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**RELIABILITY ASSESSMENT OF ROBUSTNESS FOR REINFORCED CONCRETE
MOMENT RESISTING FRAMES**

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ABSTRACT

A strain-based 5-step procedure is illustrated for the probabilistic robustness assessment of 2D reinforced concrete moment resisting frames. Considering the central column removal as failure scenario, this study compares three frames, including a code-conforming design (*frame 1*) and two enhanced designs (*frame 2* and *frame 3*). For the three frames, non-linear finite-element (NLFE) models have been defined including properly calibrated translational springs. By sampling from materials and loads, 300 displacement-controlled pushdown NLFE global analyses have been conducted to determine energy-based dynamic amplification factors (1.02 - 1.75). Next, 300 probabilistic static-equivalent NLFE global analyses, performed by removing the column and amplifying loads, permitted to monitor the aleatory peak strains in any material and point of the frames. This allowed to compute the failure probabilities with respect to the ultimate limit state. Robustness improvements lead to much lower failure probabilities (10^{-7} - 10^{-3}) in any element with a reduction of damage propagation.

Keywords: structural robustness; reliability assessment; reinforced concrete moment resisting frames; strain-based 5-step procedure; probabilistic static-equivalent NLFE global analyses; energy-based dynamic amplification factor.

1. INTRODUCTION

Terrible events like the terrorist attack of 2001, which caused the total collapse of the so-called Twin Towers and thousands of victims, represented a shock for the entire community, not only for the consequences due to the structural collapse of the buildings, but mostly for human losses. In the past decades, such catastrophic phenomena on strategic structures, caused by exceptional events, brought architecture, engineering and construction experts to show a growing interest towards structural robustness, especially with respect to more sustainable systems [1]-[3] as well as optimized and reliable solutions [4]-[6]. European code rules [7]-[11] have been integrated by specific sections regarding structural robustness with suggestions or general requirements. The concept of extreme actions and their consequences on a structure have been deepened by many international guidelines [12]-[17]. In this way, it has been more and more clear that risk analysis should become part of strategies for collapse prevention against low-probability high-consequence (LPHC) events to investigate both socially acceptable and technically feasible solutions [18]. In this context, effects of aging on structure lifetime [19] as well as extraordinary events of natural or anthropic origin, like

73 explosions, avalanches or terroristic attacks, can determine critical conditions for structural systems.
74 The local collapse of a bearing element can trigger a chain reaction of structural element failures,
75 eventually leading to partial or full structural collapse as well described in [20]-[22]. A particular
76 situation, which has been deeply analysed by many authors [23]-[26], is the progressive collapse due
77 to a sudden column loss. The study of such problem is of interest to understand the entity of the
78 resistance reserves provided by the membrane and catenary effects [27]. Previous studies have
79 predominantly concentrated on the theoretical quantification of structural robustness. For instance,
80 [28] proposed a simplified tying force method, specifically designed to assess the progressive collapse
81 performance of reinforced concrete (RC) systems. Analytical models have been developed to evaluate
82 the ultimate bearing capacity of RC slabs in scenarios involving column loss, both for one-way and
83 two-way RC slabs [29] as well as RC flat slabs [30]. The study in [31] introduces an analytical model
84 highlighting the role of catenary forces under extreme lateral loading, focusing on cases where
85 damage is localized in a single structural member without complete column loss.
86 Another group of studies focuses on quantitative risk analysis. In fact, the level of safety associated
87 to LPHC events can be reliably investigated through quantitative risk analysis in probabilistic terms,
88 since it allows to include the uncertainties affecting engineering issues [32]. For example, a sensitivity
89 analysis is elaborated in [33] to compute the bearing capacity of different RC structural members in
90 case of the removal of a central supporting element. Fragility analyses are performed in [34] in case
91 of low-rise RC buildings to compute the exceedance probability of different damage states given a
92 column loss scenario. Global variance-based sensitivity analysis is used in [35] to study the major
93 sources of uncertainties in the response of RC structures subjected to sudden column loss. A
94 reliability-based index of structural collapse in case of extreme events is computed in [36], for 2D
95 linear elastic truss systems by randomly sampling loads and strengths. A probabilistic analysis of
96 steel-concrete composite floor against progressive collapse considering uncertainties of strengths and
97 loads in steel connections is performed by [37], concluding that using load combinations from
98 "General Services Administration - GSA" [20] may lead to a non-conservative design. The reliability
99 of RC frames under different column-loss scenarios is investigated in [38], obtaining failure
100 probabilities ranging from 0.016 to 0.137 and identifying side column-loss scenario as the worst case
101 if the presence of infill walls is not considered. The work [39] proposes a multilevel scheme to
102 evaluate the reliability of planar RC frames obtaining system failure probabilities of 0.254 and 0.173
103 for inner frames considering, respectively, 6-story and 3-story office buildings.
104 The actual code rules do not provide any requirement of a progressive collapse risk assessment as
105 well as associated reliability levels for buildings or bridges. Considering a reference period of 1 year
106 for a structural system, values higher than 1 for the reliability index are recommended in [14].
107 This study describes a strain-based 5-step procedure for the probabilistic robustness assessment of
108 2D RC moment resisting (MR) frames designed in seismic area. Considering a progressive collapse
109 scenario involving the loss of a central supporting column, three different frames are analysed: one
110 designed according to seismic codes (i.e., *frame 1*) and two other ones (i.e., *frame 2* and *frame 3*)
111 where the reinforcement arrangement is properly modified to achieve a more robust behaviour in
112 terms of both bearing capacity and catenary effect, according to [40]. In detail, the *frame 2* adopts a
113 continuous reinforcement along the beams over the supports with two additional levels of side face
114 rebars, while the *frame 3* derives from the *frame 2* adopting the same reinforcement amount in all the
115 floors with symmetric cross-sections to exploit a global Vierendeel behaviour. For the three entire
116 frames, non-linear finite element (NLFE) global models have been defined in ATENA 2D including
117 the contribution of the orthogonal structural frames by means of equivalent translational springs.
118 Within a full probabilistic approach, for each frame, 100 random combinations have been sampled
119 from the aleatory properties of both materials and applied loads through the Latin Hypercube
120 Sampling (LHS) technique [41]. The aleatory properties denote the variables characterised by
121 randomness. Specifically, eleven different random variables have been sampled regarding the
122 concrete properties, reinforcing steel properties and concerning both permanent and variable loads
123 according to [7]-[10],[42]-[46]. After that, three sets of 100 displacement-controlled pushdown NLFE

124 global analyses have been performed, by applying an increasing vertical displacement where the
125 central supporting column had been removed. From the resulting capacity curves (i.e., displacement-
126 reaction curves), the energetic equivalence approach [23] has been applied to compute the
127 corresponding dynamic amplification factors (DAFs) for each one of the 300 sampled frames. These
128 DAFs have been useful to perform 300 probabilistic static-equivalent NLFE global analyses, by
129 removing the central supporting column and amplifying the loads on the spans close to the removed
130 column. Successively, the peak strains monitored in both confined concrete and reinforcement in
131 different points of the frames have been probabilistically modelled. By convolution integrals between
132 the strain distributions with the corresponding ultimate threshold distributions, the failure
133 probabilities with respect to the ultimate limit state (ULS) have been estimated in any structural
134 element of the three RC MR frames. The comparison between the three RC MR frames provides
135 reference values of the failure probabilities and shows the relevant safety advantages when robustness
136 improvements are adopted: the failure probabilities are strongly lower in the spans close to the
137 collapse scenario, the safety level increases in any structural element for increasing distance from the
138 collapse event highly reducing the damage propagation.

139 2. RELIABILITY ASSESSMENT OF STRUCTURAL ROBUSTNESS: PROPOSED 140 FRAMEWORK

141 Structural reliability means the capability of a structure or a structural member to fulfil the specified
142 requirements during the reference lifetime, for which it has been designed [47]. The basic
143 requirements are intended in terms of safety, serviceability, durability and robustness [11]. In order
144 to quantify these requirements, the so-called limit states (or performance functions) are identified as
145 the conditions beyond which the structure, or a part of it, does no longer satisfy one of its performance
146 requirements [7],[11]. Reliability analysis represents a powerful method to deal with uncertainties in
147 engineering design [48]. These uncertainties differ for many aspects (e.g., the nature of the structure
148 itself, environmental conditions and applied actions) and can be distinguished between aleatory,
149 which are related to the inherent randomness of loads and of structural properties (i.e., mechanical
150 and geometrical), and epistemic, connected to the lack of knowledge and consequential assumptions
151 to model the variables [49]. The measure of the reliability is the failure probability P_f with respect to
152 the limit state function or the reliability index β [7],[11],[50]-[51]. The methods to assess the
153 reliability can be classified in four levels [11]: full probabilistic methods (i.e., level III); probabilistic
154 methods (i.e., level II) (e.g., FOSM (first order second moment) or FORM (first order reliability
155 methods)); semi-probabilistic methods (i.e., level I); deterministic methods (i.e., level 0). In this work,
156 the full-probabilistic method has been adopted together with the Latin Hypercube Sampling (LHS)
157 technique [41], which represents a stratified sampling technique employed to generate samples of
158 random variables with an even distribution over the corresponding domains.

159 Structural robustness is one of the performance requirements that a structure should guarantee when
160 dealing with structural reliability. In general, it can be defined as the capability of a structure or a
161 structural member to avoid disproportionate damage, with respect to an exceptional action that has
162 caused the damage. The structural robustness can be assessed by means of a probabilistic risk analysis
163 (PRA), based on the concept of conditional probability [14]. Considering H as the hazard, connected
164 thus to the dangerous event, and considering LD as the local damage, the probability of structural
165 collapse $P[C]$ can be computed [14] as follows:

$$P[C] = P[C | LD] \cdot P[LD | H] \cdot P[H] \quad (1)$$

166 where $P[C|LD]$ is the conditional probability of disproportionate collapse, given the local damage
167 LD , $P[LD|H]$ denotes the conditional probability of local damage given the event H and $P[H]$
168 represents the occurrence probability of the event depending on its mean annual occurrence rate λ_H .
169 In the hypothesis to investigate the robustness of a frame after an accidental event which causes the
170 sudden removal of a supporting column, the reliability assessment can be focused on the computation
171 of the term $P[C|LD]$, considering that the local damage LD occurs (i.e., $P[LD|H]$ is equal to unity) if

172 there is the happening of the dangerous event H (e.g., gas explosion, terrorist attacks, failure at the
 173 foundation level, excessive loads, etc.) characterised by its specific mean annual rate λ_H [14],[52].
 174 For the reliability assessment of structural robustness of a frame within the full probabilistic approach,
 175 the following strain-based 5-step procedure is followed (Fig. 1), inspired by [23],[41],[53]-[54]:
 176 1. The aleatory properties of both structure (e.g., materials and geometry) and loads have to be
 177 sampled (e.g., through LHS method) defining N_{tot} aleatory structures (as detailed in Section 5).
 178 2. A preliminary analysis is needed by performing displacement-controlled pushdown NLFE
 179 global simulations, where any aleatory structure is modelled without the supporting column and
 180 a progressively increasing displacement is imposed at the point where the column is removed.
 181 From these simulations, the reaction at the same point of the removed column is monitored to
 182 define the probabilistic capacity curves (i.e., displacement-reaction curves). Additional details
 183 are given in Section 6.
 184 3. The computation of the DAFs: the dynamic amplification factor represents the increase of the
 185 structural response to a static load to reproduce the dynamic application of the same load. The
 186 energy equivalence approach can be followed, according to [23], to find the energetic
 187 equilibrium corresponding to the equivalence between the work done by the external load and
 188 the internal energy given by the structure. The approach is adopted to compute the dynamic
 189 displacement and corresponding dynamic gravitational load P_d on the probabilistic capacity
 190 curves. In this way, the DAFs can be evaluated as the ratios between the dynamic and static
 191 gravitational loads (as further described in Section 6).
 192 4. Probabilistic static-equivalent NLFE global analyses have to be performed simulating the
 193 removal of the column and amplifying the loads of the adjacent spans in each floor through the
 194 energy-based DAFs, while the other spans are loaded with the non-amplified gravity loads.
 195 Additional features are given in Subsection 7.1.
 196 5. The aleatory results in terms of peak strains for the different materials have to be treated in
 197 probabilistic terms to assess the reliability at structural level with respect to the ultimate limit
 198 state (ULS), given by the onset of the ultimate strains of the materials. An in-depth description
 199 is reported in Subsection 7.2.
 200

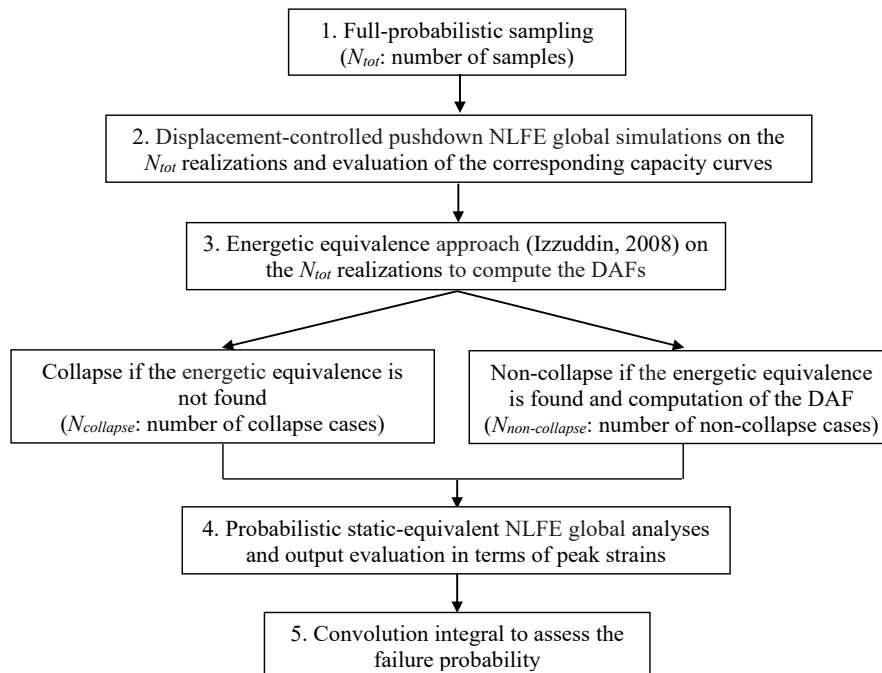


Fig. 1 Flowchart of the 5-step strain-based procedure.

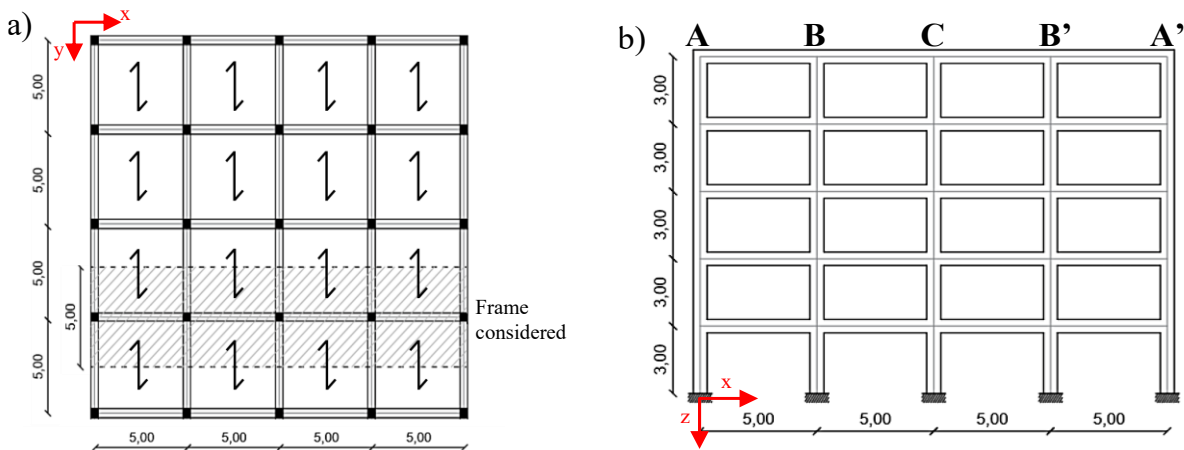
201
 202 This procedure ensures to take into account, in a simplified way, the dynamicity of such failure
 203 scenario avoiding the complexity of performing dynamic analyses but, at the same time, allowing to

204 consider both geometrical and material non-linearities within non-linear simulations of the entire
 205 structure [55]. However, it should be noted that other effects, due to dynamic features which can lead
 206 to instability phenomena, are not included.

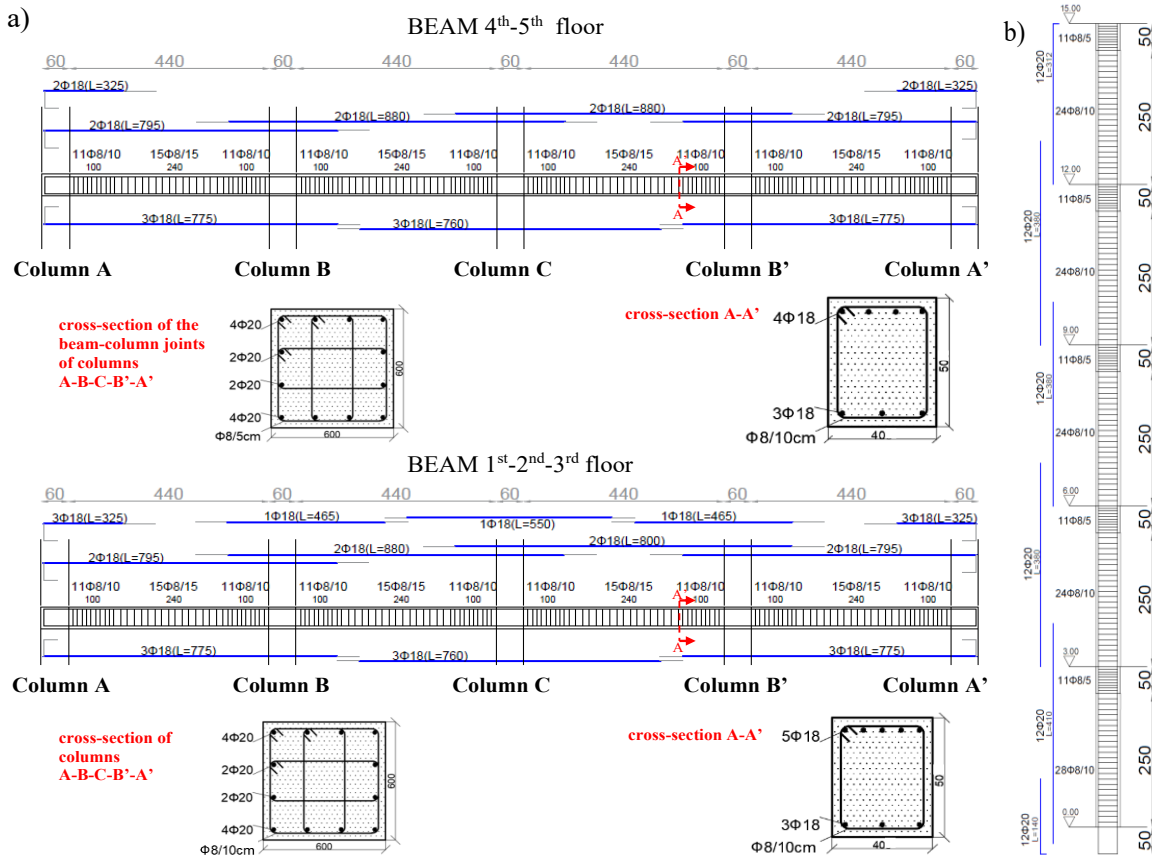
207 3. DESIGN OF 2D RC MR FRAMES IN SEISMIC ZONE WITH DIFFERENT
 208 REINFORCEMENT LAYOUTS

209 3.1 Seismic design according to current standards: "frame 1"

210 The starting point of this work is to design a 2D multi-storey reinforced concrete (RC) moment
 211 resisting (MR) frame, regular in elevation and symmetric, consisting of four floors plus roof with four
 212 spans.
 213



214 Fig. 2 Geometry of the multi-storey RC MR frame: (a) plan view with joists framework direction; (b) lateral view.
 215 Measurements in m.



216 Fig. 3 Longitudinal and transverse reinforcement arrangement for: (a) beams with specific cross-sections and their
 217 positions; (b) columns and joints. Measurements in cm.

218 The inter-storey height is equal to 3m for all the floors, the length of all the spans is equal to 5m and
219 the transverse effective width is equal to 5m (Fig. 2). The structure is located in L'Aquila city (Italy)
220 and a high ductility class is assumed according to [9],[57].

221 As for the materials, the reinforcing bars are made of B450C steel [9]-[10], while C25/30 concrete
222 [9]-[10] with 3.5cm of clear concrete cover is used in any structural element.

223 Static and modal analyses have been carried out to design the geometry together with the longitudinal
224 and transverse reinforcement with respect to ULSs, serviceability limit states (SLSs) and the capacity
225 design principles for seismic verifications in all the structural elements [8]-[10],[57]. Fig. 3 illustrates
226 the reinforcement detailing obtained from the verifications: in particular, $\phi 18$ longitudinal bars and
227 $\phi 8$ stirrups are adopted, for all the structural elements.

228 All the beams have dimensions of $40 \times 50 \text{ cm}^2$. The transversal reinforcement in the beams is made of
229 2-legs $\phi 8$ stirrups arranged with a spacing of 10cm in the dissipative zone and 15cm in the central
230 zones of the spans, for all the floors. The transversal and longitudinal reinforcement of the beams are
231 depicted in Fig. 3(a). The dissipative zone has a length of 100cm from the beam-column joint.

232 Regarding the columns, they all have $60 \times 60 \text{ cm}^2$ cross-sections. The longitudinal reinforcement is
233 made of $12 \phi 20$ bars (located symmetrically in both directions), while the shear reinforcement is made
234 of 4-legs $\phi 8$ stirrups with a spacing of 10cm, along the entire length of the structural element apart
235 from the beam-column joints where the spacing is equal to 5cm. The transversal and longitudinal
236 reinforcement of both the columns and joints are depicted in Fig. 3(a)-(b).

237

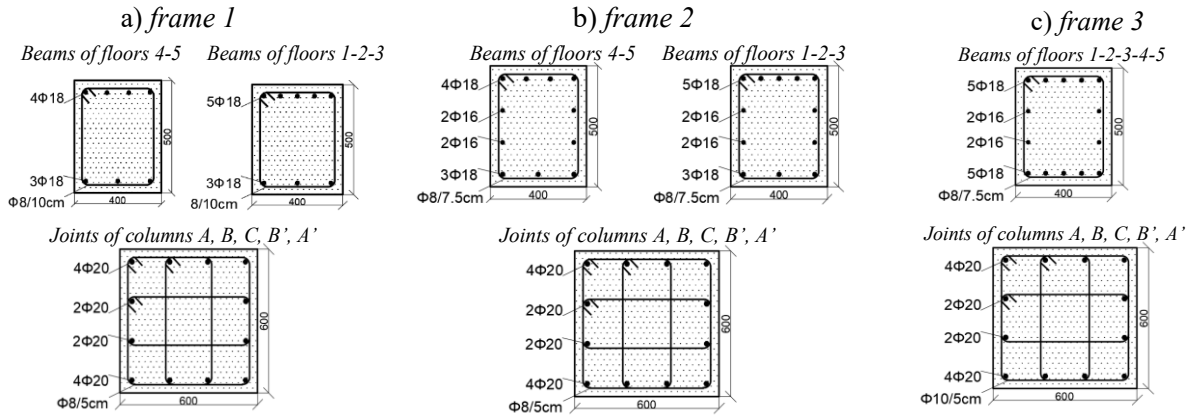
238 3.2 Seismic design according to robustness improvements: "frame 2" and "frame 3"

239 Other two frames have been designed with proper modifications in the longitudinal reinforcement
240 arrangement to improve the response against a column removal scenario, adopting the criteria
241 combined with the cyclic procedure proposed and investigated in [40].

242 The *frame 2* is characterised by the following assumptions: all the longitudinal reinforcement bars
243 are displaced continuously along the beams, respecting the suggestions to ensure the continuity over
244 the supports and for at least 30% of the span length in addition to the anchorage length, according to
245 [40] and [12]-[15]. This derives from the major extension of the bending moment in the upper chord
246 of the beams due to the column loss. It follows that the new arrangement of the longitudinal
247 reinforcement is made of $3 \phi 18$ in the lower chord of all the beams of the five floors, while $5 \phi 18$ in
248 the upper chord for the beams in the first three floors and $4 \phi 18$ for the beams in the last two floors.
249 In addition, two levels of $2 \phi 16$ side face rebars (placed at around $1/3$ and $2/3$ of the beam height) are
250 arranged continuously over all the beams for all the floors. The presence of the side face
251 reinforcement bars can anticipate the catenary effect activation because they can provide an important
252 contribution to a tension response, facilitating tie behavior [40]. It should be noted that the change of
253 the longitudinal reinforcement arrangement within the beams needed to be verified at the ULSs and
254 seismic verifications. Thus, the stirrup step in the dissipative area for the beams in all the floors was
255 modified from 10cm to 7.5cm. The other structural elements remained unchanged with respect to the
256 *frame 1* (Fig. 4(a)). This new arrangement of the *frame 2* is depicted in Fig. 4(b).

257 The third frame (i.e., *frame 3*) derives from the *frame 2* by applying a symmetric reinforcement
258 between the upper and lower chord of the beams and arranging the same amount of reinforcement in
259 the beams of the first floor (i.e., the largest one) to the beams of all the other floors. The symmetry
260 criterion, as also suggested by [34],[40], is considered because when a loss of a load-bearing column
261 occurs, some sections undergo to opposite bending moments in sign. Instead, the equal reinforcement
262 criterion is followed to take advantage of the Vierendeel effect: in fact, due to the reduced axial
263 extensibility of the columns, all floors can be assumed subjected to the same displacement in
264 correspondence of the removed column and, therefore, to the same internal actions. However, it
265 should be recognised that this arrangement contradicts optimization principles [4] exclusively in
266 favour of robustness and safety criteria. By applying these two additional criteria, $5 \phi 18$ reinforcing
267 bars are arranged continuously and symmetrically in all the beams of the five floors. To respect the
268 capacity design principles, the stirrup step becomes 7.5cm in the dissipative areas of the beams for

269 all the floors, whereas the stirrup diameter of all the beam-column joints increased to $\phi 10$ with the
 270 same step of 5cm. The other structural elements remained unchanged with respect to the *frame 1* (Fig.
 271 4(a)). This new arrangement of the *frame 3* is depicted in Fig. 4(c).
 272 It is important to underline that all the modifications in the arrangement of the reinforcement are
 273 verified according to the seismic codes [9],[57] and respect all the code limitations.
 274



275 Fig. 4 Longitudinal and transverse reinforcement in the cross-sections of the beams of Fig. 3(a) and in the beam-column
 276 joints for: (a) *frame 1*; (b) *frame 2*; (c) *frame 3*. The dimensions are in millimetres if not indicated.

277 **4. FE MODELS OF THE 2D RC MR FRAMES WITH THE COLLAPSE SCENARIO**

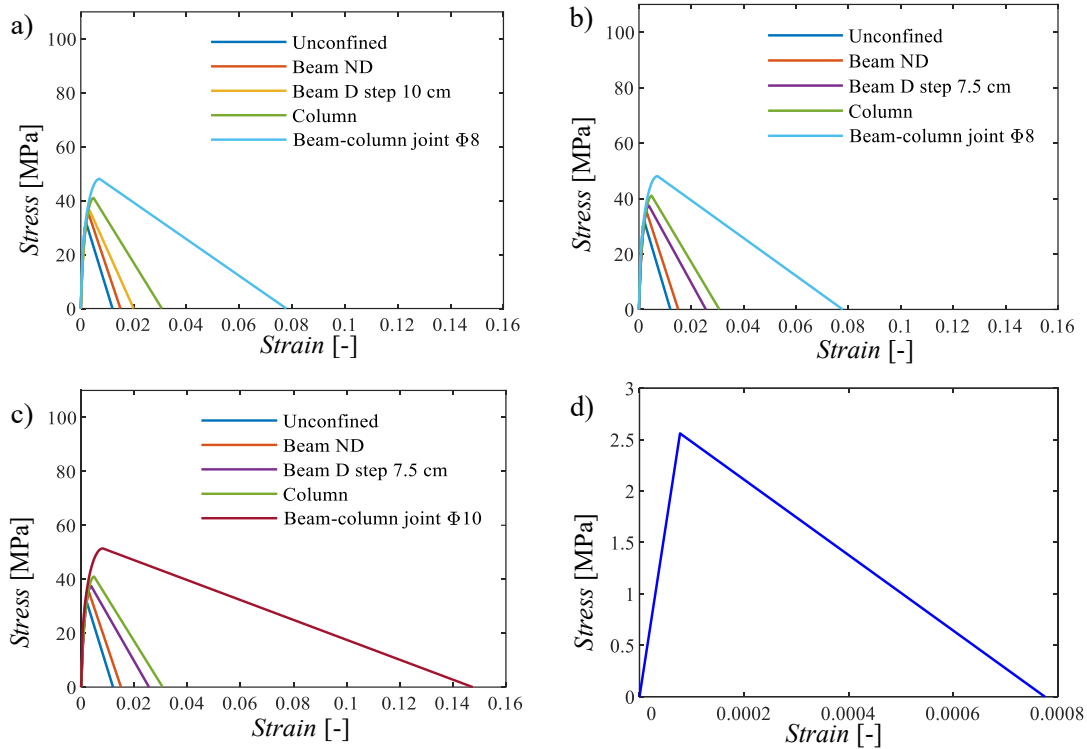
278 The FE models of the three entire frames have been defined adopting the FEM software ATENA 2D
 279 [56]. All the assumptions (i.e., typology and size of the mesh, mesh elements, constitutive laws of
 280 materials, number of iterations, tolerances, boundary conditions) [58]-[67] behind the numerical
 281 modelling have been validated by reproducing the results in terms of load-deflection curve and
 282 horizontal-vertical displacement curve of the experimental test in [68], on a beam-column
 283 subassembly with similar characteristics of the frame under study, as explained in Appendix A. The
 284 only differences concern the material properties and geometrical characteristics, while the remaining
 285 assumptions and modelling strategies are the same, as summarized in Table 1.

286 Table 1. Modelling assumptions explained in Appendix A.

<i>Equilibrium of forces</i>	<ul style="list-style-type: none"> - System of equations solved using the standard Newton-Raphson iterative procedure with “line search” - Max iteration limit: 2500 - Convergence criteria: 1.0% for the norm of displacement error; from 1.0% to 2.5% for the norm of residual force error; from 1.0% to 2.5% for the maximum error of residual forces; 1.0% for the out-of-balance energy error
<i>Kinematic compatibility</i>	<ul style="list-style-type: none"> - Macro-elements modeled as plane stress quadrilateral isoparametric finite elements, based on linear polynomial interpolation, with 4 Gauss integration points - Mesh size ranging between 5 and 10 cm. - Longitudinal and transverse reinforcement modelled by discrete approach
<i>Constitutive relationships</i>	<ul style="list-style-type: none"> - <i>Concrete compression behaviour</i>: curvilinear response with linear compression softening (LCS) - <i>Concrete tensile behaviour</i>: bi-linear law with elastic response until tensile strength and post peak linear tension softening (LTS), calibrated for ultimate strain at zero stress equal to 10 times the strain of the peak tensile strength - <i>Cracking</i>: smeared crack modelling with fixed crack model - <i>Reinforcement</i>: bilinear constitutive model with hardening law for both compression and tension

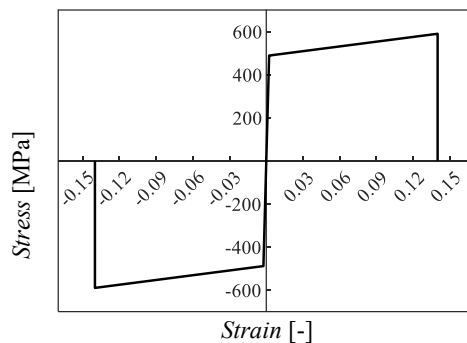
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 288 The compressive behaviour of concrete is shown in Fig. 5(a)-(b)-(c), respectively, for the *frame 1*,
 289 *frame 2* and *frame 3* considering the mean values of the mechanical properties, while the tensile
 290 behaviour is shown in Fig. 5(d). As it can be observed, the constitutive model in compression has

291 been derived for: unconfined concrete (i.e., effective concrete cover - “unconfined”), confined
 292 concrete in non-dissipative areas of beams (“beam ND”), confined concrete in dissipative area of
 293 beams (“beam D” considering the step of the stirrups), confined concrete in columns, confined
 294 concrete in beam-column joints (considering the stirrup diameter). The adoption of a constitutive law
 295 accounting for confinement effects also in beams is extensively justified in Appendix A and derives
 296 from the arching compressive forces that arise in the beams during the initial stages (i.e., flexural and
 297 softening response [40]) of the progressive collapse phenomena.
 298



299 Fig. 5 Constitutive laws: concrete compressive behaviour, respectively, for the *frame 1* (a) *frame 2* (b) and *frame 3* (c);
 300 (d) concrete tensile behaviour for all the frames and elements. Mean values of mechanical properties are assumed.

301 The compressive and tensile behaviour of steel is shown in Fig. 6 considering the mean values of the
 302 mechanical properties.
 303

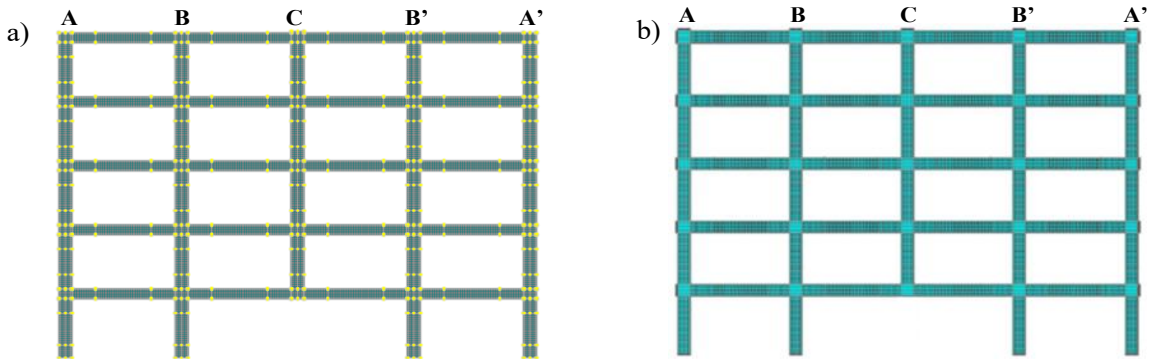


304 Fig. 6 Constitutive law for the steel in tension and in compression for all the frames and for both stirrups and
 305 longitudinal reinforcement, assuming mean values of mechanical properties.

306 More details about the statistical values of the mechanical properties for both the materials are
 307 clarified in the following section.

308 The boundary conditions in the FE models are defined as fully restraints at the base of the columns.
 309 As an example, the characteristics of the FE models in terms of joints, lines, macro-elements and
 310 reinforcement discrete elements are shown in Fig. 7 for the *frame 1*, depicting also the failure
 311 scenario.

312 The failure scenario of the central supporting column belonging to the internal frame (Fig. 2(a)) has
 313 been selected since it is not influenced by the infills located along the orthogonal perimeter frames.
 314



315 Fig. 7 Representation of the 2D entire frame in the FEM software, for example, regarding the *frame 1*: (a) joints, lines,
 316 macro-elements and mesh; (b) longitudinal and transverse reinforcement.

317
 318 **4.1 Contribution of the orthogonal framed systems**

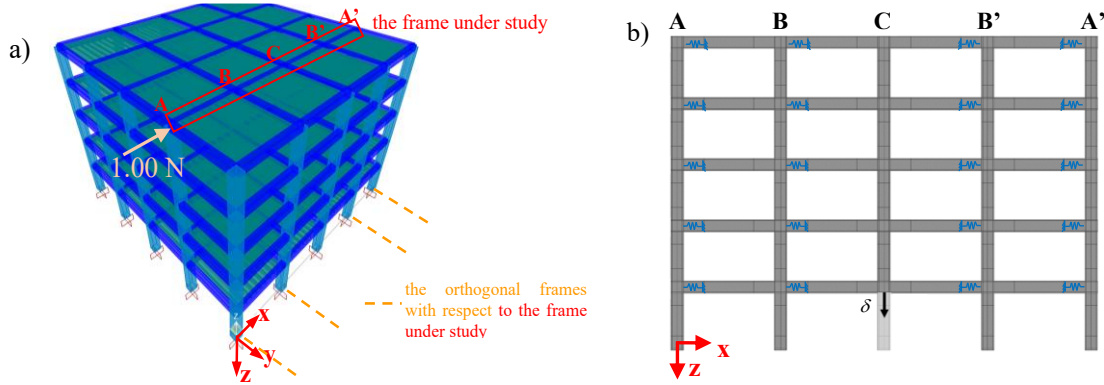
319 With the aim to include 3D effects in the 2D NLFE global analyses, the contribution of the orthogonal
 320 systems to the frames under study is modelled, in a simplified way, by means of equivalent elastic
 321 springs. In fact, in a 3D structure, the constraints due to the transverse frames (i.e., in the y-z plane of
 322 Fig. 2) surely enhance the robustness by stiffening the frame in the x-z plane [27],[69]-[74]. The
 323 transverse beams belonging to the orthogonal frames and slabs with one-way joists, have been herein
 324 modelled, in a straightforward manner, by means of equivalent elastic translational springs. The
 325 rotational stiffness has not been considered [69],[75] since it has a negligible influence on the flexural
 326 response without any influence on the development of catenary action. The stiffnesses of these
 327 translational elastic springs are properly calibrated for each joint of the five floors (apart from the
 328 ones of the column involved in the failure scenario) through elastic analyses in SAP2000 [76]
 329 conducted on the 3D structure, in absence of the central supporting column (Fig. 8(a)). Specifically,
 330 the elastic axial (diaphragm) contribution of the one-way RC slabs having thickness of 4cm [9]-[10]
 331 with the one-way joists of 20cm is modelled. The orthogonal frames (in the y-z plane) are assumed
 332 to have the same geometrical and mechanical characteristics of the frame under study (in x-z plane).
 333 In addition, the axial and bending stiffness of beams and the bending stiffness of the columns of the
 334 frame under study (in x-z plane) are nullified since their contributions are already accounted for in
 335 the FE model, while the torsional and bending stiffness of all other beams and the bending stiffness
 336 of the other columns are reduced according to [77]-[78] to consider the cracked behaviour in a
 337 simplified way.

338 Table 2. Equivalent elastic springs: stiffness for each joint of the different floors.

	Spring Stiffness for column A and A' [N/m]	Spring Stiffness for column B and B' [N/m]
1st floor	9.775E+07	9.634E+07
2nd floor	7.143E+07	7.062E+07
3rd floor	5.291E+07	5.247E+07
4th floor	4.122E+07	4.095E+07
5th floor	3.291E+07	3.270E+07

339
 340 To compute the stiffness at each joint, a unitary horizontal force in the out-of-plane direction of the
 341 orthogonal frame (i.e., x direction) is applied to that joint (Fig. 8(a)). The unitary horizontal force
 342 divided by the displacement provides the elastic stiffness of the spring at that joint. This procedure
 343 has been repeated for each joint of all the columns apart from the one subjected to the sudden collapse.
 344 It should be underlined that since the axial and bending stiffness of the beams and the bending
 345 stiffness of the column in the studied frame are nullified, the computation of the translational stiffness
 346 at one joint is not influenced by the computation at another joint. Then, a spring on each joint of the
 347 frames under study is applied in ATENA 2D considering a linear-elastic constitutive model and

348 assigning the spring to the centre line of the joint, as shown in Fig. 8(b). The stiffness values of these
 349 elastic springs for each joint and each floor, computed considering the mean values for concrete
 350 properties, are reported in Table 2 and are effective for all the three frames (i.e., *frame 1*, *frame 2* and
 351 *frame 3*). The order of magnitude of these stiffnesses are in line with the results in many works
 352 considering both sub-assemblies and frames [69],[75],[79].
 353



354 Fig. 8 Calibration of the lateral springs: (a) example of calibrating the spring stiffness for the last floor, column A, in
 355 SAP2000; (b) springs (in blue) applied to the frames under study in ATENA-2D.

356 **5. PROBABILISTIC SAMPLING**

357 A full probabilistic approach has been performed, by sampling the following basic random variables
 358 regarding both materials and loads: concrete compressive strength f_c ; reinforced concrete specific-
 359 weight ρ ; reinforcing steel elastic modulus E_s ; reinforcing steel yielding strength f_y ; reinforcing steel
 360 ultimate strength f_u ; reinforcing steel ultimate strain ϵ_{su} ; permanent structural load G_1 ; permanent non-
 361 structural load G_2 ; floor variable load Q_f ; roofing variable load Q_r and snow load Q_s . According to
 362 the first step of the procedure (Section 2), the LHS technique [41] has been adopted to sample 100
 363 realizations. A sample size of 100 is assumed in line with the “rule of thumb” for which the sampling
 364 size should be around 10 times of the number of basic variables in order to have a reliable evaluation
 365 of the output variables [41]. At the same time, a sensitivity analysis of the final results as a function
 366 of the sampling size has been performed demonstrating the effectiveness of this size: the final
 367 probabilistic results have been evaluated for increasing discrete values of the sampling size and
 368 showed a quite stable trend in correspondence of 100 sampled values.

369 Table 3. Summary of the sampled basic random variables.

	Distribution	Mean value	Coefficient of variation [-]
f_c	Lognormal [44]	31.9 MPa [7],[9]-[10]	0.15 [44]
ρ	Normal [45]-[46]	25 kN/m ³ [7],[9]-[10]	0.05 [45]-[46]
E_s	Lognormal [44]	210000 MPa [7],[9]-[10]	0.03 [44]
f_y	Lognormal [44]	488.6 MPa [7],[9]-[10]	0.05 [44]
f_u	Lognormal [44]	589.8 MPa [7],[9]-[10]	0.05 [44]
ϵ_{su}	Lognormal [44]	0.14 [79]-[83]	0.09 [44]
G_1	Normal [43]	16 kN/m [7],[9]-[10]	0.05 [43]
G_2	Normal [43]	13 kN/m [7],[9]-[10]	0.05 [43]
Q_f	Gumbel [43]	7.3 kN/m [42]	0.20 [45]-[46]
Q_r	Gumbel [43]	1.8 kN/m [42]	0.20 [45]-[46]
Q_s	Gumbel [43]	4.7 kN/m [42]	0.20 [45]-[46]

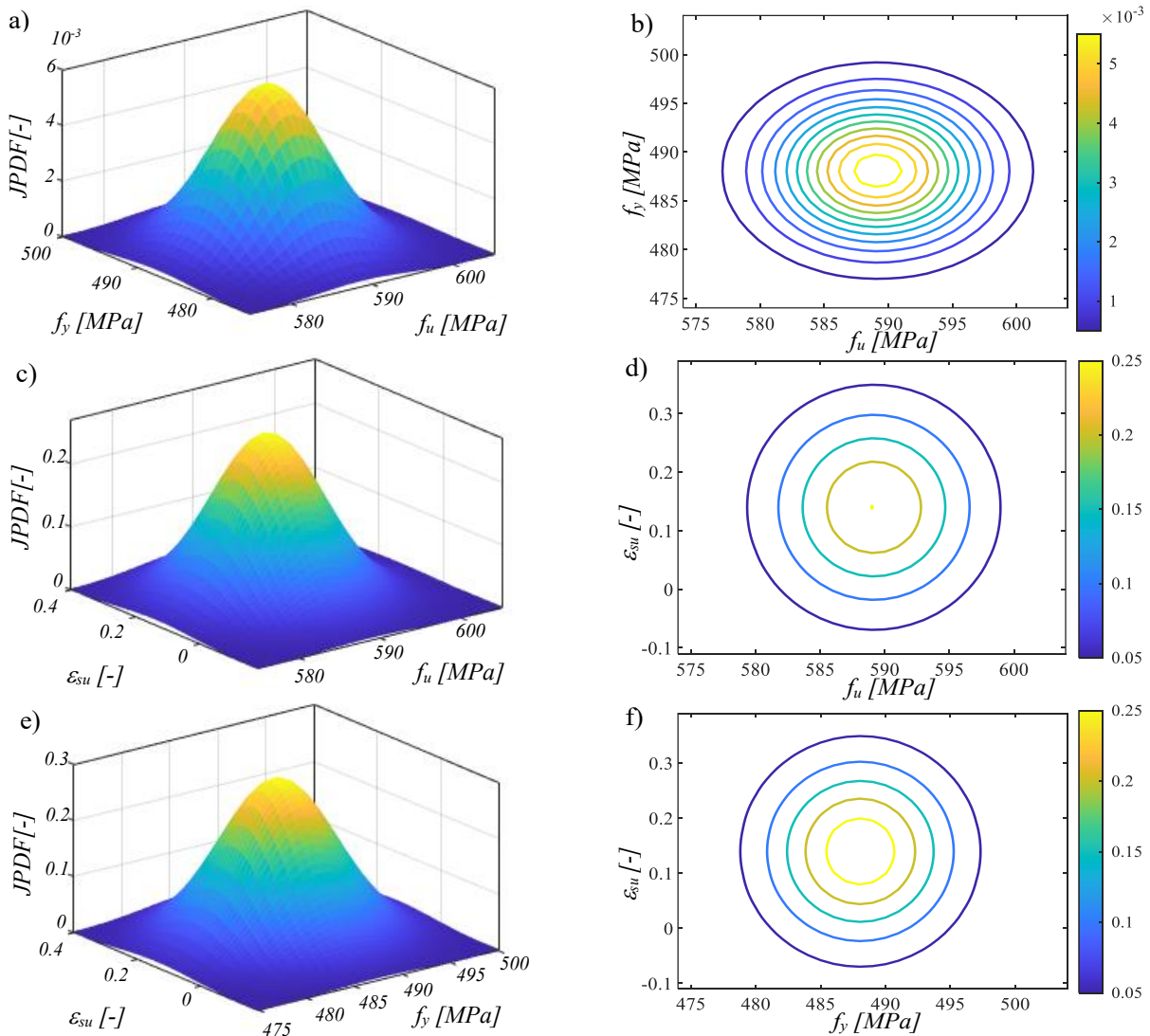
370 Mean values and coefficients of variation (CoVs) of the random variables concerning the material
 371 properties as well as the CoVs of the loads have been taken in accordance to [42]-[46]. Instead, as for
 372 the mean values of the latter ones, the nominal values deriving from the analysis of the effective area
 373 have been assumed for the permanent loads [7],[9]-[10], whereas the mean values for the variable
 374 loads have been obtained from the characteristic ones, related to a reference period of 50 years [42].
 375

376 Even if there is not a mandatory regulation, Model Code 2020 [84] suggests for the variable loads an
 377 annual reference period in case of an accidental scenario condition. However, the assumption of a
 378 reference period of 50 years for the variable loads is in line with other hazards, like seismic one,
 379 assumed in the design of the frames under investigation within a multi-risk view. Moreover, the larger
 380 the reference period the more severe is the load intensity and, thus, this hypothesis is on safe side.

381 Table 4. Full correlation matrix.

	f_c	ρ	E_s	f_y	f_u	ε_{su}	G_1	G_2	Q_f	Q_r
f_c	1	0	0	0	0	0	0	0	0	0
ρ		1	0	0	0	0	0	0	0	0
E_s			1	0	0	0	0	0	0	0
f_y				1	0.85	-0.50	0	0	0	0
f_u					1	-0.55	0	0	0	0
ε_{su}						1	0	0	0	0
G_1							1	0	0	0
G_2								1	0	0
Q_f									1	0
Q_r										1

382



383 Fig. 9 a) JPPDFs and contour plots for the following sets of correlated random variables: a)-b) f_u and f_y ; c)-d) f_u and ε_{su} ;
 384 e)-f) f_y and ε_{su} .

385 Regarding steel inelastic properties, a mean value for the ultimate strain equal to 0.14 with a ratio
 386 between the ultimate and yielding strength equal to 1.21 are assumed. These assumptions are in
 387 accordance with different experimental tests investigating the robustness of RC sub-assemblies
 388 [68],[79]-[82] as well as in line with the results of monotonic tensile tests conducted on a wide range
 389 of steel reinforcing specimens [83].

390 As highlighted in Subsection 4.1, the equivalent elastic springs have been calibrated adopting the
 391 mean properties of concrete parameters. The aleatory properties of the springs for each one of the 100
 392 random simulations have been obtained by multiplying their values by the quantity E_{ci}/E_{cm} , where
 393 E_{cm} is the mean concrete elastic modulus and E_{ci} is the concrete elastic modulus corresponding to the
 394 i^{th} sample of the concrete compressive strength. Table 3 resumes all the information regarding the
 395 distributions of the random variables with the corresponding mean values and CoVs.

396 The LHS realizations strictly depend on the correlation between the variables. In the case under
 397 analysis, correlations exist between the following variables: the tensile yielding strength f_y , tensile
 398 ultimate strength f_u and tensile ultimate strain ε_{su} of the reinforcement steel. The values suggested
 399 by [44] are used for the correlation coefficients. The full correlation matrix is reported in Table 4. In
 400 Fig. 9, the joint probability density functions (JPDFs) together with the contour plots for the three
 401 correlated variables are shown.

402 For completeness, it should be highlighted that this probabilistic analysis does not include geometrical
 403 uncertainties for their reduced influence on the global structural resistance [44],[59]-[62].

404 6. PROBABILISTIC CAPACITY CURVES TO ASSESS THE ENERGY-BASED DAFs

405 This section deals with the computation of both the aleatory capacity curves and DAFs according to
 406 the second and third step of the procedure in Section 2. These steps are needed to perform probabilistic
 407 static-equivalent NLFE global analyses accounting for the dynamicity of the column removal
 408 scenario.

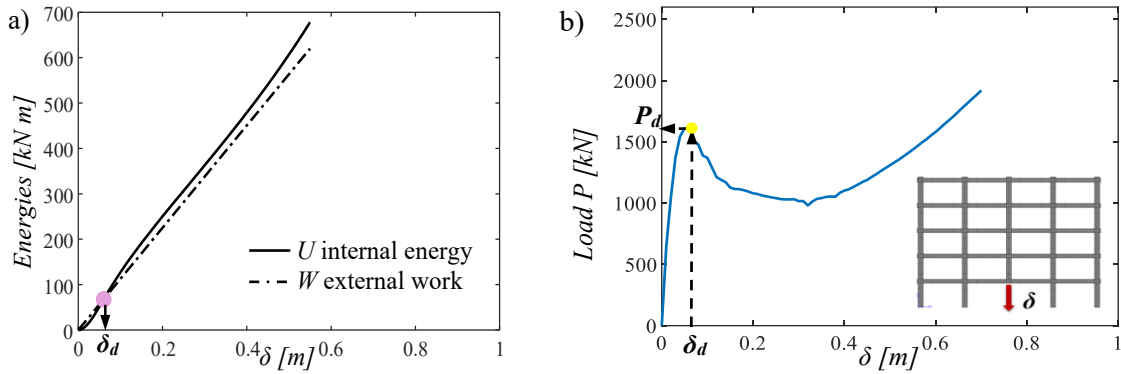
409 The DAFs are computed applying the energy equivalence method proposed by Izzuddin [23]. This
 410 method is based on the idea that an accidental column loss is a phenomenon physically associated to
 411 a sudden application of gravity loads on the affected sub-structure, i.e., the spans above. At the
 412 beginning (i.e., at the instant of column failure) the gravity load is larger than the static structural
 413 resistance, which determines an increase in the deformations and, consequentially, in the associated
 414 kinetic energy. While deformations grow more and more, the static structural resistance increases as
 415 well, causing a reduction in the kinetic energy, i.e., in the velocities. The maximum dynamic
 416 displacement δ_d is reached when its derivative is zero, thus, when the kinetic energy is null. This
 417 coincides with the performance point where the external work W , i.e., the work done by the gravity
 418 loads, is equal to the internal energy U , which is the energy absorbed by the structure. The internal
 419 energy can be computed as the area under the capacity curve (i.e., $P(\delta)$). These two quantities (i.e.,
 420 W and U) can be computed for the i^{th} realization, as follows:

$$W = \lambda_d P_o \delta_d \quad (2)$$

$$U = \int_0^{\delta_d} P(\delta) d\delta \quad (3)$$

421 The performance point where the two curves (i.e., internal energy curve and external work curve)
 422 have the intersection, corresponds to the dynamic displacement δ_d , as Fig. 10(a) illustrates for one
 423 aleatory case of the *frame 2*. Then, the dynamic load $P_d = \lambda_d P_o$ is evaluated as the load corresponding
 424 to the dynamic displacement in the pushdown curve (as shown in Fig. 10 (b)). Finally, the DAF,
 425 denoted as λ_d , can be computed according to Eq.(4) [23], where P_o represents the static gravity
 426 concentrated force at the top of the removed column.

$$\lambda_d = \frac{P_d}{P_o} \quad (4)$$



427 Fig. 10 Example of application of the energy equivalence method on a realization of the *frame 2*: (a) energy curves and
 428 calculation of the dynamic displacement; (b) pushdown curve with the calculation of the dynamic load.

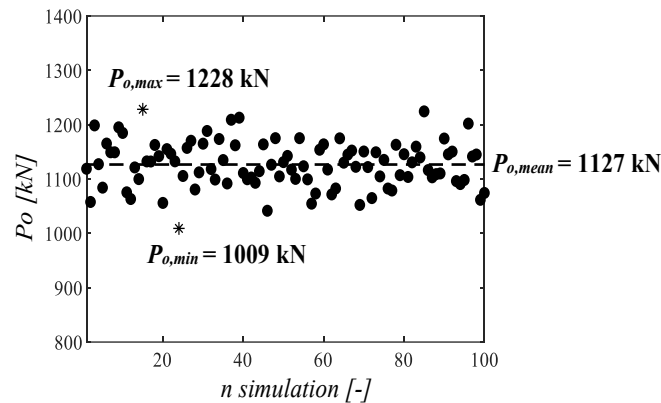
429 In the following, the probabilistic characterization of the two quantities P_o and P_d as well as of the
 430 energy-based DAFs are illustrated.

431

432 6.1 Assessment of the static gravity load P_o

433 For the calculation of the DAFs λ_d related to any aleatory frame, the static gravity concentrated force
 434 at the top of the removed column P_o needs to be evaluated. Since all the applied loads derive from a
 435 probabilistic sampling, the force P_o is consequentially a random variable. In detail, it has been
 436 computed multiplying the 100 aleatory distributed gravitational loads per unit meter (i.e., permanent
 437 structural and non-structural loads, snow load and live loads - distinguishing between the floors and
 438 the roof), placed according to the load combination of [8], by the effective width to obtain the
 439 corresponding resultant gravity force in the point of column removal. The aleatory values of P_o are
 440 shown in Fig. 11. It should be noted that these values of P_o are equal for all the three frames since
 441 they basically depend on the sampled external loads, which are equal for all the three frames (i.e.,
 442 *frame 1*, *frame 2* and *frame 3*).

443



444 Fig. 11 Aleatory values of the static gravity concentrated force at the top of the removed column for the three frames.

445 6.2 Aleatory capacity curves and assessment of the dynamic response P_d

446 To obtain the capacity curves, needed for the evaluation of the dynamic response P_d , static
 447 displacement-controlled pushdown NLFEMs have been performed [85]-[87] for the entire frames, as
 448 the second step of the procedure (Section 2). Three sets of 100 random NLFEM models have been
 449 defined, each one with different values for the material properties but having the same geometry,
 450 restrains and constraints. For these analyses, the entire frames have been modelled without the central
 451 supporting column. Each numerical simulation consists in imposing an increasing vertical
 452 displacement δ with increase of 1cm where the column is accidentally removed and monitoring the
 453 reaction in the same point. The application of such gradually increasing displacement determines a
 454 continuously changing stress path in the entire structure. The results of these NLFEM global analyses
 455 are aleatory force-displacement curves, the so-called aleatory pushdown or capacity curves.

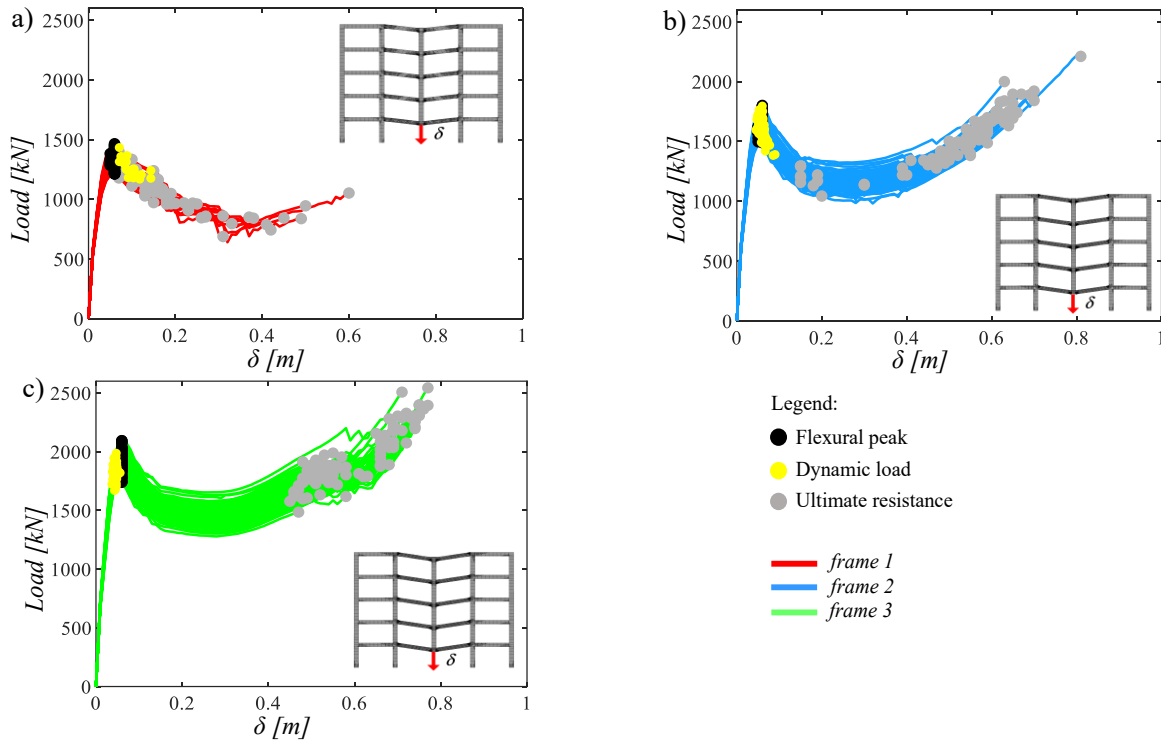


Fig. 12 Aleatory capacity curves considering flexural peak (black dots), ultimate resistance (grey dots) and the resistance at the energetic equivalence (yellow dots): (a) *frame 1*; (b) *frame 2*; (c) *frame 3*.

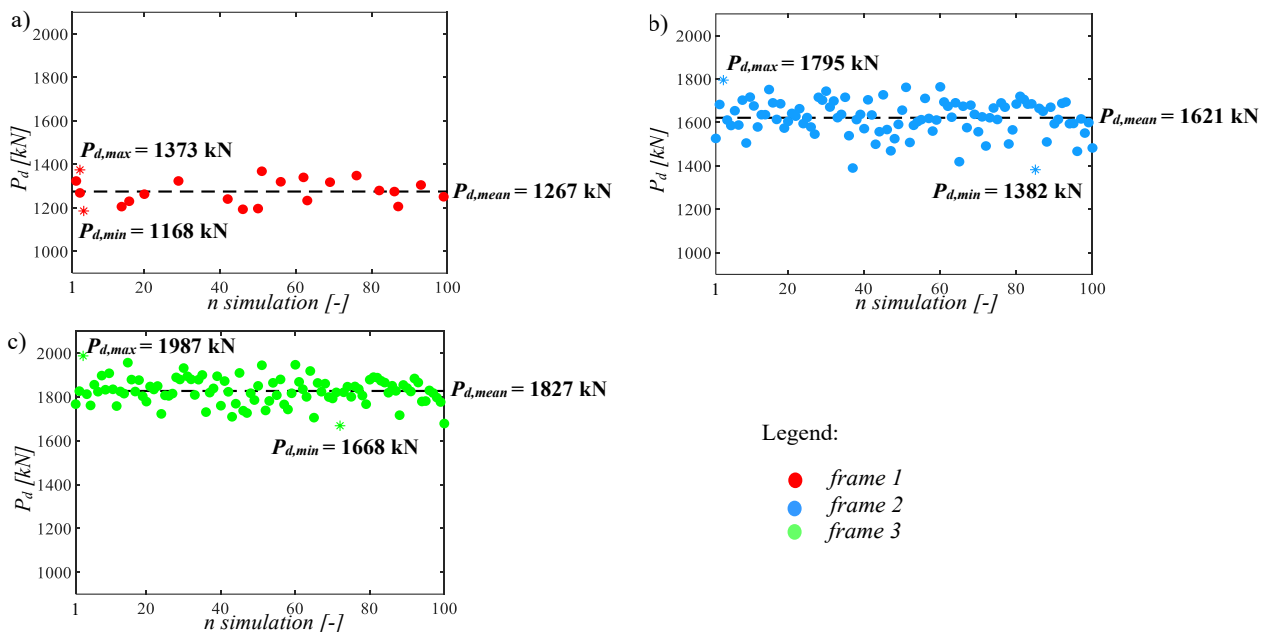
456
457

In Fig. 12, the 300 aleatory capacity curves related to the three RC MR frames (i.e., *frame 1*, *frame 2* and *frame 3*) are showed, highlighting the maximum resistant load in the flexural phase or flexural peak $P_{MAX,FL}$ (black dots), the ultimate resistance before failure $P_{MAX,RES}$ (grey dots) and the resistance at which the energetic equivalence is reached P_d (yellow dots). In addition, an example of an aleatory failure mode for each one of the three frames is shown. From the simulations, especially, of the *frame 2* and *frame 3*, three different stages can be clearly visible: flexural stage including compressive arch action, softening stage and catenary effect stage [40]. The first one is initially governed by a linear elastic behaviour of the materials until $P_{MAX,FL}$ is reached. For all the three frames, the displacements needed to reach $P_{MAX,FL}$ are similar but the values of the corresponding peak loads change. This is mainly due to the differences in the reinforcement layouts as well as to the dispersion of concrete compressive strength, which also influences the behaviour of the transverse elements as defined in Subsection 4.1, since the steel yielding strength is characterized by a quite low CoV (i.e., 0.05). After the flexural peak, the softening stage follows, characterised by a reduction of the reaction due to the crushing of concrete and consequent reduction of compression behaviour of the beams for the constraints (i.e., the surrounding columns). The amplitude of the second stage, is more varying as a function of the sampled material properties. The third stage starts when the catenary effect activates, determining an increase of the reaction for increasing vertical displacements in the point of the removal. This effect derives from a combination of both the flexural and catenary effects in the reinforcing bars and, thus, is mainly governed by the steel properties (e.g., CoV associated to the steel ultimate strain, i.e., 0.09). Consequently, also the displacement value, at which the catenary peaks are reached, varies as well. By comparing the three frames, it is more and more evident that the catenary is activated thanks to the reinforcement continuity and starts for low displacements around 0.2 m thanks to the side face rebars [40]. Moreover, the catenary effects (i.e., recovery in resistance) is evident in both the *frame 2* and *frame 3* but it is quite completely absent in the *frame 1*, confirming the effectiveness of the robustness improvements. This absence is due to concrete crushing in the softening stage as well as failure in longitudinal rebars (without the help of the side face rebars), as investigated in terms of strain results in the next sections.

By applying the energy equivalence approach in all 300 aleatory frames, the external works and internal energies are evaluated, as previously described, and then, the aleatory dynamic displacements

485
486

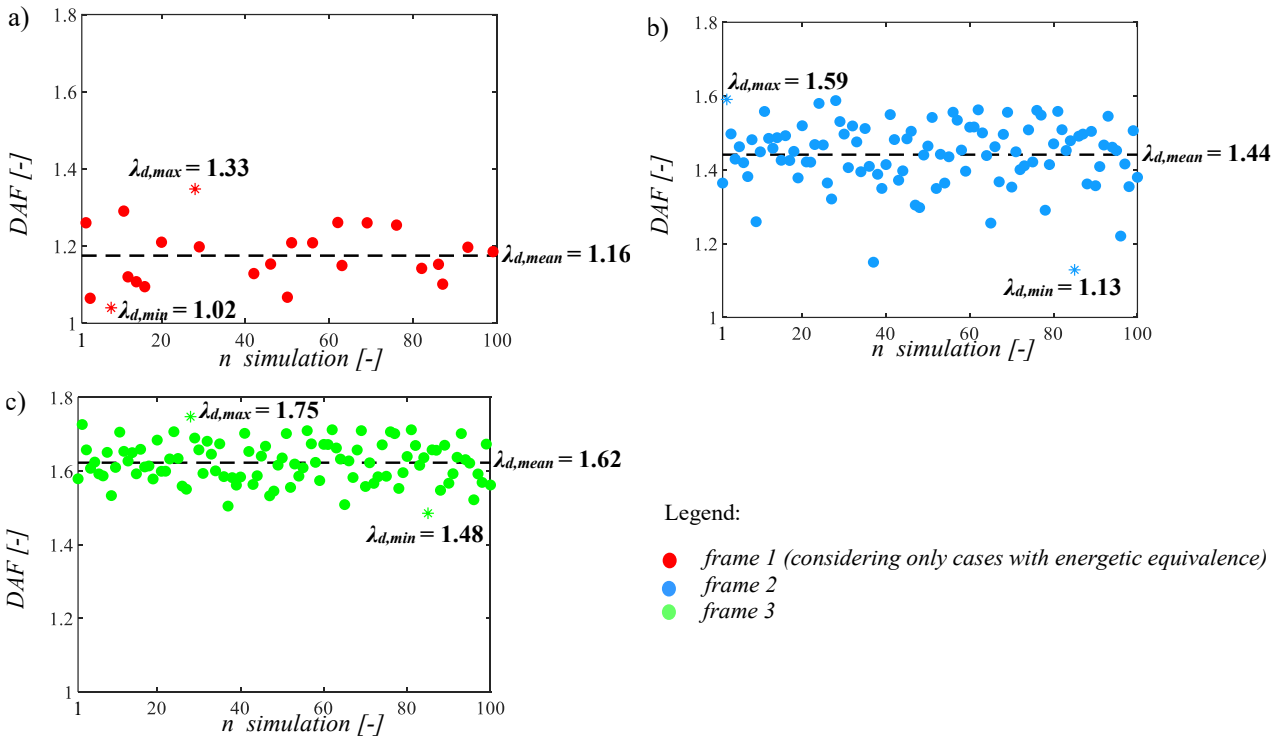
487 δ_d with corresponding dynamic loads P_d are computed. In Fig. 12, the resistances at which the energy
 488 equivalence is reached (i.e., yellow dots) are also indicated. The equivalences are obtained for lower
 489 imposed vertical displacement for both the *frame 2* and *frame 3* with respect to the *frame 1*. This last
 490 result highlights that not very large strains in the reinforcement are demanded at the performance
 491 points (i.e., yellow dots) and, also confirms the simplified assumption of the equivalent elastic springs
 492 since the horizontal displacements corresponding to performance points (i.e., yellow dots) are not too
 493 large [40]. It should also be underlined that in Fig. 12(a) for the *frame 1* only the P_d values, related to
 494 the cases in which energy equivalences are reached, are shown. The aleatory dynamic loads are shown
 495 in Fig. 13 for the three frames. It should be highlighted that for the *frame 1* (Fig. 13(a)) (i.e., designed
 496 according to current standards) the two energy curves do not find an intersection in majority of the
 497 cases, meaning that the structure, in these cases, is not able to sustain the accidental loss of the central
 498 supporting column. For this reason, in Fig. 13(a) the aleatory dynamic reactions are represented only
 499 for the cases where the energy equivalences were reached: these points are close to the flexural peaks.
 500 In addition, the *frame 3* leads to the largest values of dynamic reactions if compared to the *frame 2*.
 501 This is because the larger quantity of reinforcement implies a larger bearing capacity for the RC
 502 structure.
 503



504 Fig. 13 Aleatory dynamic gravity load P_d obtained from the capacity curves for: (a) *frame 1* (considering only cases
 505 with energetic equivalence); (b) *frame 2*; (c) *frame 3*.

506
 507 **6.3 Assessment of the aleatory Dynamic Amplification Factors (DAFs)**

508 As described in the previous subsection, once the aleatory concentrated dynamic load is computed as
 509 the value of the capacity curve corresponding to the dynamic displacement, the corresponding
 510 aleatory energy-based DAF can be computed as $\lambda_d = P_d/P_o$ through Eq.(4), and represents the third
 511 step of the procedure in Section 2. Fig. 14 shows the aleatory values of the DAFs as a function of the
 512 number of simulations (from 1 to 100) for each frame. It should be noted that the larger the capacity
 513 of the structure to sustain the accidental event, the larger is the DAF to be applied to gravity loads.
 514 For this reason, the mean value of the DAFs for the *frame 3*, equal to 1.62, is larger than the one for
 515 the *frame 2*, equal to 1.44. Again, for the *frame 1* only the cases corresponding to energy equivalences
 516 are shown, with a mean value of 1.16, confirming the trend previously explained.
 517 From these results, it is interesting to note that the aleatory DAFs in case of non-linear response are
 518 lower than 2.0, value indicated by [20]-[21] for linear static analyses. Similar results may be found in
 519 different experimental and numerical investigations [88]-[89].



520 Fig. 14 Aleatory energy-based DAFs for: a) *frame 1*; b) *frame 2*; c) *frame 3*.

521
 522

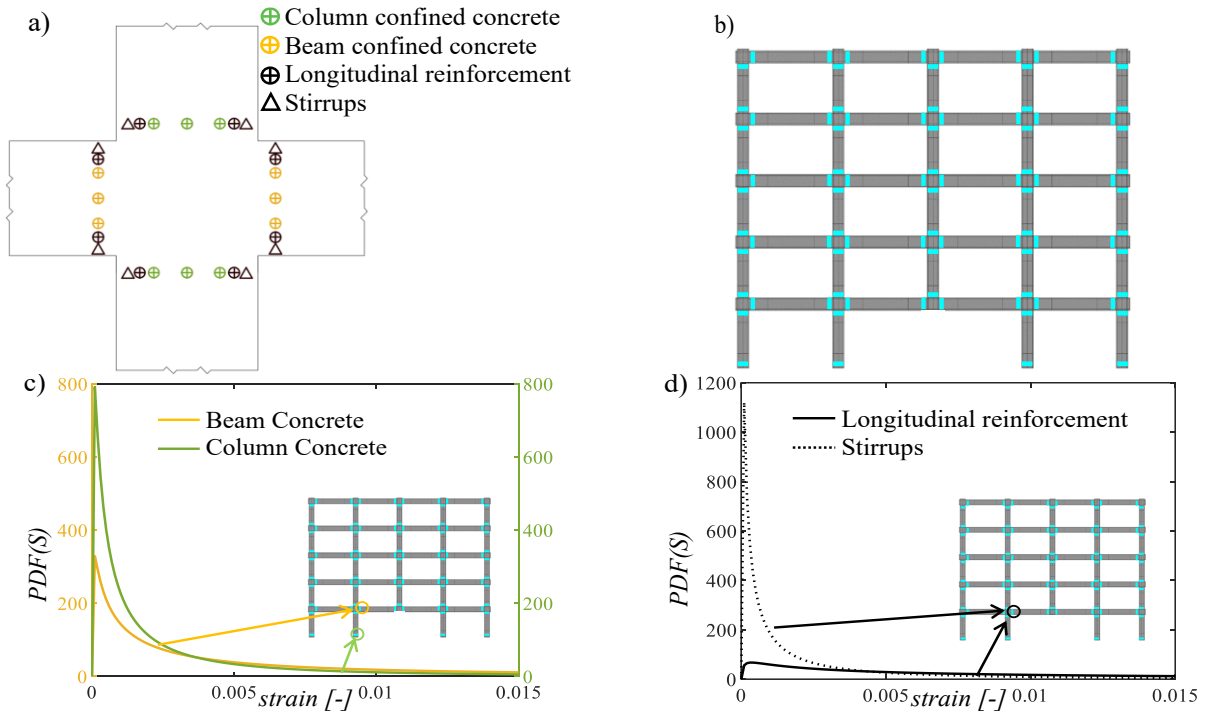
7. PROBABILISTIC ANALYSES FOR THE RELIABILITY ASSESSMENT OF THE STRUCTURAL ROBUSTNESS

523 In the first subsection, the probabilistic static-equivalent NLFE global analyses are described, and the
 524 results are probabilistically modelled by means of appropriate probability density functions (PDFs).
 525 In the second subsection, the cumulative distribution functions (CDFs) related to the ultimate capacity
 526 of the different materials as well as the failure probabilities are computed to achieve the reliability
 527 assessment of the robustness at structural level with respect to the ULS for the three frames under
 528 investigation (i.e., *frame 1*, *frame 2* and *frame 3*). A discussion of the results is in the last subsection.
 529

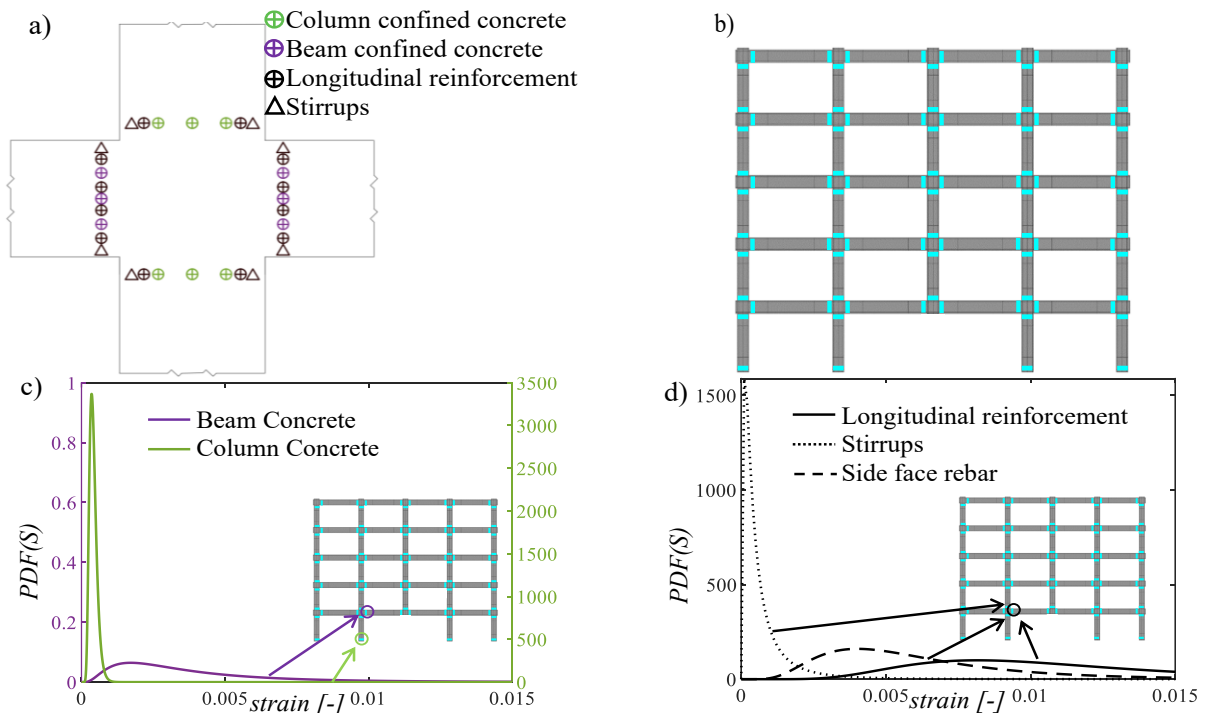
530 7.1 Probabilistic analysis: aleatory static-equivalent NLFE simulations

531 In compliance with the fourth step of the procedure (Section 2), for both the *frame 2* and *frame 3*, two
 532 sets of 100 static-equivalent NLFE global simulations have been performed by applying the gravity
 533 loads placed in line with the accidental load combination of [8] (the same adopted in Subsection 6.1),
 534 removing the central supporting column and amplifying the gravity loads only on the spans of the
 535 five floors adjacent to the central column through the previously computed DAFs. The other spans
 536 are not amplified since the propagation of the dynamic effects is damped by the structural RC
 537 elements, as achieved in some preliminary dynamic simulations and in accordance with [39],[88]-
 538 [89]. After the amplification, collapse was never reached in all the simulations confirming the
 539 prediction of the energetic approach. As for the *frame 1*, similarly, static-equivalent NLFE global
 540 simulations have been carried out when the DAFs exist, whereas, in the other cases, static-equivalent
 541 NLFE global simulations have been run achieving two possible results as predicted by the energetic
 542 approach: i) failure reached after removing the supporting column without amplifying the loads; ii)
 543 failure reached by slightly amplifying the loads after the removal. These results, in terms of collapse
 544 or no-collapse related to the three frames, also confirm that the simplification of applying a
 545 concentrated displacement instead of a distributed configuration (hypothesis closer to the reality) in
 546 the evaluation of both capacity curves and energetic performance points is a reasonable hypothesis.
 547 Then, for each set of simulations, the aleatory peak principal total strains (ϵ) in significant points of
 548 the entire RC frames (Fig. 15(a)-(b), Fig. 16(a)-(b), Fig. 17(a)-(b)) have been extrapolated at the last

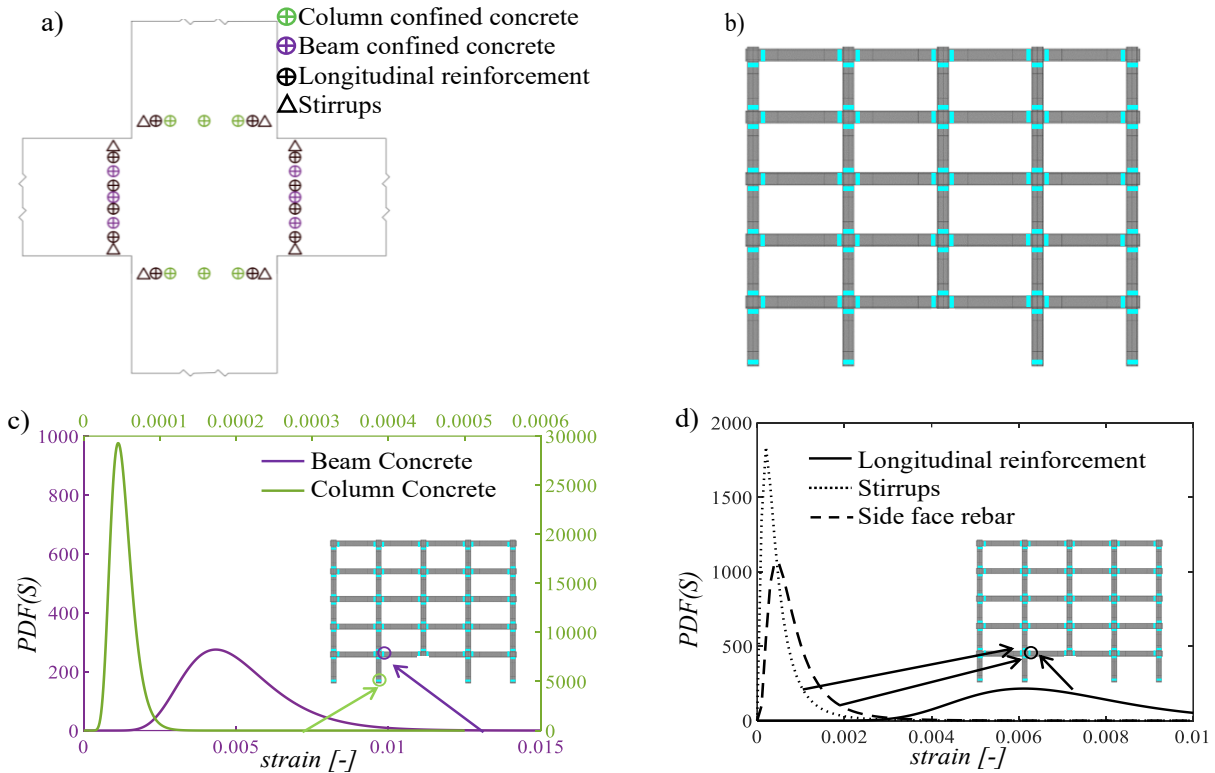
549 numerical step of equilibrium. In detail, the points of both beams and columns close to each beam-
 550 column joint have been considered, identifying four sections for each beam-column joint (except for
 551 the corner columns as well as for the last floor where three or two cross-sections are, respectively,
 552 present). Successively, in each cross-section, different points have been considered distinguishing
 553 between the points of the confined concrete core, of the upper and lower longitudinal reinforcement,
 554 of the side face rebars (only for the *frame 2* and *frame 3*) and of the stirrups. For clearness, it is worth
 555 noting that the colours adopted in Fig. 15 - Fig. 17 to distinguish between the different materials are
 556 the same of the constitutive laws shown in Fig. 5, with different characteristics.
 557



558 Fig. 15 Frame 1: a)-b) Cross-sections close to the beam-column joint where the demand is evaluated; c)-d) PDFs of the
 559 demanded aleatory peak strains in the most stressed cross-sections for each material.



560 Fig. 16 Frame 2: a)-b) Cross-sections close to the beam-column joint where the demand is evaluated; c)-d) PDFs of the
 561 demanded aleatory peak strains in the most stressed cross-sections for each material.



562 Fig. 17 *Frame 3*: a)-b) Cross-sections close to the beam-column joint where the demand is evaluated; c)-d) PDFs of the
 563 demanded aleatory peak strains in the most stressed cross-sections for each material.

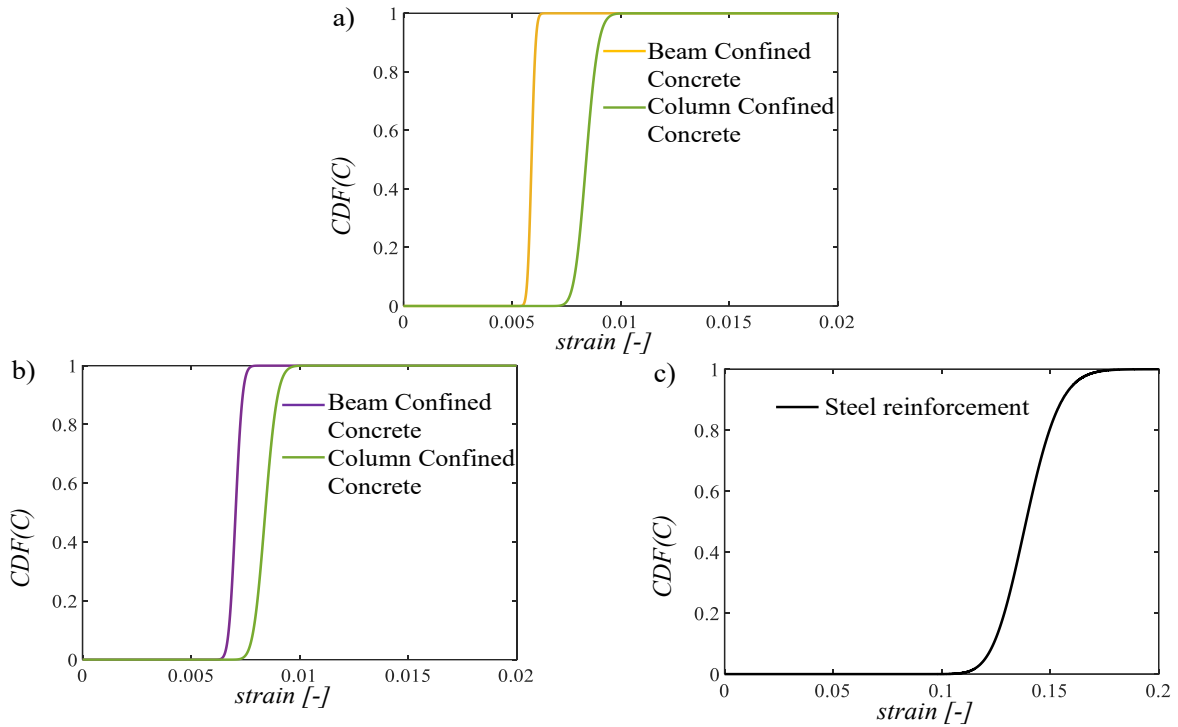
564 A statistical inference analysis was needed to investigate the PDFs for the aleatory peak strains in the
 565 different points of the structure by applying both the Chi-Square and Anderson Darling tests. In detail,
 566 the Chi-Square goodness of fit test is adopted to check whether the frequency distribution of a variable
 567 (i.e., the strain) is different from the expectation, while the Anderson Darling test is used to verify,
 568 with a certain level of significance (i.e., 5%), if a sample of data comes from a population with a
 569 specific distribution. Hence, the lognormal distribution has been selected as the proper probabilistic
 570 distribution since it has always passed the goodness of fit test with a high p-values (around 0.8). The
 571 statistical inference analysis has been performed for all the three frames leading to the same
 572 conclusions. In this way, the statistics of the output aleatory peak strains have been computed through
 573 the Maximum Likelihood technique [90] including also the statistical uncertainties according to the
 574 Fisher Information Matrix [91] due to the no very large size, especially, for the *frame 1*. Fig. 15(c)-
 575 (d), Fig. 16(c)-(d), Fig. 17(c)-(d) show examples of the PDFs of the demanded aleatory peak strains
 576 in the most stressed points for each material or component (i.e., confined concrete in beams, confined
 577 concrete in columns, longitudinal reinforcing bars, side face rebars and stirrups) related to the *frame*
 578 *1*, *frame 2* and *frame 3*, respectively.

580 7.2 Reliability assessment

581 According to the last step of the procedure in Section 2, for each material of structural elements
 582 (shown in Fig. 5), denoting as S the demand (i.e., the aleatory peak strains monitored at the points)
 583 and C the capacity (i.e., the aleatory ultimate strain of the materials), the failure probability is the
 584 probability that the aleatory demand overcomes the aleatory capacity with respect to the ULS, thus it
 585 can be expressed through the convolution integral [49] as follows:

$$p_f = P(C < S) = \int_0^{\infty} \left[\int_0^s f_C(c) dc \right] f_S(s) ds = \int_0^{\infty} F_C(s) f_S(s) ds \quad (5)$$

586 where $F_C(s)$ is the CDF of C evaluated at specific realizations of the demand (i.e., s), $f_S(s)$ and $f_C(c)$
 587 are the PDFs, respectively, of S and C .



588 Fig. 18 CDFs of the capacity for different materials: a) confined concrete in the most stressed cross-section for the
 589 *frame 1*; b) confined concrete in the most stressed cross-section for both the *frame 2* and *frame 3*; c) steel reinforcement.

590 For each of the points in any cross-section of the beam-column joints within each frame, a convolution
 591 integral is performed between the PDF of the demand $f_S(s)$, computed in the previous subsection, and
 592 the CDF of the capacity $F_C(s)$, obtained as the CDF of the sampled ultimate strains thresholds (Fig.
 593 18). In detail, for the steel, the ultimate strain is considered, as sampled in Section 5, while for the
 594 concrete, the ultimate strain in confined condition, as confirmed by the numerical analyses, is
 595 indirectly computed from the sampled concrete compressive strength, and assumed equal to the strain
 596 corresponding to a post-peak strength equal to the 85% of the peak strength, on safe side. All the
 597 ultimate strains are modelled through lognormal distributions, as described in Section 5. For example,
 598 Fig. 18(a)-(b) show the CDFs of the capacity for confined concrete related to the *frame 1*, *frame 2*
 599 and *frame 3*, respectively. Fig. 18(c) shows the CDF of the capacity for steel, that is equal for any
 600 reinforcement and for all three frames.

601 The probabilities, computed through Eq. (5), represent the failure probabilities with respect to the
 602 ULS, correspond to the term $P[C|LD]$ of Eq. (1) and are related to a system failure condition since
 603 NLFE global analyses have been carried out.

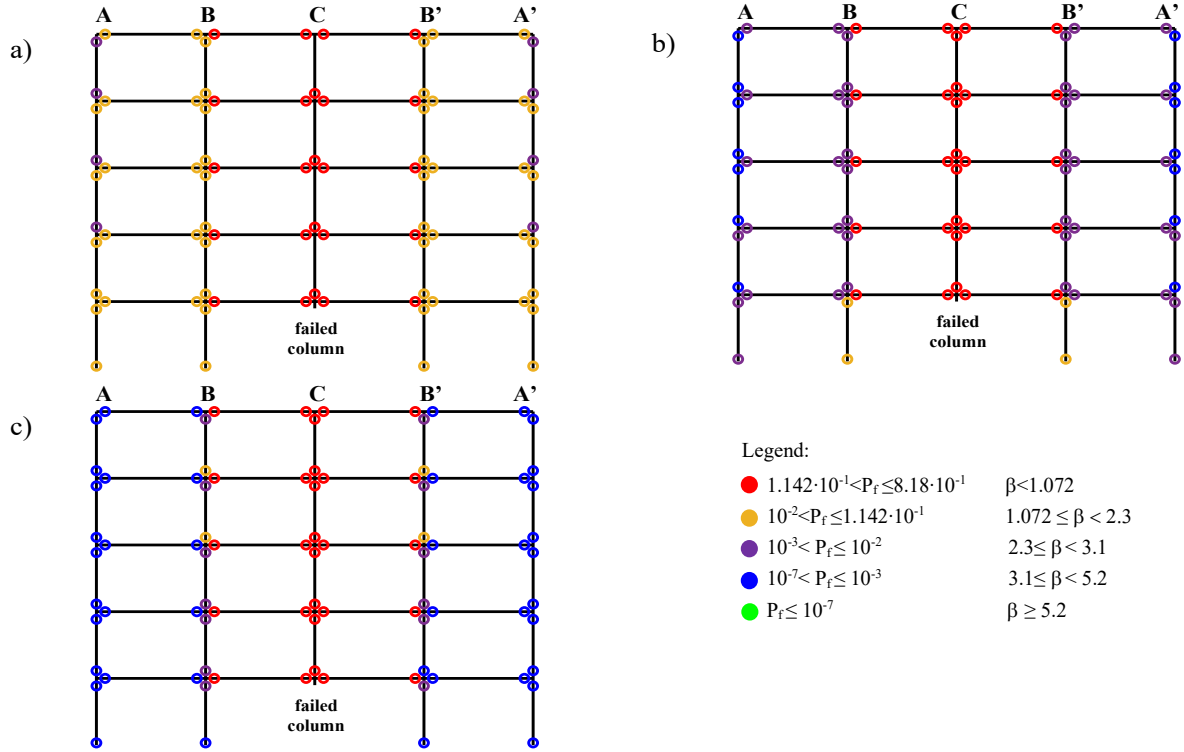
604 In Fig. 15(c)-(d), Fig. 16(c)-(d), Fig. 17(c)-(d), the PDFs of the demand S in the most stressed sections
 605 are shown for the different materials and, respectively, for the *frame 1*, *frame 2* and *frame 3*. It should
 606 be underlined that the concrete in tension is not considered for computation of the failure probabilities
 607 since it does not behave as a resisting material in the failure modes at those deformation levels.

608 Note that regarding the *frame 1*, the total probability theorem [92] has been adopted to compute the
 609 failure probabilities of the beams and columns in the spans close to the failure scenario due to the
 610 collapses of the majority cases achieved from the probabilistic static-equivalent NLFE global
 611 simulations. The expression [92] applies, as follows:

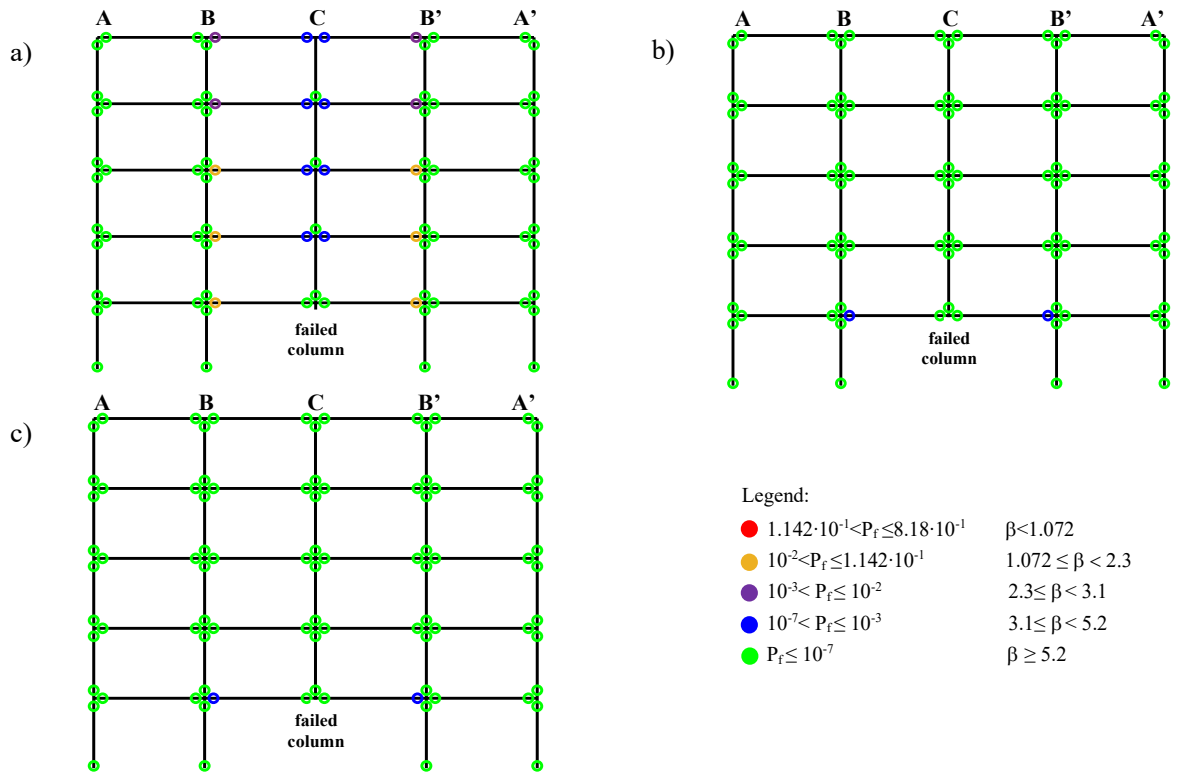
$$P_{f, frame1} = \frac{N_{non-collapse}}{N_{tot}} \cdot \left[\int_0^{\infty} F_C(s) f_S(s) ds \right]_{non-collapse cases} + \frac{N_{collapse}}{N_{tot}} \cdot 1 \quad (6)$$

612 where, $N_{collapse}$ is the number of collapse cases derived from the energy equivalence approach, N_{tot} is
 613 the total number of samples (i.e., 100) and $N_{non-collapse}$ is the complementary part of $N_{collapse}$ with
 614 respect to N_{tot} .

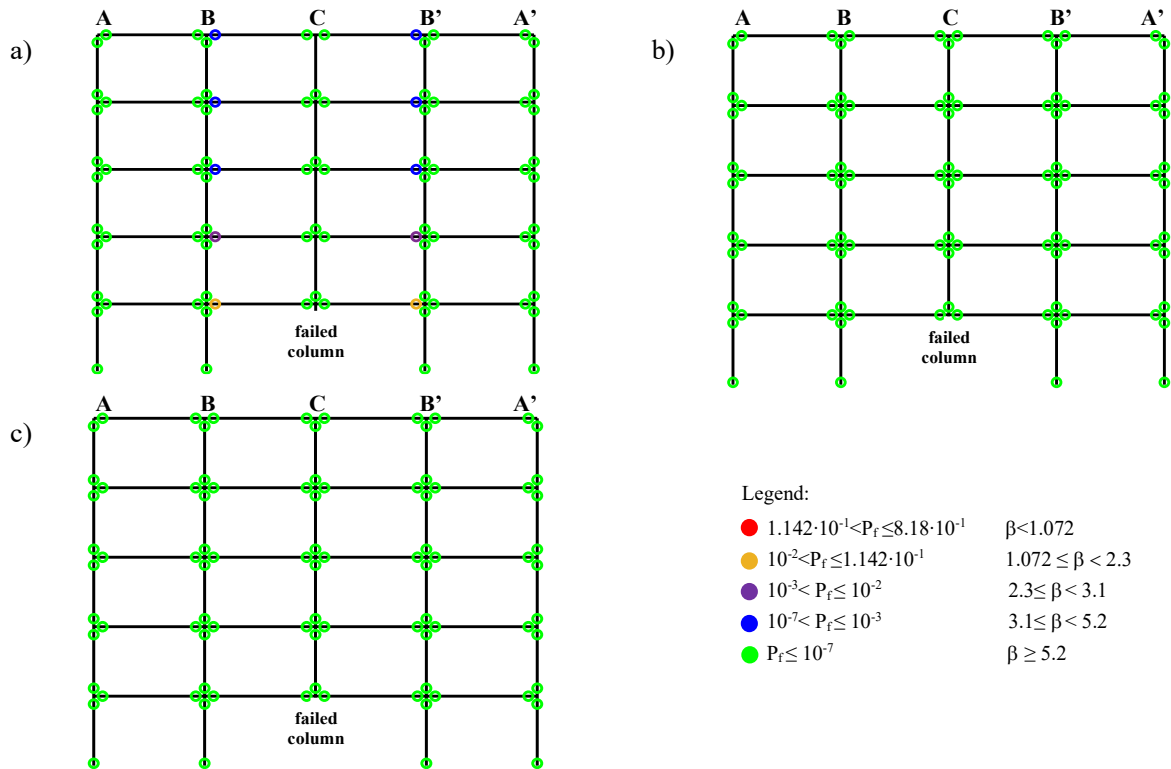
615 In the following, the failure probabilities for each section close to the beam-column joints are shown,
 616 separating the probability related to a) the confined concrete material (only in compression), b) the
 617 longitudinal reinforcement and c) the transverse reinforcement for the *frame 1* (Fig. 19), *frame 2* (Fig.
 618 20) and *frame 3* (Fig. 21), respectively. This probabilistic computation in the most stressed points
 619 allows also to define the propagation of the damage, useful for robustness considerations.
 620



621 Fig. 19 Failure probabilities for the *frame 1* considering: a) confined concrete; b) steel longitudinal reinforcement;
 622 c) steel transverse reinforcement.



623 Fig. 20 Failure probabilities for the *frame 2* considering: a) confined concrete; b) steel longitudinal reinforcement;
 624 c) steel transverse reinforcement.



625 Fig. 21 Failure probabilities for the *frame 3* considering: a) confined concrete; b) steel longitudinal reinforcement;
 626 c) steel transverse reinforcement.

627 Regarding, especially, the *frame 2* and *frame 3*, in the beams close to the failed column, the failure
 628 probabilities at the concrete level decrease in the upper stories in the cross-sections connected to the
 629 columns B and B' as well as are higher than those related to the cross-sections connected to the
 630 column C. This is due to the constrain actions, higher at the lower levels, developed by the other
 631 structural members. In Fig. 20(a), it is possible to observe that the cross-sections connected to the
 632 column C at the first floor show low failure probabilities but slightly higher than those corresponding
 633 to upper floors. This happens numerically also in the *frame 1* (Fig. 19(a)) and *frame 3* (Fig. 21(a))
 634 and it is a consequence of the absence of the failed column at the first floor.

635 As a general observation, the *frame 1* is characterized by the largest failure probabilities if compared
 636 to the other two *frames* and this is valid for all the investigated materials. In fact, the *frame 1* shows
 637 failure probabilities in concrete (Fig. 19(a)) between 10^{-3} - 10^{-1} in the lateral spans, even if they are
 638 indirectly affected by the column loss. The largest value (i.e., 0.818) is obviously reached in the beams
 639 of the first floor adjacent to the removal, but also the beams of the upper floors are subjected to values
 640 much larger than 10^{-1} . Regarding the longitudinal reinforcement (Fig. 19(b)), the failure probabilities
 641 are lower in the external spans but are always larger in the central ones (i.e., much larger than 10^{-1}).
 642 This suggests that the amount of reinforcement, derived from the standard seismic design, is not
 643 enough to guarantee resistance in robustness response. In addition, since the stirrups (Fig. 19(c)) are
 644 not subjected to the same state of stress as for the longitudinal reinforcement, the failure probabilities
 645 are lower.

646 By analysing the results of the *frame 2*, important improvements are registered, especially, regarding
 647 the failure probabilities for the longitudinal reinforcement (Fig. 20(b)). Apart from the beams of the
 648 central spans at the first floor, all the other beam-column sections are characterized by very low failure
 649 probabilities (i.e., lower than 10^{-7}). This result suggests the importance of considering the side face
 650 rebars in the design of this type of structures. Regarding the confined concrete in compression (Fig.
 651 20(a)), lower failure probabilities are obtained with respect to the previous case, especially, in the
 652 external spans, confirming that the presence of the side face rebars tend to reduce the mechanical
 653 effort at the concrete level. In fact, the safety level regarding confined concrete in terms of β minimum
 654 (in 50 years as structural reference period) is far below 1 for the *frame 1*, while it has a value larger

655 than 1 for the *frame 2* (i.e., 1.08). Similar considerations can be drawn for the *frame 3* (Fig. 21), with
656 the main difference that concrete is characterized by even lower failure probabilities with respect to
657 the previous case (i.e., β minimum equal to 1.23). The values of β higher than 1 for the *frame 2* and
658 *frame 3* represent an important achievement in line with the provisions recommended in [14] (i.e.,
659 1.5) where the structural reference period is 1 year.

660 Regarding the steel longitudinal reinforcement, the β minimum is far below 1 for the *frame 1*, equal
661 to 5.11 and 7.45 for, respectively, the *frame 2* and *frame 3*. These last two results become slightly
662 lower (i.e., up to around 4) even if the mean value of the ultimate strain of the reinforcement is
663 assumed lower than 0.14 (e.g., up to 0.075 [9]) since the energetic equivalences are reached for not
664 large vertical displacements (Section 6.2).

665 It is necessary to highlight that these very low failure probabilities at the reinforcement level (Fig.
666 20(b) and Fig. 21(b)) together with the activation of catenary effect visible in Fig. 12(b)-(c) are of
667 great importance especially when dynamic phenomena and large displacement are involved and, thus,
668 the response is mainly governed by the steel.

669 The maximum failure probabilities obtained in this investigation (i.e., 0.82, 0.14, 0.11 for,
670 respectively, the *frame 1*, *frame 2* and *frame 3*) are comparable with the ones in different studies. For
671 instance, in [34], considering an amplification of the load equal to 1.3, failure probabilities of around
672 0.75 for complete damage limit state were obtained for seismically designed 2D RC internal frames,
673 in case of lateral column failure scenario. The frames were characterised by continuous and
674 symmetric longitudinal reinforcement, equal in all the floors. However, in [34], a lower CoV (i.e.,
675 equal to 0.1) for the concrete strength was adopted. Another comparison can be carried out with
676 respect to [38], where a force-based reliability computation is considered and a failure probability of
677 0.074 for a five-storey RC frame is obtained, in case of central supporting column loss scenario. The
678 frame of [38] is a seismically-designed symmetric frame and the beams have continuity with a
679 comparable longitudinal reinforcement layout between the lower and upper chords in all the floors.
680 However, it should be underlined that in [38] a force-based approach instead of a strain-based one
681 has been adopted and the steel properties are characterised by a lower dispersion in the statistical
682 characterization. Finally, with reference to the results by [39], planar frames (both internal and
683 perimetral) of two RC office buildings are studied in terms of failure probability, and values of 0.173
684 and 0.254 were obtained for, respectively, the 3-story and 6-story internal frame. However, the
685 buildings have horizontal stiffness provided by the bracings and no symmetry or floor equality criteria
686 as well as no presence of side face rebars is included. In addition, a reference period for the variable
687 loads of 5 years is assumed.

688 689 7.3 Final discussion

690 The proposals together with the results achieved provide the reliability levels with respect to the ULS
691 in any structural element of the three RC MR frames having different design recommendations with
692 respect to robustness. From the comparison between the three RC MR frames, it is possible to observe
693 the relevant safety advantages together with a significant reduction of the damage propagation in
694 reliability terms when different robustness design improvements are adopted: the failure probabilities
695 are strongly lower in the spans close to the collapse scenario, the safety level increases in any
696 structural element for increasing distance from the collapse event highly reducing the damage
697 propagation. In addition, the adopted robustness suggestions lead to values of β minimum slightly
698 higher than 1 within a structural reference period of 50 years. This result is absolutely relevant since
699 the guidelines [14] suggests a minimum value for β equal to 1.5 considering a structural reference
700 period of 1 year. In addition, the results refer to a reference period for variable loads of 50 years
701 differently from the suggestion in Model Code 2020 [84] of an annual reference period in case of an
702 accidental scenario condition.

703 As a result, it can be concluded that for this specific failure scenario and this structural typology (i.e.,
704 cast in-situ RC structures having slabs with joists), the design with side face rebars and continuity
705 (i.e., *frame 2*) appears to be the leading solution since it presents quite the same level of safety as the

706 *frame 3* with reduced costs (i.e., reduced amount of reinforcement). Therefore, the design solution of
707 the *frame 2* represents a good trade-off between principles of robustness, sustainability [1]-[3] and
708 safety in accordance also with seismic design within a multi-risk analysis. In fact, the percentage of
709 increase of both longitudinal and transverse reinforcement with respect to the code-conforming
710 structure (i.e., *frame 1*) is equal to 21% for the *frame 2* and 36% for the *frame 3*.

711 Note also that the contribution in the global structural resistance provided by the orthogonal frame
712 affected by the same column loss, by the infills as well as by the membrane effects are not included
713 in the reliability analysis and, thus, the proposed results are on the safe side. Future developments of
714 this study could investigate different failure scenarios (i.e., corner columns or second-to-last
715 columns), also including the above-mentioned beneficial effects. In fact, the central supporting
716 column loss is one of the most critical scenarios for the absence of the other positive effects. In
717 addition, other load conditions combined with other structural typologies could be examined.

718 8. CONCLUSIONS

719 This study describes a strain-based 5-step procedure to assess the structural robustness in reliability
720 terms of 2D RC MR frames located in a seismic area, in case of the accidental loss of the central
721 supporting column. The goal was to give insights into the safety level of frames designed according
722 to actual code rules and enhanced frames in order to achieve a more robust behaviour. In fact, actual
723 regulations do not give any provision on the safety level to be respected in case of progressive collapse
724 phenomena, but only recommended values are suggested. In detail, three different frames have been
725 compared: *frame 1* designed according to the current European and Italian code rules; *frame 2* adopts
726 continuous longitudinal reinforcement with two additional levels of side face rebars, while the *frame*
727 *3* derives from the *frame 2* adopting the same reinforcement amount in all the floors with symmetric
728 cross-sections to exploit a global Vierendeel behaviour.

729 According to the 5-step procedure, a full probabilistic approach has been followed by sampling
730 different basic random variables, related to both material properties and applied external loads.

731 The main outcomes can be summarized as follows:

- 732 - the results in terms of aleatory capacity curves have demonstrated how the maximum flexural
733 capacity is found for a slightly dispersed range of the imposed vertical displacement; on the
734 contrary, the catenary peak as well as the softening branch (including arching compressive
735 forces) are strongly affected by all the non-linear properties of materials, leading to a strong
736 dispersion of the results, according to the CoVs of the material properties, especially, of the
737 ultimate strain of reinforcement steel;
- 738 - the catenary activation starts earlier when the side face rebars are added in the design and, at
739 the same time, the resistances corresponding to the achievement of the energy equivalence are
740 related to lower imposed vertical displacements;
- 741 - in most of the aleatory capacity curves, the *frame 1* was not able to sustain the accidental loss
742 of the central supporting column. For these cases, the computation of the DAFs was not
743 possible. On the other hand, the *frame 2* and *frame 3* were characterized by mean values of
744 the DAFs equal to, respectively, 1.44 and 1.62;
- 745 - the failure probabilities are drastically reduced when the side face rebars are added into design,
746 reducing not only the mechanical effort at the concrete level but also the failure probabilities
747 in the longitudinal reinforcement, reaching values lower than 10^{-7} ;
- 748 - the frame designed according to standards (i.e., *frame 1*) presents failure probabilities of the
749 order of 10^{-3} to 10^{-1} in the lateral spans, resulting in a spread of the damage from the central
750 spans (directly affected by the column loss scenario) to the external spans (indirectly affected).
751 This is not the case for the two improved solutions (i.e., *frame 2* and *frame 3*), where the
752 lateral spans are characterised by failure probabilities lower than 10^{-7} ;
- 753 - the safety level in concrete in terms of β minimum is far below 1 for the *frame 1*, while it is
754 larger than 1 in the *frame 2* (i.e., 1.08) and *frame 3* (i.e., 1.23). Regarding the longitudinal
755 reinforcement, the β minimum is far below 1 for the *frame 1*, and equal to 5.11 and 7.45 for,

756 respectively, the *frame 2* and *frame 3*;
757 - the *frame 2* appears to be the leading solution since it presents quite the same safety level of
758 the *frame 3*, with reduced costs (i.e., additional reinforcement amount of 21% for *frame 2* and
759 36% for the *frame 3*).

760 As a general conclusion, the adopted robustness improvements always present the catenary
761 response ensuring a stable response of the structure and lead to values of β minimum higher than
762 1 within a structural reference period of 50 years. This result is absolutely relevant and on safe
763 side with respect to the suggestions of β minimum equal to 1.5 related to a structural reference
764 period of 1 year and assuming a reference period for variable loads of 1 year. Therefore,
765 especially, for cast in-situ RC structures having slabs with joists, the solution strategy of the *frame*
766 2 represents a good trade-off between principles of robustness, sustainability and safety in
767 accordance also with seismic design within a multi-risk analysis.

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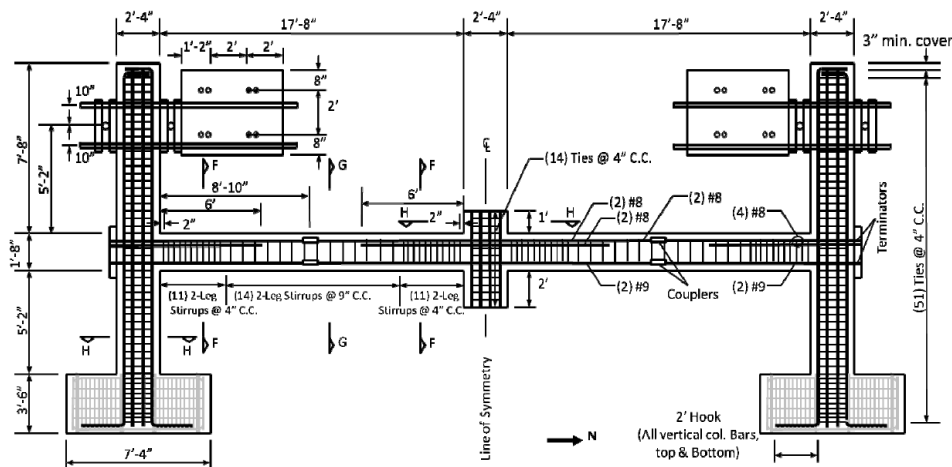
807 In line with the need of defining predictive numerical models in this work, a numerical model is
 808 calibrated to reproduce the results of an experimental test [68] on a beam-column subassembly
 809 extracted from a 10-story building designed in Seismic Category D according to ACI 318 Building
 810 Code [93]. The experimental test is selected since it has similar characteristics with respect to the
 811 case study of the present work. In detail, the sub-assembly (Fig. A1) is composed of two beams having
 812 a length of 5.5m and two columns of around 4.5m height. The beams have a cross-section of
 813 71.12x50.8cm² and are reinforced with 2 bars of 28.65mm diameter in the lower chord and 4 or 2
 814 bars of 25.4mm diameter in the upper one for, respectively, dissipative and non-dissipative area. The
 815 dissipative area has a length of 1.83m. The transverse reinforcement is composed of 2-leg stirrups
 816 having 12.7mm diameter with a step of 22.8cm or 10.16cm, respectively, in non-dissipative area and
 817 dissipative area. As for the columns, their cross-section is squared and have a size of 71.12cm, while
 818 their longitudinal reinforcement is made of 12 bars of 25.4mm and the 4-leg stirrups have a step of
 819 10.16mm and a diameter of 12.7mm.

820 Regarding the loading scheme, a displacement-controlled application of the load is assumed, by
 821 means of 4 hydraulic rams placed on top of the central column and imposing a rate of 25mm/minute.
 822 Four steel rods are fixed to the reaction wall to prevent any out-of-plane movement. Two concrete
 823 blocks are placed on the internal side of the upper part of the lateral columns and fixed to the reaction
 824 wall. Those concrete blocks are then connected to the column by means of 51mm steel plates
 825 constrained by four 32mm post-tensioning bars anchored to the columns themselves. The clamping
 826 force is of 2669kN. Those constraint conditions are representative of a roller-fixture (i.e., prevention
 827 of horizontal displacement). Finally, the longitudinal bars of the columns are fully embedded by two
 828 footings having width, thickness and height of 2.235x1.626x1.067m³, respectively.

829 As far as concern the material properties, a campaign led to the following mean values of the
 830 mechanical properties:

- 831 - concrete compressive of 32MPa;
- 832 - concrete tensile strength of 3.1MPa;
- 833 - top bars of the beams (with diameter of 25.4mm): f_y (steel yield strength)=476MPa, f_u (steel
 834 ultimate strength)=648MPa, ϵ_s (ultimate strain)=0.21;
- 835 - bottom bars of the beams (with diameter of 28.65mm): f_y =462MPa, f_u =641MPa, ϵ_s =0.18;
- 836 - bars of the columns (with diameter of 28.65mm): f_y =483MPa, f_u =690MPa, ϵ_s =0.17;
- 837 - stirrups (with diameter of 10.16mm): f_y =524MPa, f_u =710MPa, ϵ_s =0.14.

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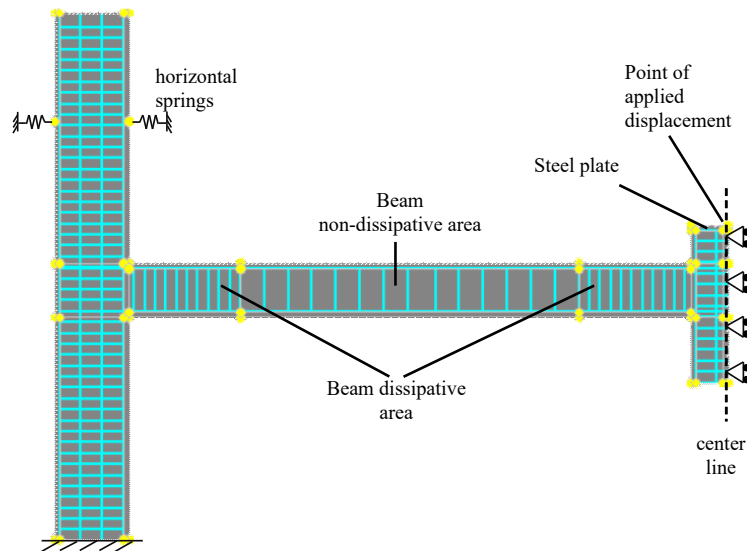


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840 Fig. A1 Geometrical characteristics and cross-sections of the experimental test in [68] (modified from [68]).

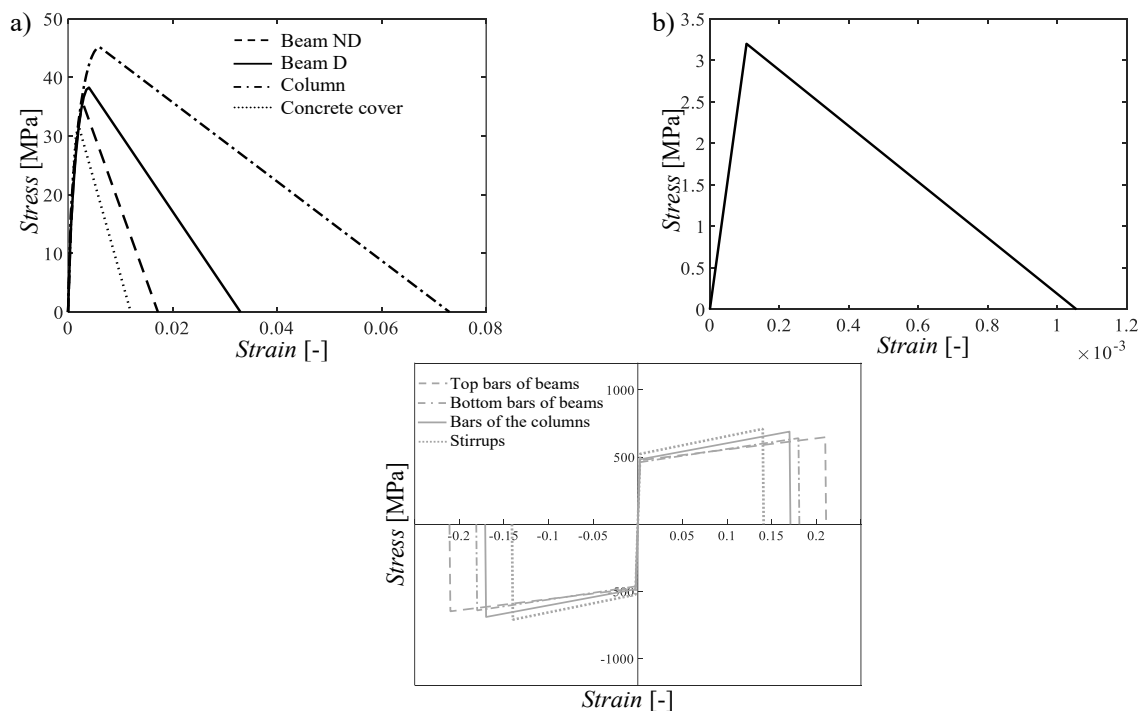
841 The experimental test is reproduced in Atena 2D [56], by taking advantage of the symmetry of the
 842 problem (Fig. A2), in fact, no horizontal movements are registered in correspondence of the center
 843 line during the test. Distinction is made between concrete core and concrete cover by considering

844 macro-elements made of plane stress quadrilateral isoparametric finite elements, based on linear
 845 polynomial interpolation, with 4 Gauss integration points. The thickness of the macro-elements is set
 846 equal to 0.7112m for all the structural elements (i.e., beam, columns and beam-column nodes), in line
 847 with the geometrical out-of-plane dimension. The mesh size varies in the range between 0.05m and
 848 0.10m, in line with a mesh-sensitivity analysis finalized to capture the experimental response in terms
 849 of load-deflection curve and horizontal-vertical displacement curve. The bars are considered as
 850 discrete elements.
 851



852
 853 Fig. A2 Representation of the FE modeling of half of the sub-assembly: joints, macro-elements and discrete bars.

854 The constraint conditions are the following (Fig. A2): horizontal movements in correspondence of
 855 the center line are avoided in order to respect the symmetry of the problem; fixed horizontal and
 856 vertical movement at the base of the columns in correspondence of half of the height of the footing;
 857 two springs (with a properly calibrated stiffness of almost 140N/mm) are placed at around 30cm from
 858 the top of the column to represent the roller-fixture.
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860 Fig. A3 Constitutive laws of materials assuming mean values of the mechanical properties for: a) concrete in
 861 compression; b) concrete in tension; c) steel bars in tension and in compression.

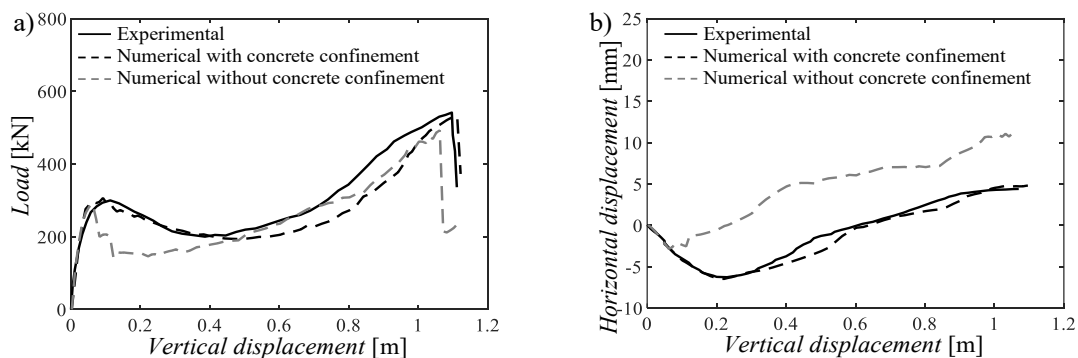
862 Concerning the concrete behaviour in compression (Fig. A3(a)), the behaviour before the peak stress
863 is derived by the formula in CEB-FIP Model Code 90 [63], while the post-peak response is described
864 by a linearly descending branch and it is based on a local strain softening according to Saatcioglu and
865 Razvi model [64], in order to account for the multiaxial state of stress due to the confinement of
866 concrete in compression. In fact, the beams are subjected to compressive forces during the initial
867 stages of the progressive collapse phenomena, which should be taken into account both in terms of
868 resistance and ductility. The reduction of compressive strength due to cracks is taken equal to 0.45,
869 representing the maximal strength reduction under large transverse strain, as suggested by [65] and
870 in line with the experimental results of 0.

871 The tensile concrete behaviour (Fig. A3(b)) has been modelled choosing a local strain tension
872 softening, accounting for the tension stiffening effect [66] through a linear post-peak branch up to the
873 zero strength in correspondence of a strain equal to $10f_{ct}/E_c$, being f_{ct} the tensile strength of concrete
874 and E_c the secant elastic modulus, computed according to [7]. Besides, the cracking process has been
875 reproduced using the “Smearred cracking” with the fixed crack direction model [65],[67].

876 As for the reinforcement steel (Fig. A3(c)), a bi-linear constitutive law in tension and in compression
877 has been adopted with hardening law. The reinforcement has been modelled with discrete bar
878 elements assuming a perfect bond with the surrounding concrete. It should be noted that, buckling of
879 reinforcement in the compressive zone is not considered since no buckling phenomena were declared
880 during the experimental test. This derives also from the seismic design with a larger number of
881 stirrups in the dissipative zones.

882 Within Atena 2D [56], the loading scheme consists in the application of the self-weight, followed by
883 an imposed vertical displacement at a rate of 1 cm in correspondence of the loading point (i.e., top of
884 the central column). In line with the experimental test, a steel plate of around 5 cm is placed on top
885 of the central column. The solution of the equations is solved by adopting a standard Newton-Raphson
886 iterative procedure considering 2500 maximum iterations and respecting contemporarily all the
887 following four tolerance criteria [58]: 1.0% for the norm of displacement error; from 1.0% to 2.5%
888 for the norm of residual force error; from 1.0% to 2.5% for the maximum error of residual forces;
889 1.0‰ for the out-of-balance energy error.

890 The comparison between the experimental and numerical capacity curves (Fig. A4(a)) and horizontal
891 versus vertical displacements (Fig. A4(b)) is shown. It should be underlined that not considering the
892 concrete confinement effects implies an underestimation of both the resistance and ductility of the
893 response with a fail in capturing the evolution of the horizontal displacements [68],[94].
894



895 Fig. A4 Comparison between numerical and experimental response: a) capacity curve; b) horizontal displacement at the
896 beam-column nodes.

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907

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910 *for existing structures*” and “*Action Group “Robustness”*” of the International Federation for
911 Structural Concrete (*fib*).

912

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914 “RELUIS- Costruzioni di Calcestruzzo Armato Gettate in Opera e Prefabbricate-WP11, Task 11.3
915 and 11.4”.

916

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