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A model to estimate the lifetime of structures located in seismically active regions

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Abstract

Understanding and modeling the evolution of structural systems throughout time, especially by considering its cumulative damage, is a key point for risk decision making. This concept is of extreme importance for design criteria definition, operational policies, robust life-cycle cost estimations and real time-dependent vulnerability assessment. In particular, an important objective of these type of analyses, is the estimation of the structure lifetime distribution and its corresponding mean time to failure (MTTF). However, analytic solutions of these two main concepts are not an easy task. Therefore, it is necessary to develop new approaches that can be used to better approximate the lifetime of structures in an accurate and useful way for engineering purposes. Within this context, the objective of this research work is to present a methodology to estimate the lifetime distribution of structures, and the derived properties of a structural system located in a seismic region.

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Mainshocks are modelled as a homogeneous Poisson process with constant mean rate of occurrence. The model, assumes that the deterioration of a given structure is mainly controlled by the cumulative damage generated by a set of consecutive earthquake events in a certain period of time. This approach can be easily implemented and provides a good estimation of the lifetime distribution of a structure, without venturing into the mathematical complexities of an analytic solution but retaining the main features of the system performance. Finally, an illustrative practical case at a real site is provided.

Keywords: Structure lifetime, Mean-Time-To-Failure, Earthquake, cumulative damage, shock-based degradation

1. Introduction

The Eurocode [1] defines the *design working life* of a structure as “. . . the period for which it [a structure] is to be used for its intended purpose with anticipated maintenance but without major repair being necessary.” Similar definitions can be found in other standards such as the American Society for Civil Engineers (ASCE) [2]; and the British Standards BSI [3]. In engineering practice it is common to establish specific minimum structural lifetime values; for example, the Eurocode defines a design working life of 50 years for buildings and 100 years for bridges [1]. However, evidence has proven that infrastructure last longer, and some times far beyond than the values specified in technical standards. Arbitrary lifetime values set by design standards are, in practice, used as input to determine other important design parameters related mostly with the definition of the demand [1, 3, 4]. For instance,

14 heavy traffic on bridges or demands imposed by extreme events (e.g., seismic
15 loading, hurricane gust winds). They are also used for considering material
16 property deterioration, or as a reference to make life-cycle costing and define
17 maintenance strategies. In summary, the structural lifetime is not explicitly
18 set as a design objective *per-se*. This "unconscious" decision transfer the
19 responsibility of maintaining and handling possible future failures to future
20 generations.

21 Mathematically, the *lifetime* of a structure is the time that takes for
22 a structural performance indicator to reach a predefined threshold, which
23 separates acceptable from unacceptable behavior. Common performance in-
24 dicators include, for example, the structural capacity or a reliability measure
25 (e.g., reliability index β , deformation threshold). Thresholds are defined de-
26 pending on the purpose of the structure and are usually referred to as *limit*
27 *states*. Two limit states are of particular interest in engineering design: ser-
28 viceability and ultimate limit states. Structures can reach these performance
29 thresholds as a result of degradation processes, cumulative damage or due to
30 the occurrence of sudden disproportionate large events (catastrophic event).

31 Since the structural performance and the occurrence of external events
32 cannot be fully predicted; the processes leading to failure (i.e., surpassing
33 a limit state) are highly uncertain. Thus, the structural lifetime, which
34 is a random variable, X , needs to be characterized through a probability
35 distribution, $F_X(t)$; where the expected value, $\mathbb{E}(X)$, is referred to as the
36 Mean Time To failure (MTTF) and it is an important parameter for decision
37 making. It is used to define important actions that go from the selection of
38 inspection and intervention times to the abandonment or replacement of the

39 project [5, 6, 7].

40 In order to estimate the lifetime distribution of a structure, it is nec-
41 essary to build a model capable of predicting the structural performance
42 and cumulative damage over time. The structural performance depends on
43 both external (e.g., environmental demands) and internal aspects (e.g., ma-
44 terial degradation) and their relationship with the structural features. As
45 stated by Ranjkesh et al. [8], most degradation models are either of the
46 deterioration-based [e.g. 9, 10, 11] or the shock-based type [e.g. 12, 13]. The
47 deterioration-based degrades due to internal factors such as ageing, overload,
48 wear, or corrosion. The second type involves systems subjected to a sequence
49 of shocks or external factors that can be modelled by a stochastic point pro-
50 cess. The methodology proposed in this work is of shock-based type without
51 the possibility of any type of repair during the degradation process.

52 For example, in high seismicity regions, the structural lifetime is mostly
53 controlled by the damage accumulated as a result of earthquakes that occur
54 randomly in a particular seismic area source or fault over a certain period of
55 time. The effect of earthquakes can be modeled as shock-based deterioration
56 process; i.e., events that cause sudden changes (damage) on the structural
57 properties (described by any performance indicators) over time [7].

58 In the literature, several models have been developed to characterize the
59 cumulative structural deterioration as a result of earthquake events [14, 15,
60 16, 17, 18, 19]. Modeling degradation, in general, has two main difficulties: (i)
61 degradation is an unobservable (latent) process [20], which results from the
62 complex combination of different mechanisms (e.g., fatigue, corrosion, etc.);
63 and (ii) there is not enough data to characterize statistically the processes

64 and the variables involved. Furthermore, analytical models become highly
65 complex very quickly and is difficult to find a close form solution [7]. The
66 long-expected lifetime of structures and infrastructure, added to the com-
67 plexities that involve modeling degradation, have put aside the problem of
68 estimating the lifetime in practical design. Thus, in addition to the already
69 existing mathematical tools [7], it is necessary to develop new approaches
70 that can be used to approximate the structure's lifetime in a way that is
71 useful in engineering practice. These tools might have an important impact
72 in defining design criteria, operational policies, and making more robust life-
73 cycle cost estimations.

74 The objective of this paper is to present a simplified and practical method
75 to estimate the lifetime (and the MTTF) of structures located in active seis-
76 mic regions using a shock-based degradation model. The model proposed
77 here describes the process of damage accumulation as the structure is sub-
78 jected to a sequence of earthquakes during a certain period of time; and
79 allows computing the distribution and the MTTF without deepening into
80 the mathematical complexities that are required for an analytical solution,
81 but preserving the main features of the system performance. The lifetime
82 distribution can be computed for various limit states, in accordance to the
83 decision makers needs. The model presented in this paper does not take into
84 account additional degradation mechanisms (e.g., fatigue, corrosion), nor any
85 kind of retrofiring, but they can be further incorporated in the methodology
86 if required. The earthquake recurrence model used follows a homogeneous
87 Poisson process (HPP), nevertheless any other recurrence model, such as
88 clusters, foreshocks and aftershocks can be easily used by modifying the syn-

89 thetic earthquake catalogue number of events and distribution in time, since
90 the method is completely versatile with respect to the recurrence model and
91 the type of structure. Finally, the proposed approach combines analytical
92 models of the structure and the local seismic hazard.

93 This paper is structured as follows. First, it presents a definition of the
94 structural lifetime and brief review of the shock-based degradation process;
95 some of the most common models are mentioned and discussed. This discus-
96 sion is used as the basis for justifying the proposed simplified model. Then,
97 the basic assumptions and the step-by-step description of the approach is
98 presented. Finally, in order to show the applicability and the results of the
99 model, it is illustrated with a case study of a concrete structure located at
100 the Euroseistes in Greece. As a result, the lifetime distributions for various
101 limit states are shown and discussed.

102 **2. Evaluation of the structural performance over time**

103 *2.1. Structural lifetime*

104 Consider a structure that is placed in operation at time $t = 0$ and whose
105 condition at time $t > 0$ is denoted by $V(t)$, which is a random variable that
106 takes values in the set of positive real numbers. It is assumed that $V(0)$
107 is deterministic and represents the structural initial condition. Note that
108 the random variable $V(t)$ characterizes the evolution in time of a specific
109 performance indicator. Furthermore, let's define the random variable $X(t)$,
110 with $X(0) = 0$ (almost surely), as the accumulated deterioration until time
111 t ; with $X(t)$ a continuous non-decreasing function in t . Then, under the
112 assumption that there is no maintenance, or any other intervention that

113 modifies $V(t)$ before failure, the system condition at time t can be computed
 114 as [7]:

$$V(t) = \max \{V(0) - X(t), 0\} \quad (1)$$

115 Note that the sets $\{V(t)\}_{t \geq 0}$ and $\{X(t)\}_{t \geq 0}$, which consists of the ran-
 116 dom variables describing the condition and deterioration for all times $t \geq 0$,
 117 constitute stochastic processes with continuous state space and continuous
 118 time. System failure occurs when the condition of the system falls bellow
 119 a predefined threshold s^* , with $0 \leq s^* \leq V(0)$, that represents any service-
 120 ability, operational or ultimate limit. Then, the system lifetime L is also a
 121 random variable given by:

$$L = \inf \{t \geq 0 : V(t) \leq s^*\} = \inf \{t \geq 0 : X(t) \geq V(0) - s^*\}. \quad (2)$$

122 Two important quantities can be derived from the lifetime distribution:
 123 i) the mean time to failure (MTTF); and ii) the reliability function. The
 124 MTTF= $\mathbb{E}[L]$, where $\mathbb{E}[\cdot]$ is the expectation operator. Besides, the reliability
 125 is defined as the probability that the system accomplishes its intended func-
 126 tion in a designated period of time t (e.g., at the system's mission time t_m).
 127 It is the probability that the condition $V(t)$ is greater than the threshold s^* ;
 128 i.e.,

$$R(t) = P(V(t) \geq s^*) = P(X(t) \leq V(0) - s^*) \quad (3)$$

129 Therefore, if the derivative of $R(t)$ exists, the density function $f(t)$ of the
 130 lifetime is given by [7]:

$$f(t) = -\frac{d}{dt}R(t) = -\frac{d}{dt}P(X(t) \leq V(0) - s^*), \quad (4)$$

131 *2.2. Structural degradation caused by shocks*

132 Degradation models describe the damage accumulation in any system as
 133 a result of its dynamic interaction with external demands or internal changes
 134 of the structure. Frequently, degradation models are divided into two ba-
 135 sic mechanisms: i) progressive degradation, in which the system condition
 136 (system properties) decreases continuously over the time (e.g., aging, wear
 137 or corrosion); and ii) shock-based degradation, where the system condition
 138 is subject to sudden changes (decays) at specific points in time (e.g., struc-
 139 tures subjected to earthquakes or blasts) [21]. In this paper we will focus
 140 only on the latter; details of progressive degradation models can be found in
 141 Sánchez-Silva and Klutke [7].

142 Shock-based degradation models consider the cumulative effect of sudden
 143 (and independent) events that remove finite amounts of the system's capacity,
 144 in the form of jumps or shocks, at discrete points in time [22, 23, 21, 24].
 145 Shock-based degradation can be described by two stochastic processes: i) the
 146 inter-arrival times $\{T_i\}_{i \geq 1}$ (with T_i the random variable of the time between
 147 the $(i - 1)^{th}$ and i^{th} shock); and ii) shock sizes $\{Y_i\}_{i \geq 1}$, which represent the
 148 damage of each shock to the system. If after each shock damage accumulates
 149 and no maintenance is performed, the total deterioration by time t , $D(t)$,
 150 can be computed as:

$$D(t) = \sum_{i=1}^{N(t)} Y_i, \quad (5)$$

151 where $N(t)$ is a random variable describing the number of shocks by time t .
 152 Hence, the system's capacity by t can be expressed as:

$$V(t) = V(0) - D(t) = V(0) - \sum_{i=1}^{N(t)} Y_i, \quad (6)$$

153 and the reliability function becomes:

$$R(t, s^*) = P(D(t) < s^*) = P\left(\sum_{i=1}^{N(t)} Y_i < s^*\right) \quad (7)$$

154 When computing the reliability function, it is important to distinguish
155 between two cases: i) models where degradation caused by one shock depends
156 on the history of the process up to that point; and ii) those where the shock
157 damages accumulates in an independent manner. The proposed model is
158 based on the second assumption, which is evaluated indirectly through sim-
159 ulations. The discussion on the details of these models is beyond the scope
160 of this paper but can be found, for instance, in Sánchez-Silva and Klutke [7].

161 An analytical solution to compute the reliability and the lifetime distri-
162 bution, involves the evaluation of the distribution of the sum of n random
163 variables (i.e., shock sizes and inter-arrival times), which requires the evalu-
164 ation of n -fold convolutions, and to take the limit $n \rightarrow \infty$. In general this is
165 a difficult problem and, with few exceptions, Monte Carlo simulation is the
166 only option to obtain a solution. Among the analytical solutions, the most
167 common models are the compound Poisson Process (CPP) [7], and the use
168 of Phase-type distributions [19].

169 **3. Simplified methodology for lifetime estimation**

170 *3.1. Basic assumptions*

171 The model is based on the following assumptions:

- 172 • All ground motions included in the analysis (i.e., actual or simulated)
173 keep the basic seismic characteristics (i.e., magnitude range, distance

174 range, site amplification, spacial distribution, among others) than those
175 recorded in the area where the structure is located.

176 • The structural performance is characterized by performing non-linear
177 dynamic analyses of a structural model under a suite of ground motion
178 records. The used numerical model allows to take into account a state-
179 dependent seismic response, or in other words, the damage caused by
180 previous earthquakes.

181 • The damage caused by an earthquake event, i.e. the engineering de-
182 mand parameter (EDP), could be quantified in buildings with both
183 peak and residual lateral inter-story drift demands. In this work for
184 the sake of simplicity the cumulated induced maximum top displace-
185 ment will be used. Nevertheless any other damage indicators are also
186 suitable to be used under the proposed methodology [e.g 25, 26, 27,
187 among others].

188 • The structure is not intervened (e.g., retrofitted) after it has experience
189 a ground motion.

190 The proposed model is highly flexible and can be adapted to the specific
191 conditions of the problem at hand. In the first assumption (i.e., selection
192 of seismic records) the main restriction is that the seismic nature of the
193 events selected should resemble as much as possible the seismicity of the
194 area. In the third assumption, other structural performance models and
195 damage measures can be implemented if needed. The last restriction cannot
196 be modified because it will change the way in which the lifetime is estimated.

197 *3.2. Overall description of the model*

198 A conceptual flow diagram of the method is presented in Figure 1. Con-
199 sider a well known structural system located in an area whose seismic activity
200 is well documented and characterized (i.e., magnitude of events M_w , recur-
201 rence, duration, peak ground acceleration, frequency content, etc.). Based
202 on existing seismic information, K trains of N earthquake records are con-
203 structed. The selection of events and their order of occurrence are selected
204 based on a previous statistical characterization and considering the local re-
205 currence model at the site of interest. The structure is then subjected to
206 every train of events until it exceeds a damage state (i.e., failure); or it has
207 been subjected to all N events in the train (no failure). Then, after subject-
208 ing the structure to a large number of earthquake trains, the probability of
209 reaching a particular damage state can be computed; and since the occur-
210 rence rate of earthquake events is known, the time to failure can be easily
211 evaluated as well. The details of the process is presented in the following
212 sections.

213 *3.3. Seismic hazard characterization*

214 The first step consists of determining the seismicity of the site in which
215 the building is located or where it will be built. The three main aspects to
216 be determined are: i) the geographical location of the building; ii) the site
217 characteristics (i.e., soil class, the average shear-wave velocity (V_s) for the
218 upper 30 m depth ($V_{S,30}$), among others); and iii) the magnitude-frequency
219 distribution and the special distribution of earthquakes. Once the previous
220 aspects are defined, there are two types of methods to perform seismic risk

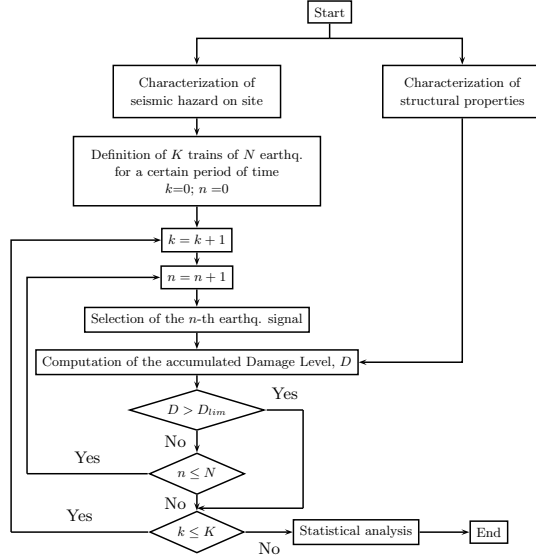


Figure 1: Flow diagram of the simplified method to estimate the lifetime of structures in earthquake prone areas.

221 assessment: Non-site-specific and site-specific approaches. The former con-
 222 siders multiple time histories from multiple sites, while the later uses only
 223 site-specific time histories based on on local parameters, local recurrence of
 224 earthquakes (or number of expected earthquakes at a given period of time).
 225 Now, if sufficient amount of data is available at the site, site-specific ap-
 226 proaches better estimate the local seismic hazard and its local variability,
 227 hence, a better estimation of the cumulative damage will be retrieve. Nev-
 228 ertheless, it is often common a lack of local data at the site of interest, then,
 229 a non-site specific approach might be required.

230 Once the main aspects that characterize the local seismic hazard are
 231 defined, it is possible to proceed with the selection of time history. Two
 232 main strategies for this purpose can be used: i) Spectral matching using

233 real accelerograms; or ii) stochastic time histories generation using local site
234 conditions. Even though both methods should be applied considering both
235 local magnitude-distance disaggregation and magnitude-frequency distribu-
236 tion (better known as truncated Gutenberg-Richter model) at the site of in-
237 terest, the later will also required extra information for the source, path and
238 site synthetic models (e.g., stress drop distribution, S and P waves velocity
239 profiles, high frequency attenuation factor (κ), among others, see Aristizábal
240 et al. [28] for further information).

241 There exist also two ways of generating the accelerograms: i) compatible
242 to the uniform hazard spectrum (UHS), or ii) compatible to the design spec-
243 tra of a given seismic design code. For case i), the authors believe that the
244 selection of accelerograms should be done based on the UHS rather than on
245 design spectra. The reason for this preference choice, is because even though
246 both are envelops of the local hazard, the UHS provides the same probabili-
247 ty of exceeding a certain ground motion level at a given site for different
248 spectral periods, while the design spectra is just a simplified and truncated
249 way of representing the variability of the local response spectra at different
250 sites for a given site class and for different spectral periods. The accelero-
251 grams can be fitted to a rock or a soil spectra, if the former is used, then a
252 soil transfer function needs to be apply to consider for local soil conditions.
253 On the other hand, concerning the stochastic time histories using local site
254 conditions, there exist several different methods to generate the synthetic
255 waveforms [29, 30, 31, 32, among others] where the magnitude and distance
256 ranges can be predefined.

257 *3.4. Definition of ground motions sequences*

258 During the structure's lifetime, damage accumulates as a result of a se-
 259 quence of events that are distributed over time. Let's assume that $S =$
 260 $\{S_{t_1}, S_{t_2}, \dots, S_{t_n}\}$ is a set of n events that occur at times t_i in a sequential
 261 manner i.e., $t_{i-1} < t_i$. Then, the total damage, $D(z)$, after the z -th event
 262 will be:

$$D(z) = \sum_{i=0}^z \Delta D(\Delta \delta_{M_i}) \quad (8)$$

263 where $\Delta \delta_{M_i} = \delta_{M_i} - \delta_{M_{i-1}}$. The term $\Delta \delta_{M_i}$ represents the change in the max-
 264 imum response deformation at the i -th seismic excitation only; with $\delta_{M_0} = 0$.
 265 However, trains of ground motions can be organized in many different ways;
 266 i.e., they can occur in different order. Therefore, the damage accumulated
 267 after the z -th event depends on the order in which the events were selected.
 268 Let's define the j -th random sequence as S_j , then, the probability that the
 269 damage exceeds a threshold s^* in the z -th earthquake event can be computed
 270 using Monte Carlo Simulation.

$$\mathbb{P}(D(z) > s^* | D(z-1) \leq s^*) = \frac{\text{Number of cases with } D(z) > s^*}{k_z} \quad (9)$$

271 where k_z is the number of sequences S in which the system has not failed
 272 at the z -th event. The number of events required for the structure to reach
 273 the threshold s^* varies; however, there might be earthquake sequences where
 274 the threshold is achieved only after a large number of events. In these cases,
 275 the probability evaluation made using equation 9 may not converge (even
 276 for an extremely large number of simulations). This can be observed when

277 plotting equation 9 vs z (see section 4). Thus, in order to ensure consistency
 278 in the assessment, the number of events in a sequence should be limited;
 279 for instance, setting a boundary on the difference between the probability of
 280 failure of event z and event $z - 1$.

281 This model considers the realistic case in which degradation caused by
 282 one shock depends on the history of the process up to that point (see section
 283 2.2); i.e., the shock size distribution of one event depends on the accumu-
 284 lated damage just before the event occurs. Increasing damage with time,
 285 conditioned on the history of the process, can be model, for instance, as [7]:

$$X(t) = \sum_{i=1}^{N(t)} g(Y_i, V(t_i^-)) \quad (10)$$

286 where Y_i is the i -th shock size, $V(t_i^-)$ is the structural condition just before
 287 the i -th event, $N(t)$ is a random variable describing the number of events by
 288 time t , and $g(Y_i, V(t_i^-))$ is a continuous function nonincreasing in V and non-
 289 decreasing in Y . Nevertheless, this path-dependency is difficult to capture in
 290 practice, mainly because there is not enough historical data. This probability
 291 is also difficult to evaluate analytically since it involves the computation of
 292 convolutions (sum of shock size distribution).

293 However, the probability distribution describing the history of the process,
 294 $G(y, z)$, where y is the damage level and z the number of the event in the
 295 sequence, can be captured in equation 9. Then, let's assume that the damage
 296 exceedence probability distribution at any event z has the same general form,
 297 but different parameters $\Phi = \{\phi_1(z), \phi_2(z), \dots, \phi_w(z)\}$; e.g., lognormal with
 298 varying mean $\mu(z)$ and variance $\sigma(z)$. Note that the distribution does not

299 depend on time but on the position in the sequence. If damage is assumed to
 300 be a monotonic increasing function, regression analysis can be used to esti-
 301 mate every parameter $\phi_i(z)$; and, consequently, it is possible to obtain a full
 302 description of $G(y, z)$. This result is important because it allows to estimate
 303 the distribution that characterize the probability that the damage exceeds a
 304 given threshold s^* , after a given number of earthquake events z in a specific
 305 seismic environment. Furthermore it captures, at least in an approximate
 306 manner, the inherent dependence of the process (i.e., accumulated damage)
 307 on the damage observed at the z -th event.

308 3.5. Time variant assessment of failure probability

309 In order to compute the lifetime distribution, it is necessary to take into
 310 account the times at which events occur. Then, note that equation 5 can
 311 be used to compute the total damage after $N(t)$ events; with $N(t)$ a time-
 312 dependent random variable. If the event $z - 1$ occurs and the accumulated
 313 damage does not exceed the threshold s^* , the probability that this threshold
 314 is surpassed in the next earthquake event (i.e., z) is computed as:

$$\mathbb{P}(D(z) > s^* | D(z-1) \leq s^*) = \int_{s^*}^{\infty} dG(y, z, \Phi_z) \quad (11)$$

315 where Φ_z is a vector with the parameters of the distribution G at the z -the
 316 event. If the system is exposed to successive shocks until it fails, and then
 317 abandoned (i.e., the system is not reconstructed or intervened in any way
 318 after failure), the system may reach the threshold s^* in the first event, or the
 319 second, or after a large number of events. Then, the probability of failure is
 320 conditioned on the number of events in a given time t , i.e.,

$$\mathbb{P}(D > s^*, t) = \sum_{z=1}^{\infty} \left(\int_{s^*}^{\infty} dG(y, z, \Phi_z) \right) \mathbb{P}(N(t) = z, t) \quad (12)$$

321 where

$$\mathbb{P}(N(t) = z, t) = \frac{(\lambda t)^z}{z!} \exp(-\lambda t) \quad (13)$$

322 where λ is the occurrence rate of earthquake events and $\mathbb{P}(N(t) = z, t)$ is the
 323 probability of having z shocks by time t ; note that it removes the condition
 324 on z . Equation 12 describes the probability of failure at a given time t , which
 325 describes its time-variant nature. A significant contribution of this approach
 326 is that in Equation 12 it is not necessary to compute the convolution of shock
 327 size distributions, making the evaluation more efficient computationally.

328 4. Illustrative example

329 4.1. Structural model

330 The reinforced concrete building chosen for the study was the one pro-
 331 posed by Marante et al. [33], Sáez et al. [34]. It is a large-scale, one-span,
 332 two-story frame model. The total structure height is 4.2 m and the width
 333 is 4.0 m (Figure 2a). The dimensions of transverse sections are 0.4 m \times
 334 0.3 m for the beams and 0.4 m \times 0.4 m for the columns. The mass of the
 335 building is assumed to be uniformly distributed along beam elements and
 336 the columns are supposed massless. The total mass of the building is 40 T.
 337 With these characteristics the fundamental period of the structure (T_{str}) is
 338 equal to 0.24 s. In order to simulate the nonlinear material response of the
 339 structure, beam-column elements with plastic hinges were used. The model

340 is based on the two-component model presented by Giberson [35] and the
 341 modifications introduced by Prakash et al. [36] to take into account axial
 342 force (P) and bending moment (M) interaction by specifying $P - M$ yield
 343 surfaces (Figure 2b). Refer to Prakash et al. [36], Sáez et al. [34] for further
 344 details about the model. All computations were conducted with GEFDyn
 345 Finite Element code [37].

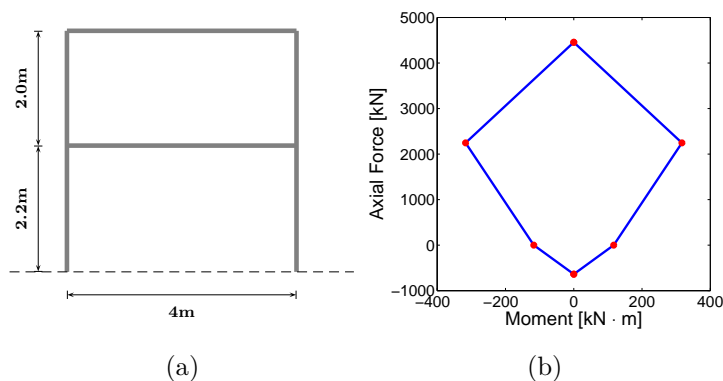


Figure 2: a) Building geometry and b) Axial force-moment interaction diagram used for plastic-hinge column elements of the building [34].

346 Let's consider that this building is located in the Euroseistest site in
 347 Greece on a hard rock site, a well-known experimental site for integrated
 348 studies in earthquake engineering [38]. The objective of this example is to
 349 obtain the lifetime distribution of the structure for various damaging limit
 350 states.

351 First of all, the dynamic response of the building model in a fixed base
 352 condition was analyzed using the whole sets of signals independently, i.e.
 353 without a specific sequence. The seismic demand on the building, namely,
 354 the maximum top displacement (u_{top}) and its corresponding shear strength

355 (V) are shown in Figure 3. In the same figure, the corresponding capacity
 356 curve obtained by a pushover test is also plotted.

357 The limit damage states (performance thresholds) adopted were those
 358 proposed by Penna et al. [39] and Del Gaudio et al. [40]; they are summarized
 359 in Table 1. Note that in Figure 3, only for some few signals a structural
 360 non-linear behaviour appears (i.e. $u_{top} > \Delta_y$, where Δ_y is the displacement
 361 corresponding to the yield capacity of structure). If μ is defined as the
 362 ductility ratio ($\mu = u_{top}/\Delta_y$, with $\Delta_y = 1.06$ cm from pushover test), in this
 363 case the ductility ratio varies from 1 to 1.35. This ductility ratio corresponds
 364 to the one obtained for the maximum u_{top} value.

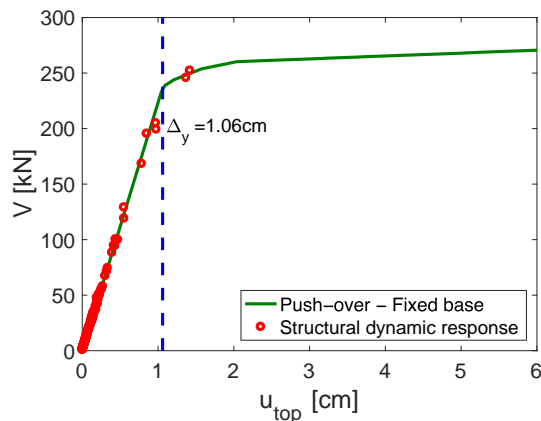


Figure 3: Structural dynamic response obtained for the building at fixed base condition compared with the static capacity curve.

365 4.2. Seismic hazard characterization

366 The next step in the process is the definition of: i) the cumulated damage
 367 of a structure; ii) its probability of failure due to different series of seismic
 368 shocks and iii) its associated uncertainty related to the order of earthquake
 369 occurrence (the Stochastic Earthquake Catalogue, SEC), as proposed

Table 1: Displacement limit states adopted in the illustrative example

Damage state	Displacement threshold	Displacement value* [cm]	Inter-story Drift Ratio (<i>IDR</i>)** [%]
Slight	$0.7 \cdot \Delta_y$	0.74	0.08
Moderate	Δ_y	1.06	0.32
Extensive	$\Delta_y + 0.25 \cdot (\Delta_u - \Delta_y)$	2.30	0.94
Complete	Δ_u	6.00	1.78

* Values used in the illustrative example [39]

** Values from Del Gaudio et al. [40]

370 by Aristizábal et al. [28] was used. The following stages were performed to
 371 obtain different series of seismic shocks:

- 372 • The catalogue of ground motions was constructed by sampling events
 373 from the European SHARE model [41] enclosing the Euroseistest site
 374 in Greece (GRA390). The magnitude-frequency distribution is shown
 375 in Figure 4a.
- 376 • In order to generate stochastic ground motion time histories, the stochas-
 377 tic method proposed by Boore [42] was used. This procedure is ex-
 378 plained in Aristizábal et al. [28]. This method is implemented using
 379 the SMSIM code [43]. Nevertheless, any other suitable method for this
 380 purpose could be used.
- 381 • The SEC was built for seismic records on hard rock ($V_s=2600$ m/s) at
 382 site; the hazard curve built from this catalogue is shown in Figure 4b.

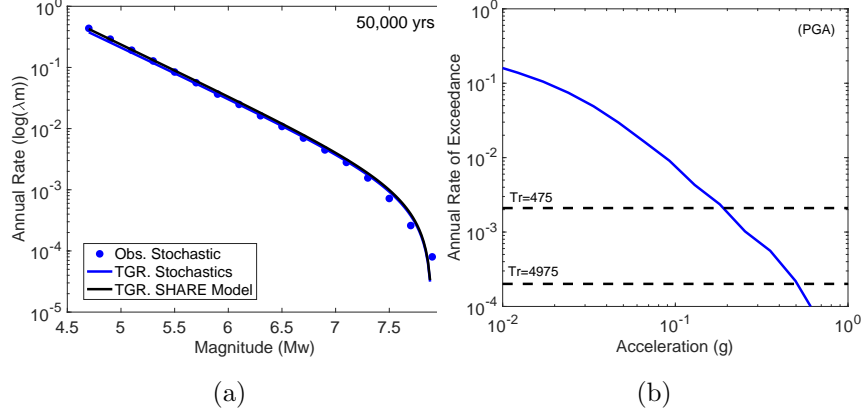


Figure 4: a) Truncated Gutenberg-Richter recurrence models of the 50,000 years catalogue lengths compared with the original recurrence model of the source zone GRAS390 in the SHARE model and b) Hazard curves for PGA spectral periods built from the SMSIM stochastic time histories. (Figures from Aristizábal et al. [28]).

383 It is created from a 50,000 year stochastic earthquakes catalogue with
 384 $\approx 21,800$ events ranging from magnitudes $M_w = [4.5 \text{ to } 7.8]$, Joyner
 385 and Boore distances [44] from $RJB = [0 \text{ to } 150]$ km and a hypocentral
 386 depth from $D_{hyp} = [0 \text{ to } 30]$ km. The probability density function (PDF)
 387 of the ground motion acceleration (PGA) on hard rock for different
 388 magnitude and distance ranges of the SEC is shown in Figure 5.

- 389 • Then, from the catalogue 21 different sub-catalogues of 100 years were
 390 randomly sampled; they correspond to 44 acceleration time histories in
 391 each sub-catalogue. Finally, to account for the randomness of shock se-
 392 quences (i.e., order of occurrences), the 44 events in each sub-catalogue
 393 were permuted randomly; this procedure is repeated 15 times for each
 394 sub-catalogue. This resulted in the final sample of 315 sub-sets to be

395 used for each computation of 44 seismic shock sequences. The same
 396 procedure was done for the 21 different sub-catalogues of 50 and 75
 397 years which contain 22 and 33 acceleration time histories each one re-
 398 spectively.

399 This work is based on a classical poissonian PSHA Gutenberg-Richter
 400 recurrence models extracted from the European SHARE model [41], since

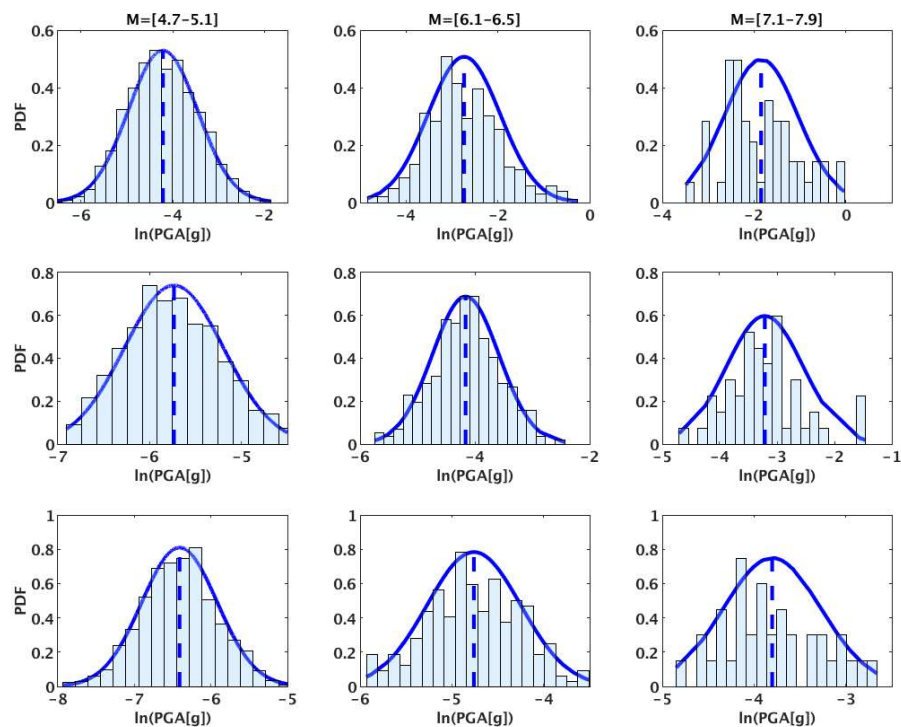


Figure 5: Probability density function (PDF) of the ground motion acceleration (PGA) on hard rock for different magnitude and distance ranges of the SEC. The Joyner and Boore distances R_{JB} from [0 to 150] km and a hypocentral depth D_{hyp} from [0 to 30] km. (Figures from Aristizábal et al. [28]).

401 this is the methodology that is widely used in earthquake engineering to de-
402 rived earthquake recurrence models and the majority of seismic hazard maps
403 worldwide. Under this assumption, the main shocks are assumed to be inde-
404 pendent, hence aftershocks and foreshocks are discarded and removed from
405 the catalogue. Nevertheless, in some specific cases to better estimate the real
406 lifetime of a structure, it is important to add clusters as suggested by several
407 authors [e.g. 45, 46, 47, 48, 49, 50, 51, 52]. But this issue is not addressed
408 in this publication, since the aim here is to present the methodology and to
409 show its versatility to better estimate the real lifetime of a structure subject
410 to cumulative damage.

411 If it is necessary to consider the additional effect of such clusters, it will
412 need only to add the foreshocks and aftershocks before and after each main
413 shock of the synthetic earthquake catalogue and run again the FEM calcu-
414 lations considering the cluster for each main shock. The methodology pre-
415 sented here is so robust, that it can easily be used with any type of recurrence
416 model (including or not aftershocks and foreshocks) that the user considers
417 “realistic” for any case example than the one presented in this article.

418 *4.3. Structural performance when subjected to trains of earthquakes*

419 In order to compute the lifetime distribution it is necessary to model
420 the structural performance when exposed to multiple sequences of ground
421 motions. This means, evaluating the accumulated damage in a sequence
422 of events until the total damage reaches a defined damage threshold; i.e.,
423 $u_{top} \geq u_{lim}$; where u_{top} is the maximum roof displacement and u_{lim} the limit
424 state under consideration or $IDR \geq IDR_{lim}$ (see Table 1). Figure 6a shows
425 u_{top} values for 315 trains of earthquakes with a time window of 100 years; i.e.,

426 every mark indicates the u_{top} for structure after 1,2,3,...44 ground motions.
 427 Two limit states, defined according to Table 1, are also indicated with dashed
 428 lines; i.e. *Slight* damage: $u_{lim} = 0.74$; and *Moderate* damage: $u_{lim} = 1.06$ cm.
 429 In a similar way, Figure 6b displays the evolution of obtained *IDR* values for
 430 the same time window. In this case the two limit state are the ones proposed
 431 by Del Gaudio et al. [40] (i.e. 0.08% and 0.32%).

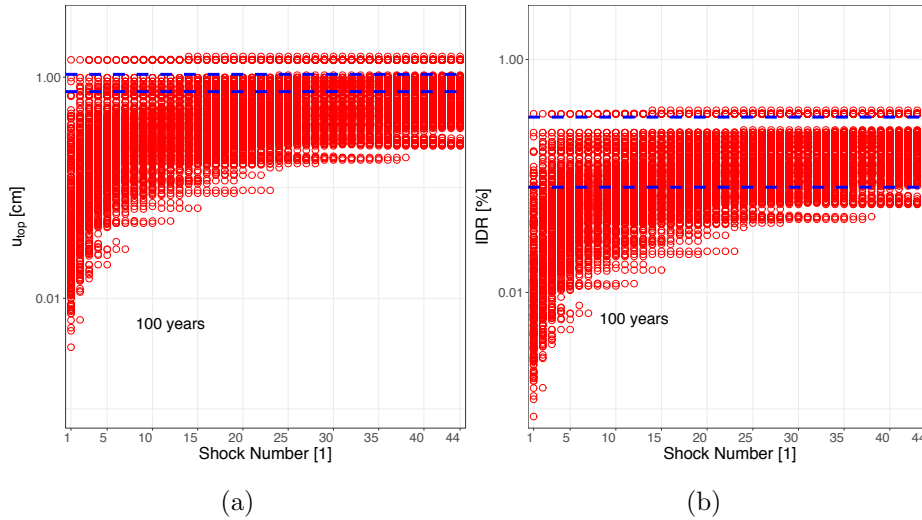


Figure 6: Evolution of a) the u_{top} and b) the *IDR*, against the number of seismic shocks, for 44 earthquake trains, a period of 100 years.

432 In addition, Figure 7 displays the box plots of the obtained u_{top} (Figure
 433 6a) and *IDR* (Figure 6b) evolution as a function of the number of seismic
 434 shocks for 100 years time windows. It is important to observe that as ex-
 435 pected, the median of two variables, u_{top} and *IDR* increases as the number
 436 of seismic events in the sequence.

437 After N simulations (i.e., trains of events), there are N measurements of
 438 u_{top} and *IDR* for every position z in the sequence. These values represent

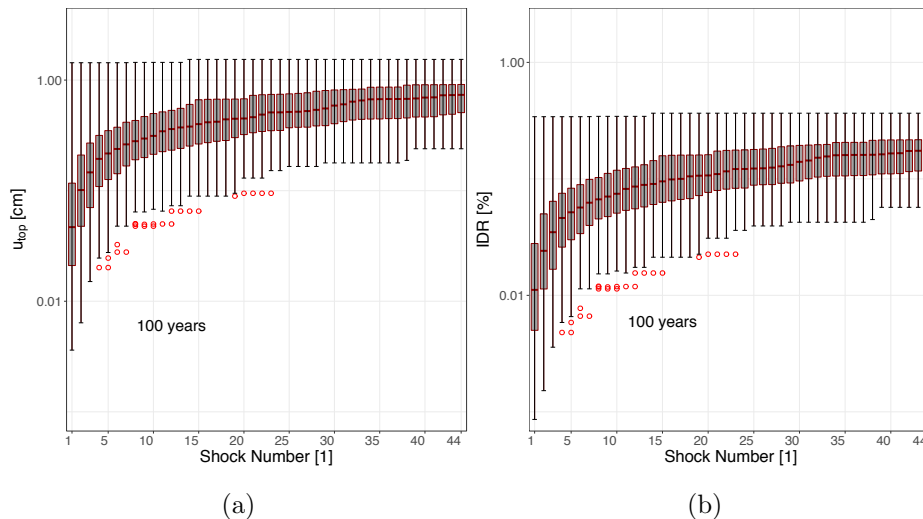


Figure 7: Box plots of the obtained a) u_{top} and b) IDR , against the number of seismic shocks, for a period of 100 years.

439 possible damage states after the structure has been subjected to z ground
 440 motions. Then, the probability distribution of the total damage (i.e., u_{top} or
 441 IDR), $G(y, z)$, can be computed for every z . For example, for the problem
 442 at hand, and $z = 25$, obtained data from u_{top} can be adjusted to a logistic
 443 distribution with parameters $\mu = 0.561$ and $\sigma = 0.306$. Although it is not
 444 necessary, it is highly convenient to adjust $G(y, z)$ to the same functional
 445 form (e.g., logistic, lognormal); however, if this is not possible, the range of
 446 z values can be divided in sectors, where each one has the same functional
 447 form.

448 In this example, the analysis showed that in most of the cases, the logistic
 449 distribution fitted very well the data. Then, the logistic distribution was
 450 selected as the base damage distribution $G(y, z)$. The fittings for the mean

451 and standard deviation could be described with the equation 14.

$$G(y, z) = \beta_0 + \beta_1 \cdot z^2 + \beta_2 \cdot \ln(z) \quad (14)$$

452 Table 2 summarizes the obtained parameter values for the equation de-
 453 scribing the evolution of mean and standard deviation of both u_{top} and IDR
 454 (Figure 8). In the Figure 8a, the *Slight* limit damage states for both u_{top}
 455 and IDR , are also plotted with dashed lines. It is important to note that in
 456 both cases, for lower shock number the mean value is lower than the damage
 457 limit, however, with the accumulation of shocks those limits are exceeded.

Table 2: Parameters for the estimated mean and standard deviation values of u_{top} and IDR as a function of z (equation 14).

	u_{top} [cm]			IDR [%]		
	β_0	β_1	β_2	β_0	β_1	β_2
$\hat{\mu}(z)$	0.0578	$9.226 \cdot 10^{-5}$	0.1468	0.0138	$2.190 \cdot 10^{-5}$	0.035
$\hat{\sigma}(z)$	0.168	$1.264 \cdot 10^{-5}$	0.0424	0.0364	$8.469 \cdot 10^{-7}$	0.0119

458 For illustrative purposes, the Empirical Complementary Cumulative Dis-
 459 tribution Function (CCDF) of the u_{top} and IDR values, for various values
 460 of z , are shown in Figure 9. In the same figure, two limit damage states
 461 are also plotted with dashed lines; i.e. *Slight* damage and *Moderate* damage.
 462 From the results of this Figure, it is noted that the CCDF form will evolve
 463 with the increasing of the shock number. It may also be concluded that , in
 464 both cases, for the same level of damage state, the probability of exceedance
 465 is greater while the number of shock increases. It is also indicated that this

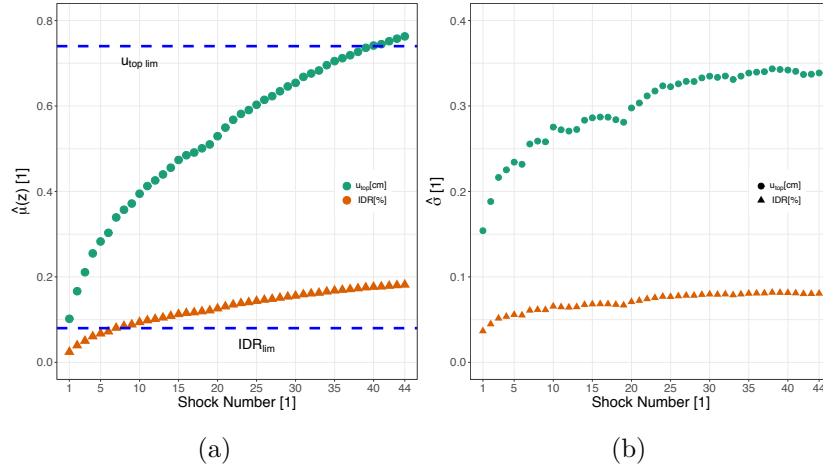


Figure 8: a) Mean and b) Standard deviation evolution for every position z in the sequence of both u_{top} and IDR .

466 particular structure would be more prone to develop *Slight* damages than
 467 *Moderate* ones.

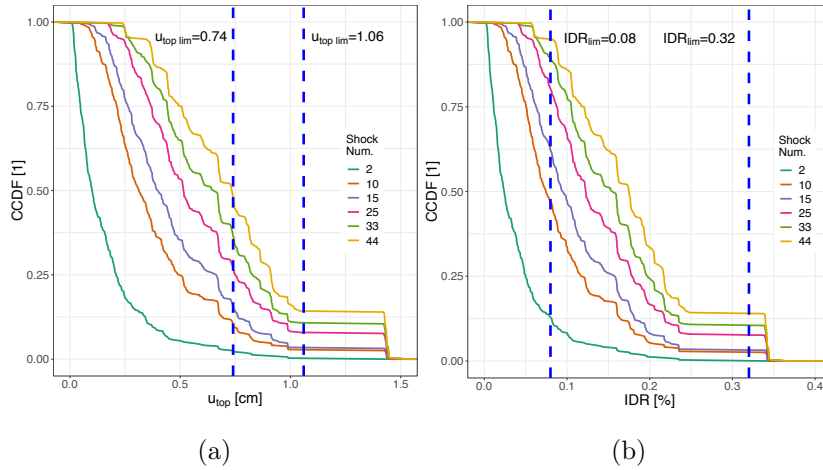


Figure 9: Empirical Complementary Cumulative Distribution Function (CCDF) of a) u_{top} and b) IDR , for some z -th events.

468 4.4. Structural lifetime distribution

469 Now the lifetime distribution can be computed for various limit states
470 and several performance indicators. Figure 10 shows the probability that the
471 system's damage reaches a value larger than a threshold $s^* = u_{lim}$ or $s^* =$
472 IDR_{lim} , as function of the number of shocks in a sequence; it is computed
473 based on equation 9. For example, the probability that after 22 shocks the
474 accumulated damage (u_{top}) is larger than 0.74 cm is about 25% and 75% for
475 the case of IDR exceeds 0.08%. It is important to stress that the probability
476 of exceedence starts fluctuating as the number of events to reach the failure
477 increases. This is caused mainly by the fact that only in few cases the
478 structure survive so many earthquakes. Therefore, the analysis was limited
479 to a maximum number of shocks so that $\mathbb{P}(u_{top} > u_{lim}, z) - \mathbb{P}(u_{top} > u_{lim}, z -$
480 $1) \leq \zeta$, with z the number of shocks and ζ a predefined small value. In this
481 example, the train of events was limited to 44 ground motions for 100-year
482 time window.

483 In addition, it is well known that the choice of time window and conse-
484 quently the size of the seismic shock sequences are key issues on the quality
485 of the lifetime predictions. Thus, in order to study the evolution of the es-
486 timated probability to reach a damage state as a function of the size of the
487 seismic shock sequences, the sub-catalogues for 50 and 75 years time windows
488 were used. Figure 11 shows the probability that the system's damage reaches
489 a value larger than a threshold $s^* = u_{lim}$ or $s^* = IDR_{lim}$, as function of both
490 the number of shocks in a sequence and the time window. It can be observed
491 that the response changes depending upon the damage level selected. Then,
492 for $IDR_{lim} = 0.08\%$, the variation of the $\mathbb{P}(IDR \geq IDR_{lim})$ with respect to

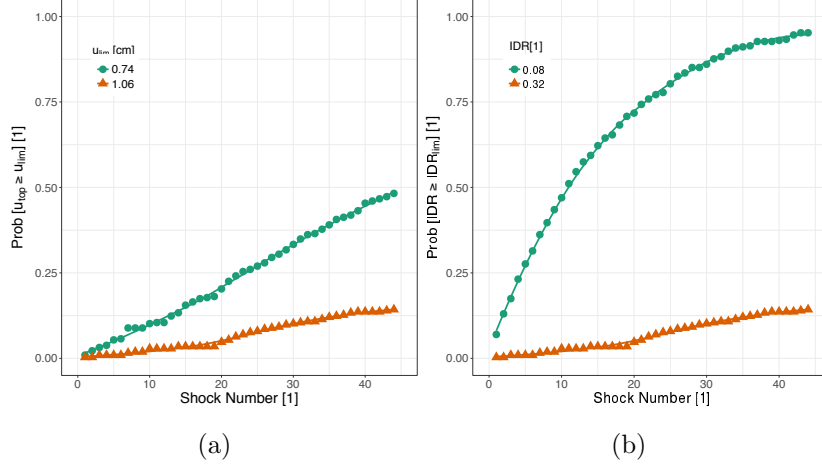


Figure 10: a) $\mathbb{P}(u_{top} \geq u_{lim})$ and b) $\mathbb{P}(IDR \geq IDR_{lim})$, for each seismic shock.

493 the time window is very small. However, for all other cases a bigger disper-
 494 sion was observed. This dispersion is a function of the information provided
 495 by the size of the motion database (i.e. sub-catalogues) over the parameters
 496 describing the probability of failure (equation 9).

497 4.5. Lifetime distribution and Mean-Time-To-Failure

498 The occurrence rate of events was obtained for the studied case. There
 499 were 44 events, with the specifications defined for this study, therefore,
 500 $\lambda = 0.44$ earthquakes/year. Then, assuming that their occurrence follows
 501 a Poisson Process, the probability distribution of the time to failure can be
 502 easily computed using equation 12. The results are shown in Figure 12.
 503 For the studied building, it is noted that for the design working life of 50
 504 years, the probability to exceed the *Slight* limit damage states is 24% and
 505 75% for both u_{top} and IDR respectively. Concerning the *Moderate* limit
 506 damage states, the probability is close to 5% for both demand parameters.

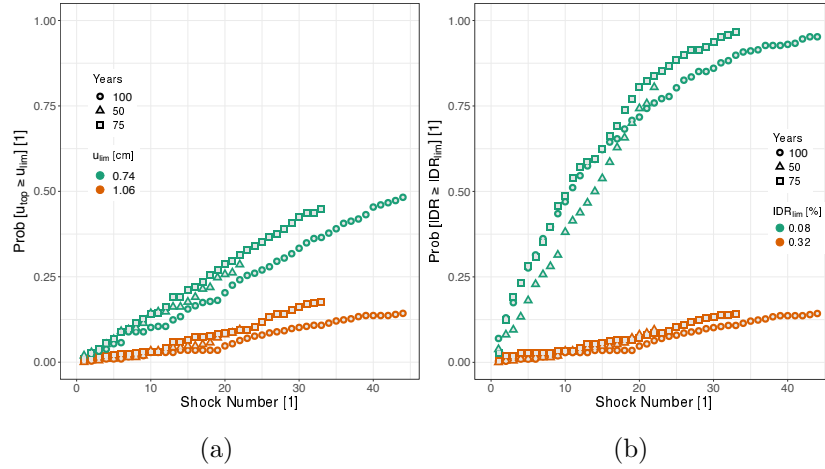


Figure 11: Effect of the time window on a) $\mathbb{P}(u_{top} \geq u_{lim})$ and b) $\mathbb{P}(IDR \geq IDR_{lim})$, for each seismic shock.

507 Those probability values are above the requirements specified in the code of
 508 practice.

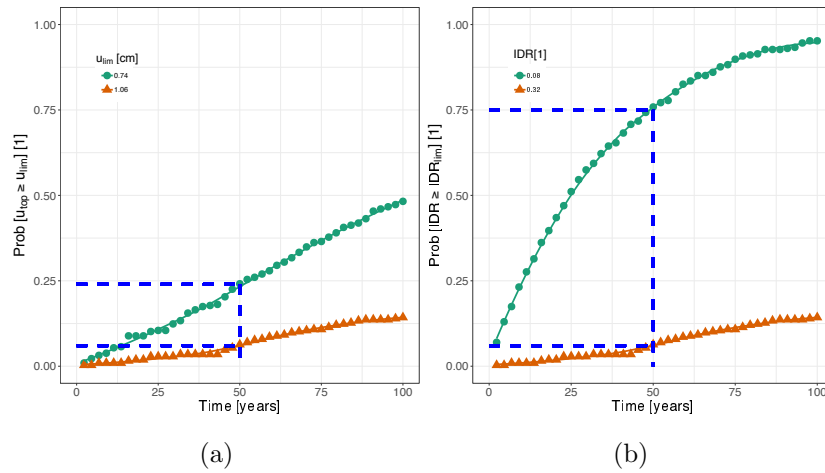


Figure 12: a) $\mathbb{P}(u_{top} \geq u_{lim})$ and b) $\mathbb{P}(IDR \geq IDR_{lim})$, evolution with time.

509 Finally, so as to assess the failure time or survival time of the studied

510 structure, a survival analysis was used. This analysis gives the probability
511 that the structure will survive past time t (i.e. the obtained demand pa-
512 rameter values are lower than the defined limit damage state). In this work
513 the non-parametric Kaplan-Meier estimator [53] was employed for estimat-
514 ing the survival functions from observed survival times. In this method, it is
515 assumed that the structure failure at each event occur independently of one
516 another. Thus, the probability of not having a damage at time t_i , $S(t_i)$, is
517 calculated by :

$$S(t_i) = S(t_{i-1}) \left(1 - \frac{d_i}{n_i}\right) \quad (15)$$

518 where $S(t_{i-1})$ is the probability of not having a damage at t_{i-1} , n_j the number
519 of case not having a damage just before t_i and d_i the number of observed
520 damages at t_i . At time $t = 0$, $S(t) = 1$.

521 Figure 13 displays the obtained survival curves for the *Slight* and *Moderate*
522 limit damage states. The 95% confidence limits of the survivor function are
523 also shown. It is noted that median survival time (MTTF) for the *Slight* limit
524 damage states is 100 and 25 years for both u_{top} and *IDR* respectively (Figure
525 13a). For the *IDR* case the time-to-event curve follows a Weibull distribution
526 ($S(t) = \exp(-\alpha \cdot t^\gamma)$) with $\gamma = 1.119$ and $\alpha = 17.8 \cdot 10^{-3}$. Concerning the
527 u_{top} case, the parameters are $\gamma = 1.285$ and $\alpha = 1.77 \cdot 10^{-3}$. Nevertheless,
528 no information could be obtained from the survival curves for the *Moderate*
529 limit damage states due to lack of observations (Figure 13b).

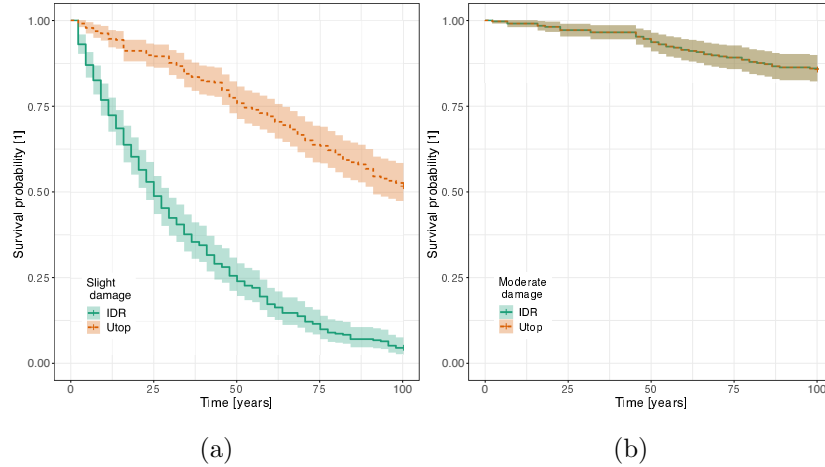


Figure 13: Cumulative survival plots by a) *Slight* and b) *Moderate* limit damage states, evolution with time.

530 5. Conclusion

531 This paper presents a simplified model that can be used to estimate the
 532 lifetime distribution and the corresponding mean time to failure (MTTF) of
 533 a structural system located in a active seismic region. The model is intended
 534 to bring into practice the possibility of estimating the lifetime of a structural
 535 system without entering into the mathematical complexities that are required
 536 for an analytical solution, but retaining the main features of the evaluation.
 537 The model assumes that the structural deterioration is mainly controlled
 538 by the consequences of earthquake main shocks events. Then, it takes into
 539 account the accumulation of damage when the structure is subjected to a
 540 sequence of main shocks that occur randomly in time following a homoge-
 541 neous Poisson process with constant mean rate of occurrence. The model
 542 allows estimating the lifetime for several performance limit states, which are

543 evaluated using the damage levels proposed in the literature, nevertheless
544 any other damage indicators are also suitable to be used under the proposed
545 methodology. The model has been developed so that it takes into account
546 important aspects of the degradation process, the associated uncertainties
547 and the actual modeling requirements. However, it makes some reasonable
548 simplifications that overcome the complexities of analytical computations.
549 Also, it uses an efficient simulation process that reduces the computational
550 costs that imply a numerical, and sometimes in-existent, solution of the prob-
551 lem. Some aspects of the model can be refined or modify when required to
552 capture specific characteristics of the seismic environment or the structure.
553 Time span, damage indexes, recurrence models, type of structure, dynamic
554 model, among others, can be modified easily together with its correspond-
555 ing uncertainties using this method. Finally, the proposed model can be
556 also used as evidence to make better decisions regarding operational policies,
557 maintenance or life-cycle cost estimations.

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