is connected at the top of
 244 the roof, whereas the inter-storey floor is not connected to the walls (Figure 6). The collapse
 245 load multiplier α_C is given by Eq. (10):

246
$$\alpha_C = \frac{W_1 \cdot \frac{s_1}{2} + W_2 \cdot \left(s_1 + \frac{s_1}{2} \cdot \frac{h_1}{h_2} \right) + \frac{P_F}{2} \cdot \frac{3s_1}{4} + \frac{P_R}{2} \cdot \left(s_1 + \frac{s_1}{2} \cdot \frac{h_1}{h_2} \right)}{W_1 \cdot \frac{h_1}{2} + W_2 \cdot \left(h_1 \cdot \frac{h_1}{h_2} + \frac{h_2}{2} \cdot \frac{h_1}{h_2} \right) + \frac{P_F}{2} \cdot h_1} \quad (10)$$



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248 Figure 6. Calculating scheme for vertical flexural behaviour without floor-walls connection.

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250 **3.6 2-storey flexural behaviour with connection between floor and walls**

251 In this model the floor is well connected to the walls so that it can pull them together, but
 252 eventually it detaches. To represent this type of connection, a friction force proportional to
 253 the floor weight is considered at the inter-storey level (Figure 7).

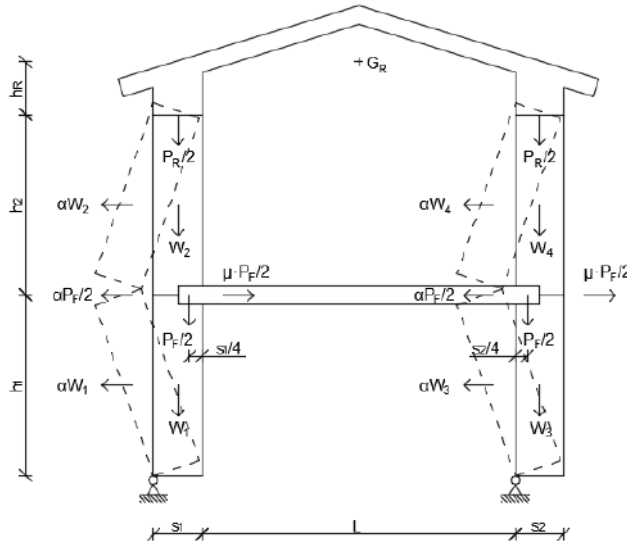


Figure 7. Calculating scheme for vertical flexural behaviour with floor-walls connection.

Eq. (11) allows to get the collapse load multiplier for this model:

$$\alpha_c = \frac{W_1 \cdot \frac{s_1}{2} + W_2 \cdot \left(s_1 + \frac{s_1}{2} \cdot \frac{h_1}{h_2} \right) + W_3 \cdot \frac{s_2}{2} + W_4 \cdot \left(s_2 + \frac{s_2}{2} \cdot \frac{h_1}{h_2} \right) + \frac{P_F}{2} \cdot \frac{3s_1}{4} + \frac{P_F}{2} \cdot \frac{s_2}{4}}{(W_1 + W_3) \cdot \frac{h_1}{2} + (W_2 + W_4) \cdot \left(h_1 \cdot \frac{h_1}{h_2} + \frac{h_2}{2} \cdot \frac{h_1}{h_2} \right) + P_F \cdot h_1} + \frac{\frac{P_R}{2} \cdot \left(s_1 + \frac{s_1}{2} \cdot \frac{h_1}{h_2} \right) + \frac{P_R}{2} \cdot \left(s_2 + \frac{s_2}{2} \cdot \frac{h_1}{h_2} \right) + (\mu \cdot P_F) \cdot h_1}{(W_1 + W_3) \cdot \frac{h_1}{2} + (W_2 + W_4) \cdot \left(h_1 \cdot \frac{h_1}{h_2} + \frac{h_2}{2} \cdot \frac{h_1}{h_2} \right) + P_F \cdot h_1} \quad (11)$$

4. Case studies

4.1 Pescara del Tronto case study

Figure 8 shows a house in Pescara del Tronto completely collapsed after the 2016 Central Italy earthquake, which can be used as an example to apply the 1-storey model. The data that were used as input in the model are the following: wall thickness ($s = s_1 = s_2$) = 0.4 m; roof thickness (s_R) = 0.15 m; wall height (h_1) = 3 m; wall length (l_w) = 7 m; span length (L) = 5 m; distance between the center of mass of the roof and the top of the wall (h_R) = 0.4 m; specific weight of masonry (P_m) = 19 kN/m³; specific weight of reinforced concrete (P_c) = 21 kN/m³; friction coefficient roof – wall (μ) = 0.4; PGA for a return period of 475 years (a_g) = 2.489 m/s²; S_s = 1 (rock soil); S_t = 1.2 (top of a hill). The weight of the walls was calculated as $W_i = h_i \cdot l_w \cdot s_i \cdot P_m$, where $i=1, 2$, while the weight of the roof as

$$W_i = 2 \cdot \left[\sqrt{(s + L/2)^2 + h_r^2} \cdot l_w \cdot s_r \cdot P_c \right].$$



Figure 8. View of the building used as case study before and after the earthquake.

As already mentioned, most of the data are geometrical so they can be easily collected. Only the PGA needs to be calculated referring to the information given by the Italian code [5]. In this case, the location is fixed, so the value can be determined analytically. The value used for S_s is referred to rock soil, while the one used for S_t is referred to the top of a hill as there is evidence of a slope behind the building.

4.1.1 Pescara del Tronto results

The results using the linear kinematic analysis and verification are reported in Table 1.

Table 1. Collapse load multipliers and safety factors for the Pescara del Tronto case study.

	1-storey overturning without ring beam	1-storey overturning with ring beam
α_c	0.10	0.26
SF	0.57	1.48

Comparing the load multipliers, it can be observed that the first mechanism is more likely to happen as the 1-storey overturning with ring beam case has a collapse load 2.6 times higher than the 1-storey overturning without ring beam. This is confirmed by the fact that the safety factor in the case without ring beam is lower and it is below the safety threshold. Indeed, as shown in Figure 8 there is no connection between roof and walls. If this kind of analysis was carried out in a pre-earthquake situation, it would have been possible to demonstrate how the building was unsafe towards a seismic event with a return period of 475 years. The actual demand of 2016 earthquake was larger than the one used as input for this verification procedure. However, the aim of the method is to report if retrofitting is needed to improve the level of safety. Validation analyses were not performed at this stage, but adequate interventions could have likely prevented the structure from a complete collapse.

A sensitivity analysis was carried out to assess the impact of the chosen input parameters on results. Most relevant outcomes were then plotted in graphs with the varying parameter on the x axis and the safety factor SF on the y axis. Figure 9(a) shows that when there is no

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300 friction the two mechanisms are equivalent because of the symmetry of the systems and the
 301 configuration is unsafe. The minimum friction coefficient to ensure a safety factor greater
 302 than 1 is 0.2. As predictable, the impact of the friction is overall positive: it is enough to build
 303 a ring beam able to ensure a friction coefficient of 0.4 and triple the safety factor. In Figure
 304 9(b) the span length is varying, resulting in an increase or decrease in the weight of the roof.
 305 This has almost no effects in the case without ring beam, which remains unsafe in the whole
 306 range of variation. On the other hand, since the friction force is proportional to the weight of
 307 the roof, increasing the span length leads to a significant increment of the safety factor. When
 308 the masonry specific weight of the masonry varies, safety factor remains almost constant
 309 (Figure 9(c)). The weight of the walls has indeed a twofold effect since it contributes to both
 310 stabilizing and overturning forces. Also, looking at the equations that lead to determine SF (in
 311 particular Eqs. (6-7) and Eq. (2)), it can be noticed how the weight appears always at both
 312 nominator and denominator. Therefore, in the calculation steps its variation tends to have
 313 negligible effects. Figure 9(d) shows the influence of the wall thickness. In the first case,
 314 without ring beam, an increment of the wall thickness corresponds to a linear and
 315 considerable increment of the *SF*. However, almost 70 cm walls are required to be in the safe
 316 area. In the second case, there is an asymptotic trend. It means that for low thicknesses the
 317 presence of the ring beam is effective (e.g. for $s = 20$ cm the *SF* is two times the *SF* in the
 318 case with no ring beam), but for high thicknesses the structure is so massive that the presence
 319 of the ring beam at the top is irrelevant. Finally, in Figure 9(e) different soil categories are
 320 taken into account by varying the abovementioned parameter S_s . Following the guidelines
 321 provided by the Italian seismic codes, the possible values S_s can assume were calculated
 322 (Table 2). A rock soil allows to have higher safety factors, whereas if the quality gets worse
 323 there is a decreasing trend, except for category E (coarse soil upon a stiff or soft soil).

324
325 Table 2. Values of the parameter S_s for different soil categories.

Soil category	Description	S_s
A	Rock soil ($V_{s30} > 800$ m/s)	1.00
B	Soft rock and very dense soil (360 m/s $< V_{s30} < 800$ m/s)	1.16
C	Stiff soil (180 m/s $< V_{s30} < 360$ m/s)	1.33
D	Soft soil ($V_{s30} < 180$ m/s)	1.48
E	Coarse soil upon stiff or soft soil	1.33

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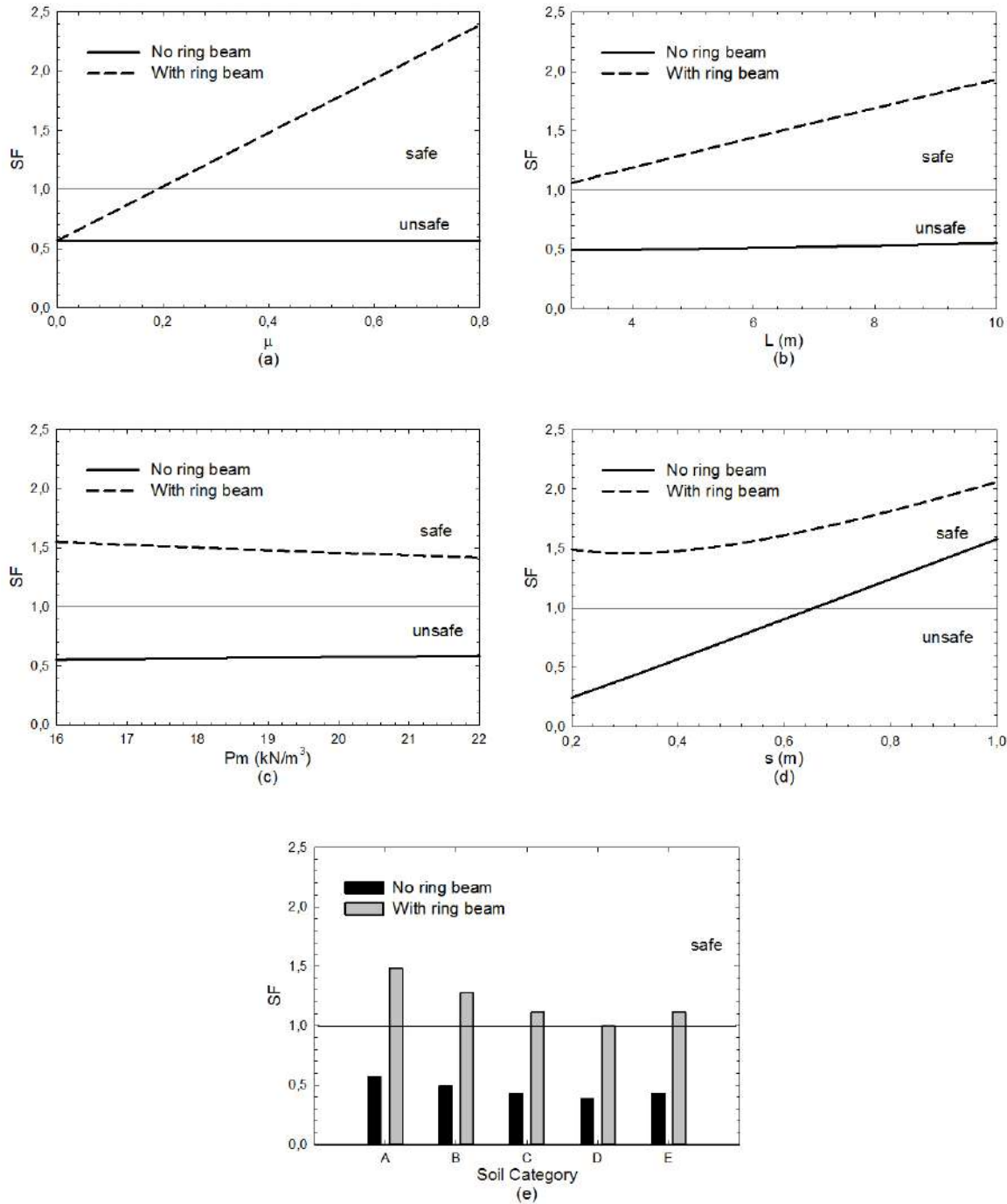


Figure 9. (a) Friction coefficient vs. safety factor, (b) span length vs. safety factor, (c) masonry specific weight vs. safety factor, (d) wall thickness vs. safety factor, (e) soil category vs. safety factor.

4.2 Amatrice case study

The 2-floor models were tested through the building in Figure 10 which was located in Amatrice. The data used as input for the analysis are the following: wall thickness ($s_1 = s_2$): 0.4 m; roof thickness (s_R): 0.15 m; wall height ($h_1 = h_2$): 3 m; wall length (l_w): 11 m; span length (L): 6.5 m; distance between the center of mass of the roof and the top of the wall (h_R):

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 770 340 0.4 m; specific weight of the masonry (P_m): 19 kN/m³; specific weight of the reinforced
 771 341 concrete (P_c): 21 kN/m³; floor weight (P_f): 4 kN/m²; friction coefficient roof – wall (μ): 0.4;
 772 342 friction coefficient floor – wall (μ): 0.5; PGA for a return period of 475 years (a_g): 2.538
 773 343 m/s²; S_s : 1 (rock soil); S_i : 1 (flat area).
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787 345 Figure 10. View of the building used as case study before and after the earthquake.
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 791 347 *4.2.1 Amatrice results*

792 348 The results obtained using the linear kinematic analysis are summarized in Table 3:
 793
 794 349

796 350 Table 3. Collapse load multipliers and safety factors for the Amatrice case study.
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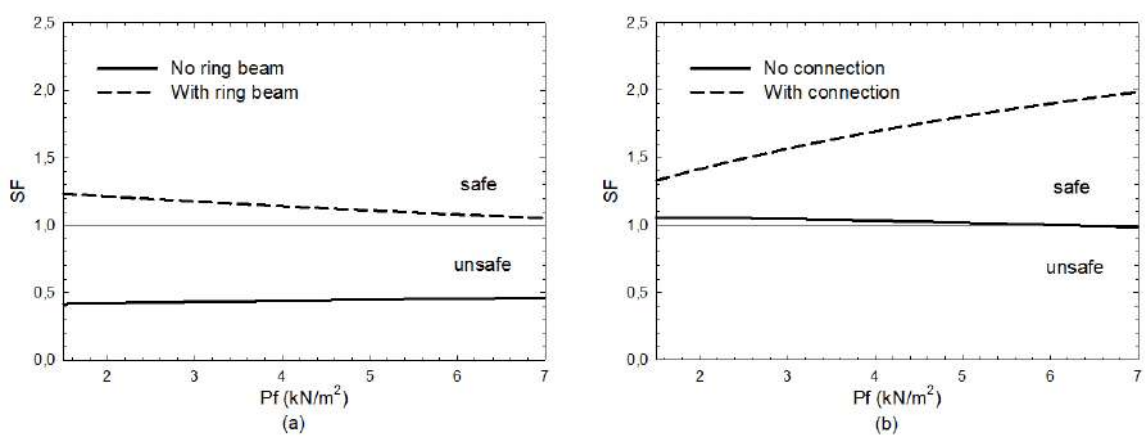
	2-storey overturning without ring beam	2-storey overturning with ring beam	2-storey vertical flexural behaviour without connection between floor and walls	2-storey vertical flexural behaviour with connection between floor and walls
α_C	0.06	0.16	0.16	0.27
SF	0.44	1.14	1.03	1.70

805 351
 807 352 Comparing these results, it is clear that the configuration of overturning with no ring beam is
 808 353 the less safe and the most probable. The presence of a ring beam increases the collapse load
 809 354 multiplier of about 2.7 times for the overturning mechanism. On the other hand, the model
 810 355 with connection at the top of the walls and at floor level is the safest and less probable case.
 811 356 For the vertical flexural behaviour, the case with connection between floor and walls has 1.7
 812 357 times as high of a collapse load as the case without connection has. Finally, it is possible to
 813 358 observe that the mechanisms of overturning with ring beam and vertical flexural behaviour
 814 359 without connection at floor level are almost equivalent.
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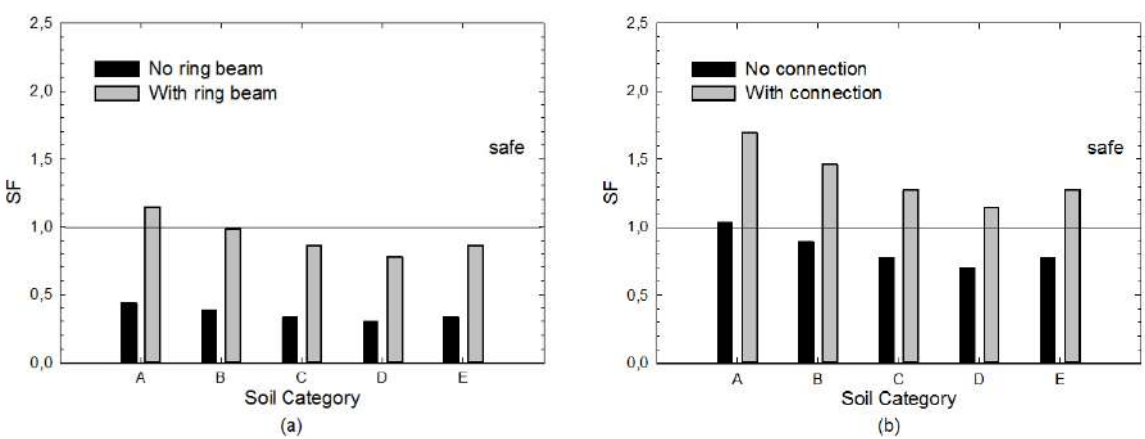
817 360 Also for this case study, sensitivity analyses were performed for all parameters. The trends of
 818 361 variation of other parameters are really similar to the previous ones, thus the considerations
 819 362 done for the 1-storey model are still valid. The substantial difference between 2-storey
 820 363 overturning and vertical flexural behaviour is that in the second case the safety factors are
 821 364 higher. Omitting obvious and well-known results, it is worth to comment what happens when
 822 365 the floor weight is changing (Figure 11). In both cases of overturning and in the case of
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366 vertical flexural behaviour without connection, when the floor weight varies the safety factor
 367 is not subject to significant changes. A slight reduction of SF can be observed in the
 368 overturning scenario for the model with ring beam as the floor weight increases. A heavier
 369 floor leads to greater horizontal inertial forces which contribute to the overturning. Instead,
 370 for the mechanism of vertical flexural behaviour with connection, a heavy floor has a positive
 371 impact since the stabilizing force increases. As far as the variation of soil category is
 372 concerned, the values of the S_s coefficient that were used in the previous case study are still
 373 valid (Table 2). It is interesting to notice that if the category of soil was not “A” (rock soil),
 374 then only the mechanism of vertical flexural behaviour with connection would be safe
 375 (Figure 12).



376
377 Figure 11. Floor weight vs. safety factor for the (a) overturning mechanism and the (b)
 378 vertical flexural behaviour.



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381 Figure 12. Soil category vs. safety factor for the (a) overturning mechanism and the (b)
 382 vertical flexural behaviour mechanism.

384 **5. Nonlinear kinematic analysis**

385 Nonlinear kinematic analyses were carried out with the aim of supporting the results obtained
 386 from linear analyses. The whole procedure follows the method proposed by Italian codes [5,

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387 6], which is presented hereafter just in its main steps. The underlying idea of the method is to
 388 determine the trend of the horizontal action that the structure is progressively able to
 389 withstand during the collapse mechanism's evolution. This can be seen as the capacity curve
 390 of an equivalent single degree of freedom system. The ultimate displacement capacity of the
 391 local mechanism is then defined and compared with the seismic demand. Similarly to the
 392 linear case, a multiplier α is introduced and defined as the ratio between the applied
 393 horizontal forces and the displacement d_k of a control point. The horizontal multiplier of
 394 loads is evaluated at various configurations of the kinematic chain until reaching the collapse
 395 condition, which is identified by a null multiplier α , and a displacement $d_{k,0}$. Assuming that
 396 involved actions (i.e. weights, external and internal forces) are constant during the evolution
 397 of the mechanism, the curve is almost linear. In this case, only the evaluation of the
 398 displacement $d_{k,0}$ is required, and the curve is described by Eq. (12):

$$\alpha = \alpha_0(1 - d_k / d_{k,0}) \quad (12)$$

400 where α_0 denotes the value of the multiplier capable of activating the analysed mechanism.
 401 The problem can be solved considering a configuration varied from the static condition and
 402 calculating the induced finite rotation θ_k by means of virtual work principle. Reaching the
 403 collapse situation, the overturning moment equals the stabilizing moment, and the resulting
 404 nonlinear equation gives the final rotation $\theta_{k,0}$. Once the latter is determined, the
 405 corresponding displacement $d_{k,0}$ can be obtained. Let the control point be the center of gravity
 406 of vertical forces, and h_{bar} be its distance from the base hinge. Eq. (13) expresses the relation
 407 between the rotation angle and the displacement of the control point related to ultimate
 408 capacity towards horizontal actions.

$$d_{k,0} = h_{bar} \sin \theta_{k,0} \quad (13)$$

410 At this point, it is possible to define the $\alpha - d_k$ curve according to Eq. (12). The equivalent
 411 capacity curve should now be determined. It describes the relation between the acceleration
 412 a^* that activates the mechanism and displacement d^* . The first is obtained through Eq. (2) as
 413 in the linear case, whereas d^* is given by Eq. (14).

$$d^* = d_{k,0} \frac{\sum_{i=1}^{n+m} P_i \delta_{x,i}^2}{\delta_{x,k} \sum_{i=1}^{n+m} P_i \delta_{x,i}} \quad (14)$$

415 where: $n+m$ is the number of weight forces that generates horizontal forces upon the macro-
 416 elements; P_i is the generic weight force; $\delta_{x,i}$ is the virtual horizontal displacement of the
 417 application point of P_i ; $\delta_{x,k}$ is the horizontal virtual displacement of the control point.

418 Making the same assumption done for the $\alpha - d_k$ curve, the capacity curve can be derived from
 419 Eq. (15).

$$a^* = a_0^*(1 - d^* / d_0^*) \quad (15)$$

421 where d_0^* is the equivalent displacement corresponding to $d_{k,0}$.

422 The verification for the life safety limit state consists in a comparison between the ultimate
 423 displacement capacity d_u^* of the local mechanism and the spectral displacement evaluated at
 424 the period T_s (Eq. (16)):

$$T_s = 2\pi \sqrt{\frac{d_s^*}{a_s^*}} \quad (16)$$

426 where a_s^* , is the acceleration correspondent to the displacement d_s^* , which is equal to $0.4d_u^*$,
 427 and $d_u^* = 0.4d_0^*$. Therefore, if d_u^* is greater than the spectral displacement $S_{De}(T_s)$, the
 428 verification is fulfilled (Eq. (17)).

$$SF = \frac{d_u^*}{S_{De}(T_s)} \geq 1 \quad (17)$$

430 Table 4 shows obtained results for all analysed models. As expected, a lack of connection is
 431 responsible of a small safety factor, always around 1 for the three studied cases. On the other
 432 hand, the presence of a ring beam or floor-wall connections ensures much more capacity to
 433 withstand larger ultimate displacements, which also means a higher safety level. The
 434 difference is less exaggerated in the case of vertical flexural behaviour since the connection
 435 between the floor and the wall makes the structure more rigid.

436 Comparing the linear with the nonlinear kinematic analysis in terms of safety factors, the bar
 437 chart of Figure 13 can be plotted. The nonlinear kinematic analysis usually provides safety
 438 coefficients that are two times or more the coefficient obtained with the linear analysis. This
 439 confirms that a fast and preliminary analysis, such as the linear one, provides precautionary
 440 results and it is recommended to understand if further investigations are needed. The
 441 described trend is inverted in the cases of vertical flexural behaviour, although the nonlinear
 442 safety factors are just slightly lower than linear ones. This phenomenon is due to the fact that
 443 the weight of a reinforced concrete roof has a particularly positive effect for these two
 444 models. In addition, the same analyses were repeated several times increasing the weight of
 445 the roof for each iteration. In all cases the safety factor increased, but the increment in the
 446 linear analysis was greater than in the nonlinear one.

447 Table 4. Results of the nonlinear kinematic analysis.

	Pescara del Tronto		Amatrice			
	1-storey overturning without ring beam	1-storey overturning with ring beam	2-storey overturning without ring beam	2-storey overturning with ring beam	2-storey flexural behaviour without connection	2-storey flexural behaviour with connection
d_u^* [m]	0.089	0.227	0.093	0.326	0.041	0.074
SF	1.04	2.66	1.02	3.02	1.12	1.95

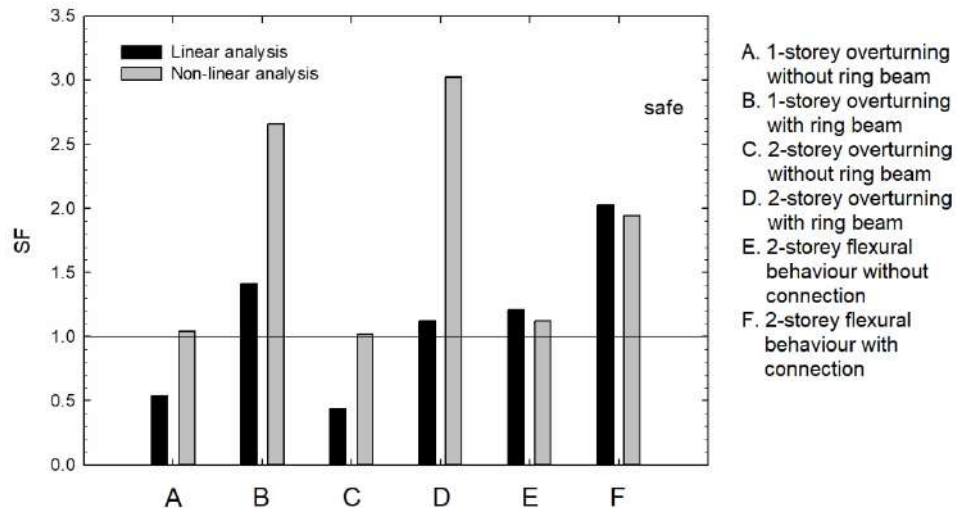


Figure 13. Comparison between safety factors resulting from linear and non-linear analyses.

6. Conclusions

The 2016 Central Italy earthquake caused damages and collapses of many masonry buildings, including those retrofitted with reinforced concrete roofs. This type of retrofit intervention seems to facilitate some collapse mechanisms if not properly executed. The paper presents a simplified procedure to evaluate the seismic performance of masonry buildings retrofitted with reinforced concrete roofs based on the linear kinematic analysis. Despite the analysis method is well known, the proposed models have the advantage of explicitly considering the interaction between the roof and the walls. Moreover, they require only few input parameters that can be easily obtainable. The collapse load multiplier obtained as output of the linear kinematic analysis was used to define a safety factor to have an idea whether a structure can be considered safe towards a certain collapse mechanism.

The introduced analytical models are based on the overturning and vertical flexural behaviour collapse mechanisms. In particular, the overturning scenario was analysed both for a 1-storey and a 2-storey building, while the vertical flexural behaviour was considered for a 2-storey building. Each scenario is studied twice: one configuration assuming that there is no connection among the involved macro-elements and another one that provides for an effective connection. The resulting six models were applied to two case studies, a 1-storey and a 2-storey building both collapsed during the 2016 Central Italy earthquake. Results highlight the importance of an efficient connection between the reinforced concrete roof and masonry walls and that some configurations are unsafe, which means additional investigations and possibly retrofit intervention are required. To account for the uncertainties introduced by the simplified models, sensitivity analyses were performed. These allowed to evaluate the influence of each input parameter to the overall safety level of the structure, highlighting peculiar aspects in each mechanism. Nonlinear kinematic analyses were also carried out and results were compared to those obtained from linear analyses. It was possible to observe that the linear analysis is faster and more precautionary as it provides smaller safety factors with respect to the nonlinear one.

Overall, the proposed analytical models represent an effective yet simple tool that can serve as a preliminary evaluation of the safety level of masonry structures retrofitted with reinforced concrete roofs. The method is not meant to be an alternative to other more refined analyses, such as finite element methods, that would allow for a more accurate safety verification. However, due to its simplicity, the procedure could be recommended when only few input data are available and be applied even by non-professional users to understand if further investigations and/or retrofit interventions are needed.

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