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1 **Fragility curves for risk-targeted seismic design maps**

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14

15 *Abstract*

16 Seismic design using maps based on ‘risk-targeting’ would lead to an annual probability of attaining  
17 or exceeding a certain damage state that is uniform over an entire territory. These maps are based on  
18 convolving seismic hazard curves from a standard probabilistic analysis with the derivative of fragility  
19 curves expressing the chance for a code-designed structure to attain or exceed a certain damage state  
20 given a level of input motion (e.g. peak ground acceleration, PGA). There are few published fragility  
21 curves for structures respecting the Eurocodes (ECs, principally EC8 for seismic design) that can be  
22 used for the development of risk-targeted design maps for Europe. In this article a set of fragility  
23 curves for a regular three-storey reinforced-concrete building designed using EC2 and EC8 for  
24 medium ductility and increasing levels of design acceleration ( $a_g$ ) is developed. These curves show  
25 that structures designed using EC8 against PGAs up to about  $1 \text{ m/s}^2$  have similar fragilities to those  
26 that respect only EC2 (although this conclusion may not hold for irregular buildings, other geometries  
27 or materials). From these curves, the probability of yielding for a structure subjected to a PGA equal  
28 to  $a_g$  varies between 0.14 ( $a_g=0.7 \text{ m/s}^2$ ) and 0.85 ( $a_g=3 \text{ m/s}^2$ ) whereas the probability of collapse for a  
29 structure subjected to a PGA equal to  $a_g$  varies between  $1.7 \times 10^{-7}$  ( $a_g=0.7 \text{ m/s}^2$ ) and  $1.0 \times 10^{-5}$  ( $a_g=3$   
30  $\text{m/s}^2$ ).

31 *Keywords:* seismic risk; fragility curves; Eurocode 8 (EC8); risk-targeting; reinforced concrete;  
32 Eurocode 2 (EC2)

33 *1. Introduction*

34 In the past decade the philosophy often known as risk-targeting has started to be employed to develop  
35 maps of design accelerations for use with seismic building codes (Luco et al., 2007; Luco, 2009). The  
36 aim of this method is to estimate seismic accelerations that, when used for design purposes, lead to a  
37 chosen level of risk (e.g. annual probability of collapse of  $10^{-5}$ ) that is uniform over an entire territory.  
38 This procedure relies on convolving hazard curves from a standard probabilistic seismic hazard  
39 assessment with fragility curves that are a function of the design acceleration (e.g. buildings designed  
40 against a higher acceleration have fragility curves shifted to the right). This approach has recently been

41 tested for mainland France by Douglas et al. (2013) with respect to the recent French seismic zonation  
42 to be used in conjunction with Eurocode 8 (EC8). Amongst other conclusions, they emphasized the  
43 lack of studies on how fragility curves change as a function of the design acceleration. This deficiency  
44 of knowledge is a current block on the adoption of risk-targeting for the next generation of seismic  
45 design codes.

46 The purpose of this article is to investigate the impact of the design acceleration on fragility curves by  
47 deriving such curves for a series of reinforced-concrete (RC) buildings designed using EC8 for  
48 different levels of earthquake loading. The next section presents the series of structures, emphasizing  
49 the parts of their design that are functions of the input acceleration. In Section 3 fragility curves are  
50 derived for these different structures using a standard technique based on dynamic nonlinear analysis  
51 using hundreds of input accelerograms. The influence of the design acceleration on the structure's  
52 fragility is discussed in the penultimate section along with how the results of this study influence the  
53 risk-targeting approach. The article ends with some brief conclusions and suggestions for future work.

## 54 *2. Eurocode 8-designed structures*

55 In the present study, a three storey (3 m high)-three bay (4 m long)-four frame (4 m long) RC structure  
56 is considered. This structure is, from our point of view, rather representative of modern European  
57 buildings. A regular structure is considered, so that simpler design procedures could be used (EC8  
58 4.2.3): a 2D model instead of a more complex 3D model and the lateral force method instead of the  
59 more complex modal approach. In addition, EC8 4.2.1 advises to design structures to be as regular as  
60 possible, because irregularities may greatly affect the structure's seismic resistance. The building is  
61 considered to be a typical residential or office building (importance class 2 in EC 8, common  
62 buildings). In this section details of the design of this structure to conform to Eurocode 2 (EC2) (CEN,  
63 2004a) and EC8 (CEN, 2004b) are provided.

64 The vulnerability of RC structures to earthquakes has been extensively studied in the past decades,  
65 with the development of reliable modelling tools, e.g. fibre based-models, with nonlinear behaviour of  
66 the materials composing the structural elements (Spacone et al., 1996). Nevertheless, they remain

67 complex structures, with many geometrical intricacies, that are often neglected. Thus, the  
68 reinforcement varies along the beams as a function of the flexural moment. The stirrup density along  
69 the beams changes as well, to cope with the high shear solicitation near the columns. The joint zones,  
70 between columns and beams, can also present certain complexities, particularly the joint zones  
71 involving the side columns. In these zones, the beam reinforcing bars (rebars) are sometimes bent so  
72 that they are in the same direction as the column rebars, resulting in complex 3D joint zones. A final  
73 example of complex detailing is the intricate integration of the structural elements in the slabs, which  
74 generates a complex 3D structure in which the slabs play a non-negligible role.

75 In the present study, it has been decided not to use highly-refined models for two reasons. Firstly, no  
76 robust models have yet been developed to take account of some of these aforementioned complexities  
77 on earthquake response. Secondly, inclusion of these details would have meant the conception of very  
78 complex numerical models, requiring much care and time, and probably suffering from considerable  
79 numerical instabilities. Also, design choices would have been required, which could have led to over-  
80 specific models, less adapted to describe the generic response of structures. It is interesting to note that  
81 using more sophisticated models does not always lead to more accurate results (T. Rossetto, personal  
82 communication, 2013).

### 83 *2.1. Design for different levels of acceleration*

84 The lateral force method detailed in EC8 4.3.3.2 is used to design the buildings, using a linear  
85 structural model and a reduced elastic spectrum. Models with reduced stiffness are considered (as  
86 stated in EC8 3.2.2.5), to roughly take into account the beneficial effects of the nonlinearities (in  
87 particular the ductile behaviour of the structural elements). Since the considered structures are regular,  
88 the procedures are carried out using simplified 2D models.

89 The studied structure is first loaded with the gravity actions as in design using EC2. The vertical loads  
90 applied to the model are taken as a combination of permanent and variable loads. They are arranged to  
91 produce the most critical conditions for the structure.

92 Then a lateral load is imposed to check the shear resistance of the building. Firstly, the spectral  
93 acceleration at the fundamental period of the structure is taken from the standard Type 1 design  
94 spectrum (EC8 3.2.2.5). The spectrum is computed for site class B, the most representative site class  
95 in Europe (Lemoine et al., 2012). It is fully constrained by the value of the required design (peak  
96 ground) acceleration (PGA, known in EC8 as  $a_g$ ), corresponding to the anchoring spectral acceleration  
97 at  $T=0$  s. The computed spectral acceleration, the mass layout and the ductility class, which is  
98 employed within EC8 to define the capacity of the structure to dissipate energy (behaviour factor), are  
99 then used to compute the lateral load pattern to be applied to the building. The accidental torsion  
100 associated with small irregularities of the structure is also considered, as an extra multiplicative factor  
101 on the force pattern (EC8 4.3.3.2.4). These horizontal loads are then imposed on the numerical model  
102 and the lateral resistance is assessed by carrying out various checks, at micro and macro scales.

103 Firstly, the overall stability of the structure is verified by checking that the inter-story drift of each  
104 floor is not too large (EC8 4.4.3.2). In this study, this criterion was always respected up to  $a_g$  of about  
105  $2 \text{ m/s}^2$ . The limited influence of P-Delta effects is also assessed (EC8 4.4.2.2).

106 In addition to these macro-scale criteria, the adapted behaviour of each structural element is assessed.  
107 Freedom is given in EC2 to the design engineer to determine moment and shear forces when assessing  
108 the individual member resistance of each element of the structure. In this study, it has been decided to  
109 follow the procedures detailed in the guide by Bond et al. (2006).

110 The moment resistance is assessed at several locations over the beam (e.g. at mid-span and at member  
111 ends) by considering the material properties and the section geometry. When, the moment resistance is  
112 found to be inadequate, the area of longitudinal reinforcement is modified. In all cases, the  
113 reinforcement areal ratio should stay within the range of variation specified in EC2 9.2.1.1. The  
114 spacing of the reinforcement bars in the element sections should also be carefully monitored to ensure  
115 a good anchoring of the bars (EC8 5.6.2.2), and also to ensure that no contact occurs between adjacent  
116 bars (EC2 8.2). When these spacing criteria cannot be fulfilled, the concrete section must be increased.

117 The criterion concerning the moment resistance was found to be the most critical for the beam design.  
118 The level of deflection (EC2 7.4) was also assessed, but it was sufficiently small once the  
119 aforementioned criterion was enforced. Finally, the beam shear resistance (EC2 6.2) has to be checked  
120 as well. This last condition affects only the stirrup spacing along the beam, which could be only  
121 accounted for in our models by changing the concrete confinement ratio longitudinally. Such a change  
122 would make the models much more complex for probably only a slight improvement in their accuracy.  
123 Hence, we did not consider this condition.

124 In the case of the columns, the moment resistance is similarly assessed [by computing the design  
125 moment with EC2 5.8.8.2 and then computing the minimum steel area based on Figures 9a to 9e in  
126 Bond et al. (2006)]. However, contrary to the case of beams, it appears that for the structure  
127 considered here, this criterion is not critical. In fact, the ruling criterion is the one detailed in EC8  
128 4.4.2.3, ensuring that the columns are stronger than the beams. A last condition that ensures an  
129 adapted behaviour of the column in the case of biaxial loading is also considered (EC2 5.8.9), but had  
130 no influence over the element design in the present study.

131 The following example helps to clarify the main steps used to design the structure to different levels of  
132 PGA. Consider that a building has already been designed for a given  $a_g$  and that its design has to be  
133 adjusted for a larger acceleration. Since the multiplicative factor on the horizontal load pattern is  
134 larger, a higher moment should be resisted in the beams and, hence, the steel area should be increased.  
135 Now suppose that the steel content in the beams was already critical, i.e. if the steel area is increased  
136 then the spacing criteria will no longer be respected. Consequently, the beam sections must be  
137 increased. The beams being then stronger, the dimensions of the columns should also be increased, to  
138 stay more resistant than the beams. Finally, as the whole building design is modified, all criteria  
139 should again be carefully checked and adjustments made.

140 Six versions of the structure for different  $a_g$  were designed. One building is designed with EC2 only  
141 (i.e.  $a_g=0\text{m/s}^2$ , although this is not strictly true because of the condition of “plastic hinges in beams” is  
142 imposed by EC8 and not EC2), and four other buildings for the design accelerations of the French

143 seismic zonation map (0.7, 1.1, 1.7 and 3.0 m/s<sup>2</sup>). Finally, a last structure is designed for  
144  $a_g = 2.3$  m/s<sup>2</sup>, to fill the gap between the two highest accelerations in the French code (these zones  
145 being the zones of moderate seismicity in southern France and high seismicity in the French Antilles,  
146 respectively). All the structures were designed assuming medium ductility.

147 The beams and columns sections of these structures are detailed in Table 1. The concrete section is the  
148 same for the structures designed with  $a_g$  up to 1.1 m/s<sup>2</sup>. For higher design accelerations, the sections  
149 have been increased because the spacing between the rebars was no longer sufficient. Standard values  
150 are used for the reinforcement bars: 6, 8, 10, 12, 16, 20, 25, 32 mm and so forth. Using standardized  
151 diameters leads to non-optimum designs because the necessary reinforcement areas cannot always be  
152 exactly obtained; a larger reinforcement area than required often needs to be used. A consequence is  
153 that two structures designed for different design accelerations can have the same sections, if the  
154 difference between the design levels is not large. This is what almost occurred with the structures  
155 designed for 0.7 and 1.1 m/s<sup>2</sup>, whose design differs only for the upper area of reinforcement of the  
156 beams.

157 Our designs can be checked by comparing them with those proposed by Fardis et al. (2012) for a two-  
158 storey RC frame structure whose geometry is relatively close to ours. No information about the  
159 reinforcement content of the structural elements is provided in their article, but the main dimensions of  
160 the sections presented in their Table 1 gives valuable information. The building designed for an  $a_g$  of  
161 3 m/s<sup>2</sup> has the same column and beam dimensions but slight differences can be noticed for the  
162 structures designed for lower accelerations. This is probably because, contrary to their study, we  
163 allowed our structures to have slender columns.

### 164 3. *Development of fragility curves*

165 Nonlinear time histories analyses are carried out, using the finite element software Opensees  
166 (McKenna et al., 2000), to assess the seismic vulnerability of each structure. The plane-frame  
167 structures are modelled with force-based beam-column elements, each structural component being  
168 discretized by four elements (Neuenhofer and Filippou, 1998). The finite elements are discretized by

169 three integration points over their lengths. The uniaxial material Concrete06 is used for the concrete  
170 and Steel01 for the reinforcement steel. Table 2 and Table 3 detail the properties of these materials. In  
171 addition to the material nonlinearities, geometric nonlinearities (P-Delta effects) are also taken into  
172 account, the corotational transformation being used for the columns.

173 Several methods have been used to derive fragility functions from dynamic time-history analysis. Here  
174 the approach called “linear regression on a cloud” by Baker (2007) is followed. In this technique a set  
175 of unscaled accelerograms (see below) are used as input. All the used accelerograms being  
176 independent, a great diversity in their spectral content is ensured. In this technique, a least-square  
177 optimisation leads to a robust relation between the maximum transient drift and the intensity measure  
178 (here, PGA), from which the fragility function parameters are finally obtained, assuming lognormal  
179 distributions. This method is particularly useful for higher damage states, for which the data are  
180 scarce, because the obtained relation could be extrapolated (although this is a source of additional  
181 uncertainty).

### 182 *3.1. Strong-motion records*

183 The structures are subjected to a set of unscaled accelerograms selected from the Internet Site for  
184 European Strong-motion Data (Ambraseys et al., 2004) and from the PEER NGA database (Chiou et  
185 al., 2008). Records have been firstly selected by considering magnitudes between 4.5 and 6.5 and  
186 source-to-site distances up to 100 km, which roughly covers the earthquake scenarios of most interest  
187 for much of Europe. A further selection based on the PGA has then been considered to remove many  
188 weak records that do not damage the structures. Figure 2 confirms that the distribution of PGA is  
189 relatively uniform. In total, 183 records are used. This is a sufficient number for deriving reliable  
190 fragility functions when using a regression-based method (Gehl et al., 2014).

### 191 *3.2. Damage thresholds*

192 To characterize the damage state at the end of the dynamic simulations, a standard inter-storey drift  
193 criterion at each floor is considered. It is common to compute the drift corresponding to each limit  
194 state by exploiting the results of a static push-over analysis. In this study, the standard method

195 proposed by Milutinovic and Trendafiloski (2003) was tested. In this approach, the drift thresholds for  
196 each damage state are derived from the yield drift, corresponding to the first occurrence of plasticity,  
197 and the ultimate drift, corresponding to a 15% drop in strength. Both values are read from the push-  
198 over curve. Figure 3 shows the evolution of the base shear, normalized by its peak value, with the  
199 drift, for all the considered structures. Since the curves have a similar shape, using the same drift  
200 threshold for all the structures is a valid simplification.

201 The structures show high ductilities, the drop of 15% of the maximum base-shear appearing after a  
202 drift of about 10%. Consequently, the use of the aforementioned method would lead to unusual and  
203 probably unrealistic drift thresholds. Consequently, a different approach was finally chosen: the  
204 generic thresholds proposed by Ghobarah (2004) for ductile moment-resisting frames. There is  
205 considerable uncertainty in the assessment of drift thresholds, particularly for the highest damage  
206 states (e.g. collapse), and other thresholds could be envisaged. As proposed by Crowley et al. (2011)  
207 and Gehl et al. (2013), results for only two damage states are presented: damage state D1, which is  
208 termed 'yield', and D4 and D5 that are merged into 'collapse'. Both these damage levels are  
209 interesting for the risk-targeting approach. Luco et al. (2007) uses the collapse damage state for their  
210 US risk-targeting approach, whereas Douglas et al. (2013), who focused on mainland France, pointed  
211 out that yielding might be more appropriate for countries of moderate seismic hazard.

### 212 *3.3. Results*

213 Lognormal cumulative distributions are assumed for the fragility curves. The obtained parameters are  
214 shown in Table 4 and the corresponding curves are plotted in Figure 4. As expected, the building  
215 designed for the highest accelerations are the least vulnerable. Nevertheless, the vulnerabilities of the  
216 buildings designed for  $a_g$  less than or equal to  $1.1 \text{ m/s}^2$  are almost the same. This is unexpected, and  
217 suggests that the building designed using only EC2 already presents adequate resistance against  
218 moderate earthquake loading. For these low accelerations, the additional criteria of EC8 are not very  
219 restrictive and the designed structures are not greatly modified from those obtained only considering  
220 EC2. It should be noted, however, that this finding may not hold for irregular structures, other

221 geometries or materials. The value of the standard deviation seems quite high for the three structures  
222 designed for moderate accelerations. As Douglas et al. (2013) and Chapter 21 of the ASCE Standard  
223 7–10 assumed standard deviations of 0.5 and 0.6, respectively, for fragility functions corresponding to  
224 the entire new building stock we would expect that fragility functions corresponding to a single  
225 geometry and building material should present lower standard deviations. In addition, we did not  
226 consider variability in the material (steel and concrete) properties, which could have increased the  
227 standard deviations of the fragility functions; although the contribution of this variability to the total  
228 standard deviation is usually much lower than the contribution of the strong-motion variability (e.g.  
229 Kwon and Elnashai, 2006).

#### 230 *3.4. Comparison with previous results*

231 To check the obtained fragility functions, a comparison with curves developed in previous studies for  
232 similar structures is made here. To determine the most appropriate functions for this comparison, the  
233 software Fragility Function Manager 2.0 developed during the FP7 Syner-G project (Silva et al., 2013)  
234 is used. A search considering only low-rise RC structures within this database was made. The most  
235 appropriate functions were then chosen manually. It is worth noting that many of the functions  
236 returned by the original search correspond to studies for Turkish buildings but only one of these  
237 functions is considered here to avoid being too geographically specific. Three fragility functions were  
238 finally chosen: two are based on numerical simulations (Kirçil and Polat, 2006; Kwon and Elnashai,  
239 2006) and one based on empirical data (Rossetto and Elnashai, 2003).

240 The first comparison is made for a three-storey RC frame designed using the 1975 Turkish seismic  
241 code, whose seismic vulnerability was assessed by Kirçil and Polat (2006), using Incremental  
242 Dynamic Analyses (IDA) and twelve artificial ground motions. The design acceleration is not given in  
243 the article, but the building is described as typical for Istanbul. The 1975 Turkish code does not  
244 directly define design accelerations but a “seismic zone coefficient” varying between 0.03 and 0.1  
245 depending on the zone. If this coefficient can be identified as the design acceleration, then the design  
246 level of the 1975 code corresponding to Istanbul can be obtained. As the city is in zone 2, the design

247 PGA is 0.08 g (0.8 m/s<sup>2</sup>), which seems quite low. In the 1997 code an effective ground acceleration  
248 coefficient is used; for Istanbul it is equal to 3.0 m/s<sup>2</sup> (Sezen et al., 2000). If this last observation is  
249 disregarded, the structure designed for  $a_g=0.7$  m/s<sup>2</sup> seems the best candidate for the comparison. The  
250 corresponding function is almost the same as that for  $a_g=0$  m/s<sup>2</sup>; see Figure 5 (for yield) and Figure 6  
251 (for collapse). The match seems reasonable given the differences between the two structures and  
252 codes. The fragility functions of the present study show, nevertheless, a larger standard deviation. As  
253 stated above, the variability of our results could be too large. Nevertheless, the lower variability of the  
254 compared functions could be due to the use of IDA and fewer ground-motion records.

255 The next comparison is with the functions of Kwon and Elnashai (2006), who quantify the fragility of  
256 a three-storey RC moment resisting frame designed only for gravity loads. The building is said to be  
257 representative of central northern Europe and USA and is consequently well adapted for this  
258 comparison. Kwon and Elnashai (2006) divide their ground-motion records into three datasets  
259 depending on their PGA/PGV ratio. According to their classification, most of our accelerograms  
260 (54%) present high PGA/PGV ratios, the rest of them being equally shared in the two remaining  
261 categories (24% present medium ratios and 22% low ratios). To obtain reference fragility functions  
262 more adapted to the particularities of our dataset, a weighted sum of the fragility functions of Kwon  
263 and Elnashai (2006) is first computed, and the resulting functions is then idealized by a lognormal  
264 function. The fragility curves corresponding to the three ratio categories (dashed blue lines), and the  
265 combined idealized functions (solid blue lines) are displayed in Figure 5 (for yield) and Figure 6 (for  
266 collapse). Once again, a good fit is noticed with our fragility functions (see curves  $a_g=0.0$  m/s<sup>2</sup>); the fit  
267 is particularly good for yield. The standard deviation of our fragility functions for collapse damage-  
268 state is again larger than in the functions of Kwon and Elnashai (2006).

269 A last comparison is carried out with functions obtained by a different procedure: empirical functions  
270 obtained by Rossetto and Elnashai (2003) from processing post-event assessments of over 340 000  
271 structures realized after 19 earthquakes. Contrary to the two previous comparisons, the fragility  
272 functions of Rossetto and Elnashai (2003) are not expressed as a lognormal cumulative distribution,  
273 but are given as list of values. Their damage levels “slight” and “extensive” are used for the

274 comparison with our curves. In Figure 5 (for yield) and Figure 6 (for collapse), it can be observed that  
275 the damage probabilities from Rossetto and Elnashai (2003) are much lower than all the other  
276 functions, e.g. the yield curve of Rossetto and Elnashai (2003) is similar to the collapse curves of the  
277 other studies. This problem with their empirical curves is noted by Rossetto and Elnashai (2005) who,  
278 after comparing their empirical and analytical curves (from their 2005 article), wrote: ‘these  
279 observations give rise to substantial doubt as regards the reliability of observation-based vulnerability  
280 functions and confirm the importance of analytical methods for the generation of fragility curves’.

281 In conclusion, two of the fragility functions considered here fit the functions obtained in the present  
282 study reasonably well. Nevertheless, the standard deviations of these functions, particularly those for  
283 the collapse damage state, are lower than the ones obtained in this study. This discrepancy might be  
284 explained by differences in the methods used to obtain the fragility functions. The higher spectral  
285 variability of the records used in this study could explain part of the observed differences. On the other  
286 hand, the functions presented in Rossetto and Elnashai (2003) show a poor match with ours. This  
287 highlights the great uncertainty in fragility functions that can be found in the literature.

#### 288 4. Conclusions

289 Within the risk-targeting approach it is assumed that the probability of collapse when the observed  
290 PGA equals  $a_g$  is constant for buildings designed against different acceleration levels. This conjecture  
291 is evaluated in Table 5, where the probabilities of yield and collapse at  $a_g$  are listed for all the  
292 structures studied in the present article. The probability of yielding for a structure subjected to a PGA  
293 equal to  $a_g$  varies between 0.14 ( $a_g=0.7 \text{ m/s}^2$ ) and 0.85 ( $a_g=3 \text{ m/s}^2$ ) whereas the probability of collapse  
294 for a structure subjected to a PGA equal to  $a_g$  varies between  $1.7 \times 10^{-7}$  ( $a_g=0.7 \text{ m/s}^2$ ) and  $1.0 \times 10^{-5}$   
295 ( $a_g=3 \text{ m/s}^2$ ). In Douglas et al. (2013), a value of  $10^{-5}$  is proposed for the probability of collapse given  
296 a PGA equal to the design value when conducting risk targeting. This value seems roughly suitable for  
297 all the structures, except for the one designed for  $a_g=0.7 \text{ m/s}^2$ . This exception is related to the similar  
298 vulnerabilities presented by structures designed for  $a_g \leq 1.1 \text{ m/s}^2$ . It suggests that designing regular  
299 structures against low accelerations is not useful when the overall design is controlled by modern

300 (non-seismic) codes, such as EC2. Douglas et al. (2013) suggested the idea of targeting the ‘yield’  
301 damage state rather than ‘collapse’ in areas of moderate seismicity. Again, Table 5 seems to support  
302 this idea. A probability of yielding when a structure is subjected to its design PGA of 40% to 80%  
303 could be considered for that purpose.

304 In conclusion, the present study confirms the hypotheses made in the risk-targeting approach. This is  
305 an important stage toward the generation of new seismic design maps for Europe. Nevertheless the  
306 limits of the study should be carefully considered. Specifically, only a single characteristic structure is  
307 considered here; the next step could be to generalize the developed method for other structural  
308 geometries and types. In particular, it would be interesting to see if the structural type has a big impact  
309 on the fragility curve. If it does, then the current risk-targeting approach is not appropriate but would  
310 require a series of maps to be produced per building type.

311 One could also take advantage of the designed structures to carry out economic studies on the  
312 additional cost of seismic reinforcement. This could lead to design using a cost-benefit point of view,  
313 which could be useful in improving the efficiency of design codes.

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320

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380

381 Table 1. Geometry of the sections designed with EC2 and EC8.

	Beams			Columns	
$a_g$	dim (H×B)	upper reinf.	lower reinf.	dim (H×B)	reinf.
$m/s^2$	m × m	nb × mm (mm <sup>2</sup> )		m × m	nb × mm (mm <sup>2</sup> )
<b>0.0</b>	0.35 × 0.30	6 × 16 (1206)	5 × 10 (393)	0.35 × 0.35	5 × 20 (1571)
<b>0.7</b>	0.35 × 0.30	3 × 25 (1473)	6 × 10 (471)	0.35 × 0.35	4 × 25 (1963)
<b>1.1</b>	0.35 × 0.30	5 × 20 (1571)	5 × 12 (565)	0.35 × 0.35	4 × 25 (1963)
<b>1.7</b>	0.35 × 0.30	6 × 20 (1885)	4 × 16 (804)	0.40 × 0.40	4 × 25 (1963)
<b>2.3</b>	0.35 × 0.30	4 × 25 (1963)	3 × 20 (942)	0.45 × 0.45	6 × 25 (2945)
<b>3.0</b>	0.40 × 0.35	3 × 32 (2413)	4 × 16 (804)	0.45 × 0.45	6 × 25 (2945)

382

383 Table 2. Reinforcement steel properties considered (Steel01), where  $F_y$  is the yield strength,  $E_0$  the  
 384 initial young modulus and b the strain-hardening ratio.

	value	unit
$F_y$	575	MPa
$E_0$	$2.00 \times 10^5$	MPa
<b>b</b>	0.001	

385

386

387 Table 3. Concrete properties considered (Concrete06), where  $f_c$  is the compressive strength,  $e_0$  the  
 388 strain at compressive strength,  $E_c$  the Young's modulus and  $f_{cr}$  the tensile strength, which is set to zero  
 389 since it has little influence on the global behaviour of the element and it generates numerical  
 390 instabilities. The other numerical properties of the model (shape factors) are not detailed, but can be  
 391 easily deduced from the parameters shown.

	value	unit
$f_c$ (conf.)	-38.9	MPa
$e_0$ (conf.)	$-4.14 \times 10^{-3}$	
$f_c$ (unconf.)	-33.0	MPa
$e_0$ (unconf.)	$-2.07 \times 10^{-3}$	
$E_c$	$3.15 \times 10^4$	MPa
$f_{cr}$	0	MPa

392

393 Table 4. Median ( $\mu$ ) and standard deviation ( $\sigma$ ) of the lognormal fragility curves for yielding and  
 394 collapse.

$a_g$ (m/s <sup>2</sup> )	yielding		collapse	
	$\mu$ (m/s <sup>2</sup> )	$\sigma$	$\mu$ (m/s <sup>2</sup> )	$\sigma$
<b>0.0</b>	1.12	0.51	8.79	0.51
<b>0.7</b>	1.20	0.50	8.77	0.50
<b>1.1</b>	1.26	0.49	8.78	0.49
<b>1.7</b>	1.53	0.44	10.07	0.44
<b>2.3</b>	1.72	0.41	11.24	0.41
<b>3.0</b>	2.03	0.37	14.59	0.37

395

396

397 Table 5. Probability of damage at  $\text{PGA}=\text{a}_g$  for different values of  $\text{a}_g$ .

$\text{a}_g$ ( $\text{m/s}^2$ )	yield	collapse
0.7	0.139	$1.73 \times 10^{-7}$
1.1	0.392	$9.80 \times 10^{-6}$
1.7	0.592	$2.58 \times 10^{-5}$
2.3	0.763	$5.23 \times 10^{-5}$
3.0	0.854	$1.04 \times 10^{-5}$

398



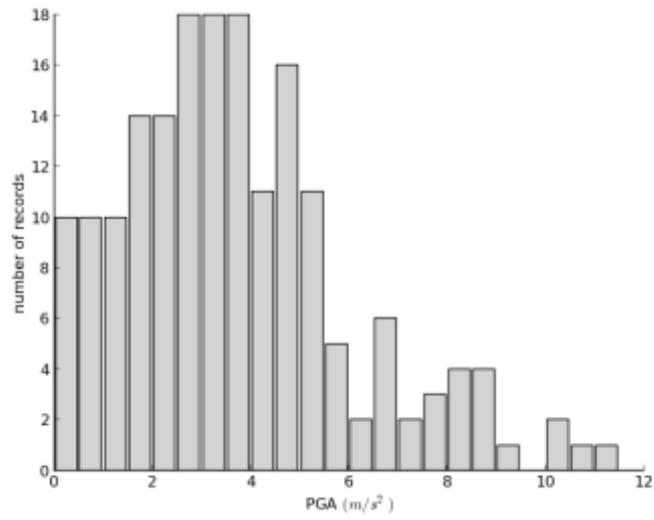
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400 Figure 1. 3D geometry of the structure (a three storey-three bay-four frame building) considered here.

401 The building being regular, a 2D representation of the structure (one frame) is used in the design and

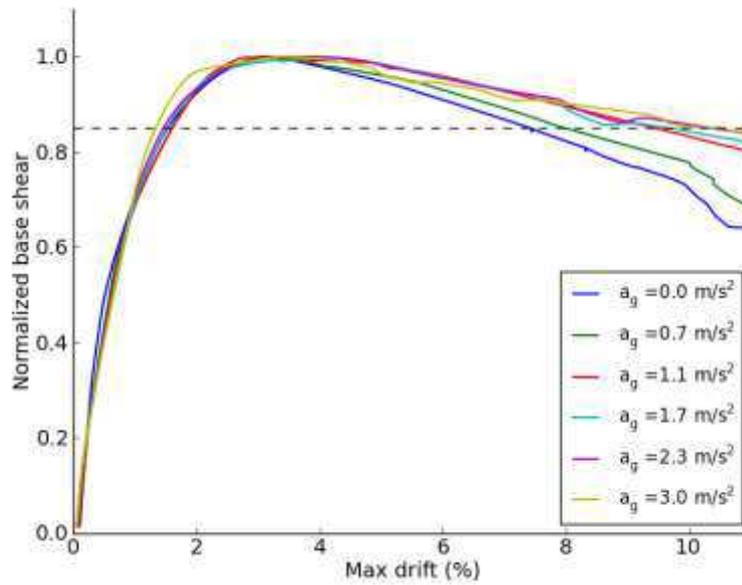
402 the dynamic analyses process.

403



404

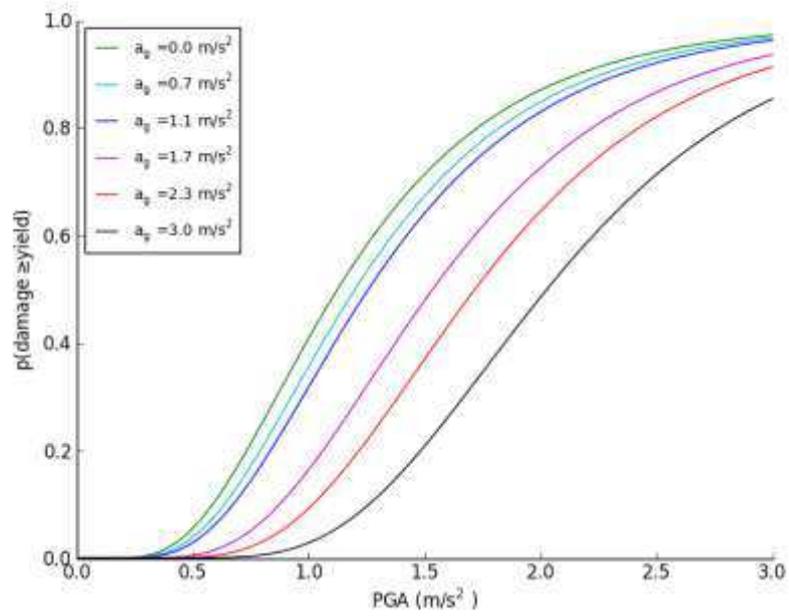
405 Figure 2. Histogram showing the PGAs for the set of accelerograms.



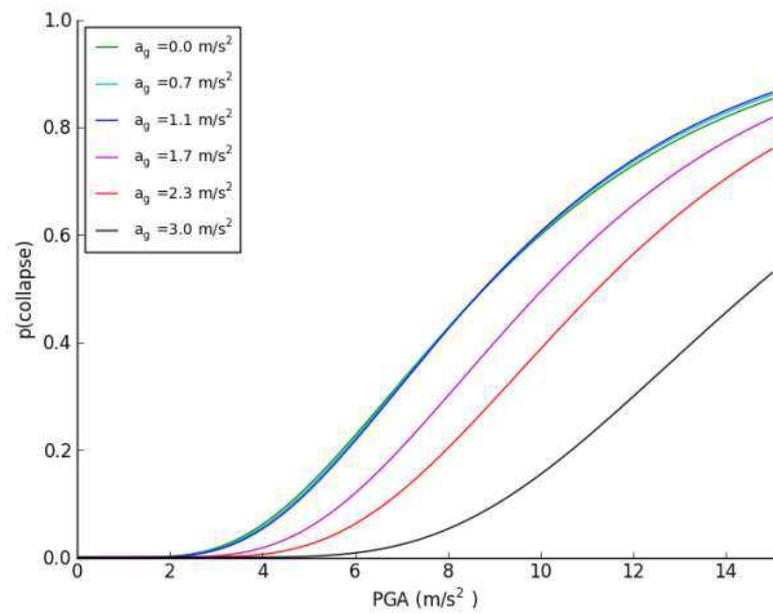
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407 Figure 3 Normalised push-over curve for the six versions of the structure. The 15% drop, which gives  
408 an idea of the structures' ductility, is shown by the dashed line.

409



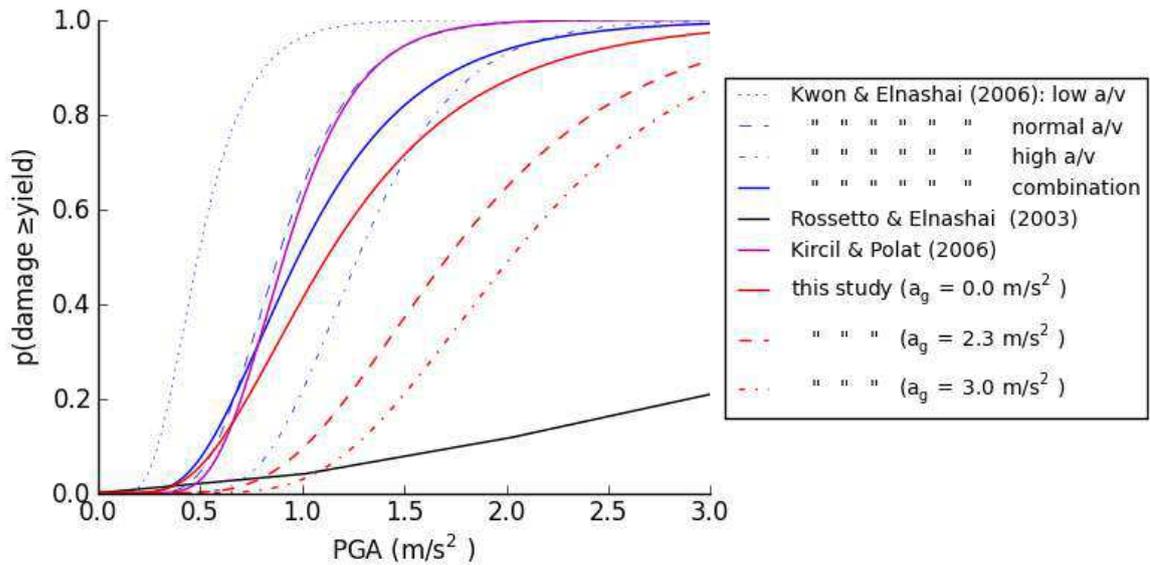
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411

412 Figure 4. Fragility curves of the six designed structures: (a) yield and (b) collapse. Note that the curves

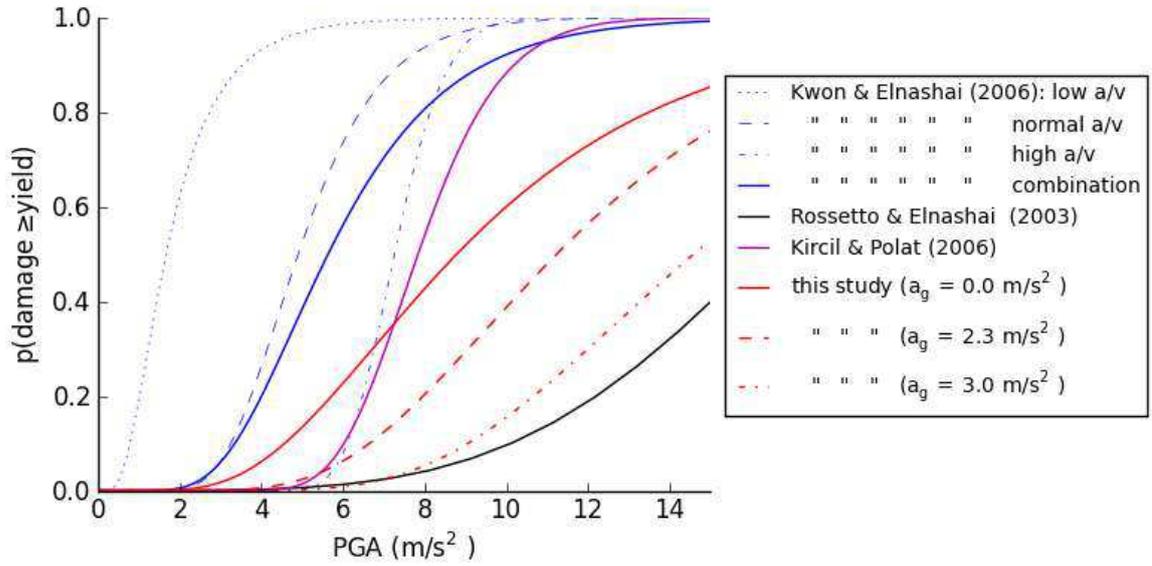
413 for  $a_g \leq 1.1 \text{ m/s}^2$  are almost the same.



414

415 Figure 5 : Comparison of the obtained fragility functions with selected fragility curves from the  
 416 literature for the yield damage state. In Kwon & Elnashai (2006) the building is designed only for  
 417 gravity loads so the functions with  $a_g=0 \text{ m/s}^2$  seem the best candidate for comparison; for Kirçil &  
 418 Polat (2006) we determined that the best candidate for comparison is the function with  $a_g=0.7 \text{ m/s}^2$ ,  
 419 which is almost the same as the function for  $a_g=0 \text{ m/s}^2$ . Finally, the curves of Rossetto and Elnashai  
 420 (2003) apply for the whole RC building stock so all the displayed functions are eligible for  
 421 comparison.

422



423

424 Figure 6 : Comparison of the obtained fragility functions with selected fragility curves from the  
425 literature for the collapse damage state.