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Survival analysis of a liquefiable embankment subjected to sequential earthquakes

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Abstract

In practice, the performance of the structure is studied based on a seismic scenario composed of independent single earthquakes. But in real life, the structure is subjected to multiple earthquakes during its typical design working life, which will produce an evolution of damage with time. The main purpose of this paper is to quantify the liquefaction-induced damage of an embankment due to sequential earthquakes during a defined working life. Moreover, a non-parametric survival analysis is used to estimate the time (in years) until a defined damage level is reached during a specific time interval. For this purpose, a site was chosen where its seismicity and its Probabilistic Seismic Hazard Analysis (PSHA) were identified. First, a site-specific seismic analysis was assessed, that consists in finding the relation between the Intensity Measures (IM) and the Engineering Demand Parameter (EDP). Second, in order to estimate the lifetime distribution as well as the Mean Time To Failure (MTTF) of the embankment, survival functions were drawn. The used

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time histories were stochastically generated from synthetic ground motion models. In this study, an elastoplastic multi-mechanism soil behavior model was used. According to the obtained results, after the sequential loading, the cumulative damage is either progressive with or without extensive damages or sudden with drastic damages. Moreover, based on the chosen ground motion model, the embankment reaches a moderate damage level before its defined working life. In addition, a numerical parametric analysis is performed in order to quantify the impact of considering (or not) the loading history and the recovery time between each ground motion on the obtained MTTF of the embankment. This study pointed out on the importance of the history of loading since it affects the overall performance of the embankment. Finally, two synthetic ground motion models were assessed in order to generalize, to a certain extent, this work.

Keywords: working life, sequential, survival analysis, liquefaction, synthetic models, elastoplastic soil behavior

1. Introduction

Seismic hazard analysis involves the quantitative estimation of the ground motion (GM) hazards of a specific area. It requires the knowledge of the geologic evidence, the fault activity, the magnitude and the historical seismicity of the studied region [30, 6]. The *Probabilistic Seismic Hazard Analysis (PSHA)* considers the uncertainties in the earthquake size, location and occurrence time. It estimates the mean frequency of exceedance of any spectral acceleration at the site [5]. The level of shaking produced from this analysis comes from the contribution of the magnitude M_w , the source-to-site dis-

10 tance R and often the deviation of the GM from the predicted value (ε) [5].
11 Given the aforementioned information, the Ground Motion Prediction Equa-
12 tions (GMPEs) create the relationship between the magnitude, distance, and
13 other model parameters and the Intensity Measures (IM). In this context, the
14 study of the non linear behavior of the structures needs a recall to a large
15 number of acceleration time histories. In addition and for particular scenar-
16 ios, available data resources are sometimes inadequate to characterize the
17 models due to several problems (i.e. ground motions from very large magni-
18 tude earthquakes, near-fault ground motions, basin effects) [57, 37, 54, 50, 60,
19 among others]. For this reason, artificial or synthetic earthquakes could be
20 used. They are conducted based on several methods (i.e. stochastic ground
21 motion model, the composite source method, among others) and are useful
22 when real motions are not available.

23 In practice, structures are designed to resist the first damaging earth-
24 quake scenario [23]. But during their service life, the structures are not only
25 exposed to a single seismic event but also to multiple or repeated earthquake
26 shocks. Previous works in this context have been conducted on various struc-
27 tures like buildings or bridges [20, 49, 47, 18, 63, 48, 17, 25, 41, 24, 14, among
28 others]. As a consequence of the later, structural damage accumulation by
29 consecutive earthquake loading will be produced. The damage accumulation
30 according to Iervolino et al. [25], is mainly due to two phenomena: i) contin-
31 uous deterioration of the material which is called “*aging*” or ii) cumulative
32 damage due to repeated load, also known as “*sequential earthquakes*”. The
33 cumulative damage of the structure during its working life, is known as the
34 *Life Cycle* of the structure [62, 52, 49, 25, 46, 51, 33, among others]. In

35 another context, the life cycle of the structure can take lots of definitions.
36 It can be considered as the cycle needed for a structure to be constructed,
37 maintained and economically valued (i.e. LCSA [26]). It can also be consid-
38 ered as the time length of the structure until the occurrence of an event of
39 interest (i.e. equipment failure, damage, complex system), or in other words,
40 the time-to-event study. The later is known as the *Survival Analysis*. It is
41 generally defined as a set of statistical methods to analyze data that has the
42 time of occurrence of an event of interest as the outcome. Such analysis is
43 not a new subject in medicine precisely [21, 13, 9]. For example, it is used
44 to validate the impact of a certain disease on different types of patients, or
45 the occurrence of specific symptoms after a drug. Reflecting this analysis in
46 the geotechnical field, it is, to the knowledge of the authors, still a new topic
47 [39, 12, 15].

48 Otherwise, the behavior of the structure (e.g. reinforced concrete build-
49 ings) under seismic sequence loading is assessed based on the *Incremental*
50 *Dynamic Analysis (IDA)*. It consists in subjecting the structural model to
51 multiple ground motion records each scaled to different intensities [59]. Then,
52 a limit state is considered in which the structure reaches failure when it ex-
53 ceeds the limits. On the other hand, previous studies in structural analysis
54 have shown that, for mid-rise buildings, ten to twenty records are enough to
55 have an estimation of the seismic demand [55, 59, among others]. IDA in
56 this case, is easily applied since it does not have a large set of earthquake
57 scenarios to draw fragility curves. Whereas in earthquake geotechnical en-
58 gineering and particularly in liquefaction related problems, this approach is
59 not enough to represent the overall response of the geo-structure due to i)

60 the multi-physical aspects of the soil (solid, water and air), ii) its history of
61 loading that will affect its future behavior [56, 36] and iii) the correlation of
62 the soil response with several intensity measures of the real seismic motions
63 (i.e. Arias intensity, number of cycles) [11]

64 The present work aims to quantify numerically the liquefaction-induced
65 damage on an embankment due to sequential earthquake loading. Following
66 the Performance Based Earthquake Engineering (PBEE) approach, a PSHA
67 should be conducted in which the seismicity of the site and the occurrence
68 rate of earthquake are identified. In this work, the site of concern is located
69 in Mygdonia, Greece. The reference to the fully probabilistic hazard analysis
70 in this study are based on the work of Aristizábal et al. [2]. A large number
71 of time histories was generated using stochastic simulations from synthetic
72 ground motion models (e.g. Rezaeian and Der Kiureghian [45] and Boore
73 [7]). Nevertheless any other stochastic models are also suitable to be used
74 under the proposed methodology. At the beginning of this work, the induced
75 damage was quantified based on a set of GM records without sequences sim-
76 ilarly to a site-specific seismic analysis. Concerning the sequential analysis,
77 the methodology adopted in this study is shown in Figure 1. Assuming that
78 the working life of the embankment (T_{window}) is 100 years, and according
79 to the PSHA and the catalog GM constructed for this site, the event rate
80 of the mainshocks ($\lambda_{earthquakes}$) is 0.44 events/year. Thus, 44 acceleration
81 time histories (N_{shocks}) should occur during this period. Then the sequential
82 loading is obtained by a random permutation of the obtained number of main-
83 shocks. In order to calculate the survival function ($P(\sum D(t) < D_{threshold})$),
84 a threshold damage $D_{threshold}$) should be identified. Hence, the lifetime dis-

85 tribution of the embankment can be estimated as well as its Mean Time To
 86 Failure (MTTF, the expected time to failure for a non-repairable system).
 87 In this study, the survival analysis is computed based on a non parametric
 88 statistical method [27]. The main advantage behind this method is that it
 89 does not require the assumptions of a particular probability distribution (i.e.
 90 Weibull, exponential,log-logistic) of the structure’s survival function. Also in
 91 this work, a numerical parametric analysis is performed in order to quantify
 92 the impact of considering (or not) the loading history and the recovery time
 93 between each ground motion on the obtained MTTF of the embankment.
 94 This study points out the importance of the history of loading since it affects
 95 the overall performance of the embankment. Finally, two synthetic ground
 96 motions models are assessed in order to generalize, to a certain extent, this
 97 work. The 2D finite element calculations were performed using the GEFDyn
 98 code [3]. For the soil behavior, an elastoplastic multi-mechanism model that
 takes into consideration the history of loading was used.

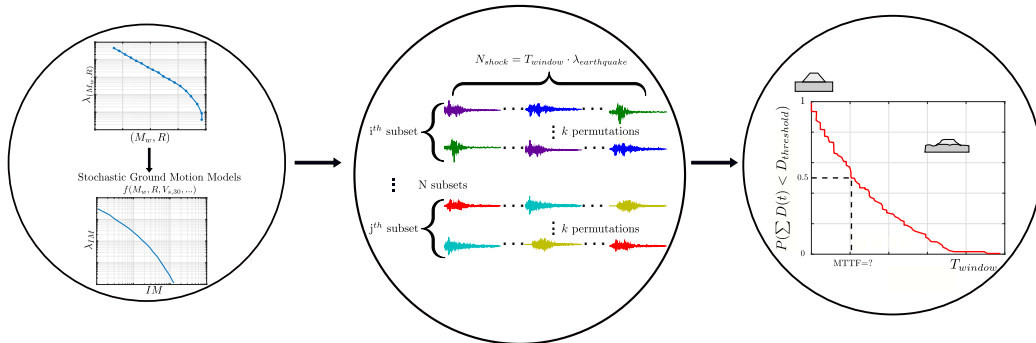


Figure 1: Schema of the used methodology to estimate the lifecycle of an embankment

99

100 The paper is structured as follows. It starts by introducing the theory
 101 behind the survival analysis in Section 2. The geometry and the numerical

102 model are shown in Section 3. The development of the used synthetic ground
 103 motion model is presented in Section 5. The site-specific seismic analysis of
 104 the embankment is developed in Section 6. Then, in Section 7, the sequen-
 105 tial and the survival analysis are presented. Finally, the different types of
 106 sequential analysis approaches are developed in Section 8, as well as the
 107 consideration of different synthetic GM models. The paper is closed with
 108 conclusions.

109 2. Overview of the Survival Analysis

110 The survival analysis is the analysis of time-to-event data. These data
 111 describe the length of time until the occurrence of a well-defined end point of
 112 interest [9, 28, 53, among others]. Survival analysis is conducted via survival
 113 (or survivor) functions or hazard functions. Let T be a non-negative random
 114 variable that represents the surviving time. Denoting the duration of each
 115 event as t , the probability density function of T is $f(t)$, and its cumulative
 116 distribution is $F(t) = P\{T < t\}$. First, the survival function is:

$$S(t) = P\{T \geq t\} = 1 - F(t) = \int_t^{\infty} f(x)dx \quad (1)$$

117 The survival function is non increasing (i.e. at $t = 0$, $S(t) = 1$) and when
 118 the time increases, it tends to approach zero.

119 Second, the hazard function which represents the instantaneous rate of
 120 occurrence over time, is:

$$h(t) = \lim_{\Delta t \rightarrow 0} \frac{P(t \leq T < t + \Delta t | T \geq t)}{\Delta t} = \frac{f(t)}{S(t)} \quad (2)$$

121 Both, the survival and hazard functions are inversely proportional so that
 122 when the hazard increases, the survivor declines and vice versa [28, 53, among

123 others]. The survival time response are usually continuous. When they are
124 not completely observed, they are called censored.

125 The focus of this paper is to conduct a survival analysis to find the life-
126 time of an embankment subjected to sequences of ground motions during a
127 working life of 100 years. The event of interest in this case is the occurrence
128 of these motions, which happens randomly in real life. In addition the param-
129 eters or the shape of the distribution of the embankment survival function
130 are unknown. For these reasons, non parametric analysis will be conducted.
131 In this section, the three methods to analyze the survival data are developed
132 [22, 28]:

- 133 • *Non-parametric method* is a widely used method. It consists in plotting
134 the Kaplan-Meier curve [27]. The simplicity of this curve is that it does
135 not need any assumptions for the distribution of the survival time, or
136 the relationship between the covariates and the survival time.
- 137 • *Semi-parametric method*, in which there is also no assumption for the
138 distribution of the survival time but assumes the relationship between
139 the covariates and the hazard (also the survival) function. This method
140 uses the Cox Proportional Hazard (Cox PH) model.
- 141 • *Parametric method* assumes the distribution of the survival time and
142 the form of the covariates.

143 2.1. Kaplan-Meier estimator

144 The Kaplan-Meier (KM) estimator (or product-limit estimator) $\hat{S}(t)$ in-
145 corporates information from censored and uncensored observations. It con-
146 sideres the survival function to any point in time as series of steps defined by

147 the observed survival and censored times [27]. The probability of surviving
 148 an event in time t_i is calculated from the probability of surviving the event
 149 at time t_{i-1} . Hence the KM estimator of the survival function is::

$$\hat{S}(t_i) = \prod_{t_i \leq t} \left[1 - \frac{d_i}{Y_i}\right] \quad , \quad (3)$$

150 with d_i is the observed cases that reached failure and Y_i is the cases that
 151 are still at risk. Normally, at $t_0 = 0$, $S(0) = 1$. Several approaches are
 152 used to calculate the variance of the KM estimator. The commonly used
 153 approach is the *delta method*. The KM estimator is viewed as a product
 154 of two proportions. Hence, in order to calculate its variance, it is better to
 155 derive one for its logarithm since the variance of a sum is simpler to calculate
 156 than the variance of a product [21]. Hence, the Greenwood formula for the
 157 variance of the survival function will be:

$$Var[S(t)] = Var[\exp[\ln(\hat{S}(t))] = [S(t)]^2 \sum \frac{d_i}{Y_i(Y_i - d_i)} \quad , \quad (4)$$

158 2.2. Cox Proportional Hazards model

159 The basic Cox PH model fits the survival data with the covariates z to a
 160 hazard function. Actually, this model does not directly estimate the survival
 161 functions, instead it attempts to fit it with the hazard function that has the
 162 form of

$$h(t | z) = h_0(t) \exp(\beta' z) \quad , \quad (5)$$

163 where $h_0(t)$ is the baseline hazard and β is a parameter that represents the
 164 effect of covariate on the outcome. Assuming that one event occurs at a time
 165 t_i , the parameter β can be calculated by solving the partial likelihood:

$$PL(\beta) = \prod_{t_i} \frac{\exp(\beta z(t_i))}{\sum_{j:t_j \leq t_i} \exp(\beta z(t_j))} \quad . \quad (6)$$

166 *2.3. Parametric models*

167 It is possible to estimate the survival function by making parametric as-
168 sumptions. Some commonly used distributions are the Weibull (or its special
169 case the exponential) and the log-logistic distribution. The advantages of this
170 model is its high efficiency when it deals with small sample size. However,
171 it is difficult sometimes to find the best distribution that fits the given data
172 which may mislead the analysis. For more details about each distribution as
173 well as more developed information about the survival analysis, a reference
174 to Hosmer Jr and Lemeshow [21] is useful.

175 **3. Geometry and Soil Numerical Model**

176 *3.1. Geometry*

177 The model's geometry is a levee of 9 m high composed of dry dense sand.
178 The foundation is formed of 4 m loose to medium sand (LMS) on the top of
179 a 6 m dense sand. The bedrock is located under the dense sand. The water
180 table starts 1 m below the surface to keep the dam dry. The inclination of
181 the levee is a slope of 1:3 (vertical: horizontal). The geometry in this work
182 is inspired from Rapti et al. [43], Lopez-Caballero and Khalil [34], and is
183 detailed in Figure 2.

184 *3.2. Soil Constitutive Model*

185 As for the constitutive model, the *Ecole Centrale Paris (ECP)* elastoplas-
186 tic multi-mechanism model (also known as *Hujeux* model) is the one chosen
187 for this study and is written in terms of effective stress. The non-linearity
188 of this model is represented by four coupled elementary plastic mechanism:

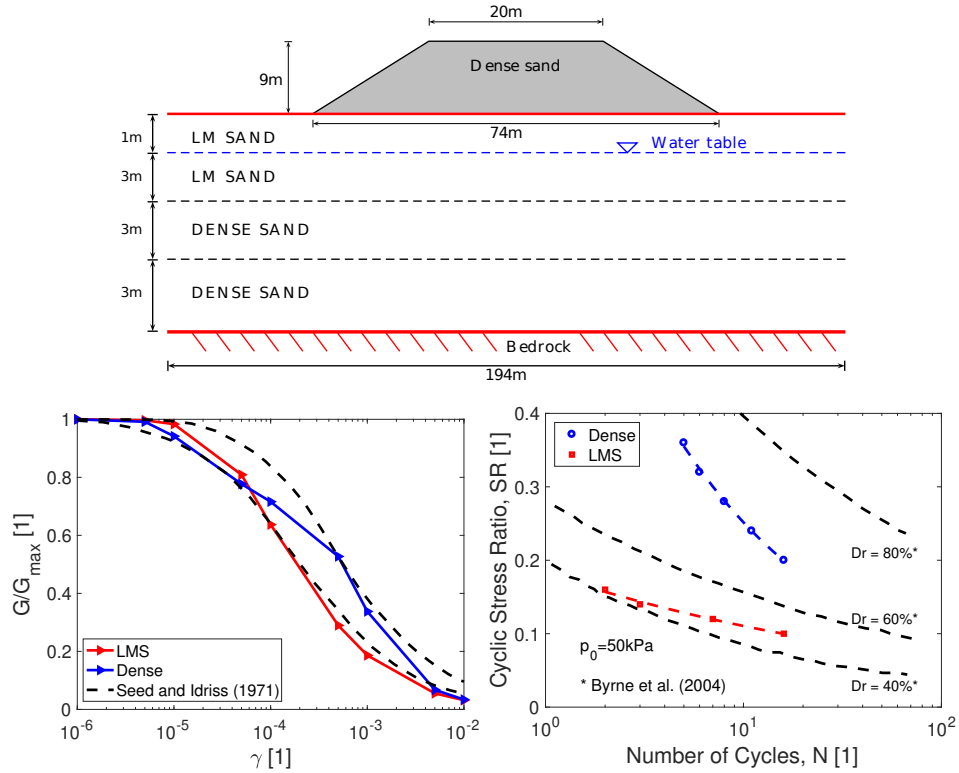


Figure 2: Geometry and behavior of the soil [34]

189 three plane-strain deviatoric plastic strain mechanism in three orthogonal
 190 planes (k - planes) and an isotropic plane to take into account normal forces.
 191 The model follows a Coulomb type failure criterion, contemplate the existence
 192 of dilatancy/contractancy phenomena, and use the critical state concept.
 193 The cyclic behavior is taken into account by a kinematical hardening
 194 that is based on the state variables at the last load reversal. The model is
 195 written in the concept of the incremental plasticity which divides the total
 196 strain into an elastic and a plastic part. Refer to [4, 35, 19, among others]
 197 for further details about the ECP model. For the sake of brevity only, some
 198 model definitions will be developed in the following. Considering the well-

199 known sign convention of the soil mechanics which sets the positive sign to
 200 the compression forces, the yield surface of this numerical model is written
 201 in the k plane as follows:

$$f_k(\sigma, \varepsilon_v^p, r_k) = q_k - \sin \phi'_{pp} \cdot p'_k \cdot F_k \cdot r_k, \quad (7)$$

202 where p'_k and q_k are the effective mean and deviatoric values of the stress ten-
 203 sors and ϕ'_{pp} is the friction angle at the critical state. The parameters that
 204 control the behavior of the soil are F_k , which controls the isotropic harden-
 205 ing associated with the plastic volumetric strain and r_k , which controls the
 206 isotropic hardening generated by the plastic shearing. These two parameters
 207 represent progressive friction mobilization in the soil. At perfect plasticity,
 208 the product $F_k \cdot r_k$ reaches unity, and the Mohr-Coulomb criterion will be
 209 satisfied. The friction angle at the critical state and F_k depends on ε_v^p such
 210 that

$$F_k = 1 - b \ln \left(\frac{p'_k}{p_c} \right), \quad (8)$$

$$p_c = p_{c_0} \exp(\beta \varepsilon_v^p), \quad (9)$$

211 with β is the plasticity compression modulus and p_{c_0} is the critical stress that
 212 corresponds to the initial void ratio. The parameter b shapes the form of the
 213 yield surface in $p' - q$ plane and varies between $b = 0$ where it verifies the
 214 Mohr-Coulomb criterion and $b = 1$ which will be the Cam-Clay criterion. The
 215 third variable of the yield surface which is the degree of mobilized friction
 216 angle r_k is linked to the plastic deviatoric strain ε^p . It shows the effect of the
 217 shear hardening and decomposes its behavior into pseudo-elastic, hysteretic
 218 and mobilized domains. It is given by:

$$\dot{r}_k = \dot{\lambda}_k^p \frac{(1 - r_k)^2}{a}, \quad (10)$$

219 where $\dot{\lambda}_k^p$ is the plastic multiplier of k mechanism and

$$a = a_1 + (a_2 - a_1) \alpha_k(r_k), \quad (11)$$

220 with,

$$\begin{aligned} \alpha_k &= 0 && \text{if } r_k^{elas} < r_k < r_k^{hys}, \\ \alpha_k &= \left(\frac{r_k - r_k^{hys}}{r_k^{mob} - r_k^{hys}} \right)^m && \text{if } r_k^{hys} < r_k < r_k^{mob}, \\ \alpha_k &= 1 && \text{if } r_k^{mob} < r_k < 1. \end{aligned} \quad (12)$$

221 Notice that a_1 , a_2 and m are model parameters and r_k^{hys} and r_k^{mob} designates
222 when the domain shows hysteresis degradation. The isotropic yield surface
223 is assumed to be :

$$f_{iso} = |p'| - d p_c r_{iso}, \quad (13)$$

224 with :

$$\dot{r}_{iso} = \dot{\varepsilon}_{v_{iso}}^p \frac{(1 - r_{iso})^2}{c_{mon} \frac{p_c}{p_{ref}}}, \quad (14)$$

225 where d is a model parameter representing the distance between the isotropic
226 consolidation line and the critical state line in the $(e - \ln p')$ plane and c_{mon}
227 controls the volumetric hardening. In the model, an associated flow rule in
228 the deviatoric k plane is assumed and the Roscoe's dilatancy law is used to
229 obtain the increment of the volumetric plastic strain in terms of the charac-
230 teristic angle ψ and a constant parameter α_ψ such that:

$$\dot{\varepsilon}_{vk}^p = \dot{\lambda}_k^p \cdot \alpha_\psi \cdot \alpha_k(r_k) \left(\sin \psi - \frac{q_k}{p'_k} \right), \quad (15)$$

231 ψ is the characteristic angle and α_ψ a constant parameter. The density
232 hardening is characterized by the critical stress p_c (Eq. 8) that considers all

233 the mechanisms (k - planes and isotropic plane). This can be related to the
 234 plastic volumetric strain such that:

$$\varepsilon_v^p = \sum_{k=1}^3 (\varepsilon_v^p)_k + \varepsilon_v^{iso} = \frac{1}{\beta} \log \frac{p_c}{p_0} \quad (16)$$

235 3.3. Finite Element Model

236 The computations were conducted by a 2D coupled FE modelling with
 237 GEFDyn Code [3], using a dynamic approach derived from the $\underline{u} - p_w$ ver-
 238 sion of the Biot's generalized consolidation theory [64]. The FE model is
 239 composed of quadrilateral isoparametric elements (3.5 m by 1 m) with eight
 240 nodes for both solid displacements and fluid pressures. An implicit Newmark
 241 numerical integration scheme with $\gamma = 0.625$ and $\beta = 0.375$ was assumed in
 242 the dynamic analysis [31]. The FE analysis is performed in three consecu-
 243 tive steps: i) a computation of the initial in-situ stress state due to gravity
 244 loads; ii) a sequential level-by-level construction of the embankment and iii)
 245 a sequential seismic loading analysis in the time domain. This computation
 246 is used in Section 6 and 8.1. For the computation of the sequential seismic
 247 loading developed in Section 7, and for the first motion precisely, the initial
 248 effective stresses, pore-water pressures and model history variables are stored
 249 to be used as initial state for the computation of the second ground motion.
 250 The storage of the history variable of the i th computation will be used as
 251 initial state of the i th+1 computation. More details regarding the calculation
 252 procedures are developed in each section.

253 3.4. Boundary Conditions

254 In the analysis, equivalent boundaries have been imposed on the nodes
 255 of lateral boundaries (i.e., the normal stress on these boundaries remains

256 constant and the displacements of nodes at the same depth in two opposite
257 lateral boundaries are the same in all directions). They are the response of a
258 modeled infinite semispace. Hence, only vertically incident shear waves are
259 introduced into the domain. The model is wide enough (194 m) to ensure that
260 the effect of the boundaries on the response of the model can be neglected
261 and also to satisfy the free field condition at the lateral boundaries. For
262 the half-space bedrock's boundary condition, paraxial elements simulating
263 deformable unbounded elastic bedrock have been used [38]. The incident
264 waves, defined at the outcropping bedrock are introduced into the base of
265 the model after deconvolution.

266 **4. Assumptions for this study**

267 For the study of the life-cycle of a levee subjected to sequential signals,
268 basic assumptions are made:

- 269 • The cumulative damage of the levee is due to the effect of the series of
270 mainshocks only. The effect of aftershocks is not taken into account.
- 271 • The effect of aging is not considered. For example, there is no con-
272 sideration of the rain or sun, the wind load or any other type of loads
273 that may be caused from external uncontrolled conditions. Also aging
274 needs a deeper study of the material resistance, origin and age, which
275 are not considered in this study.
- 276 • At the beginning of the study, before the first seismic loading, the
277 embankment is considered in its virgin and stable state. It does not
278 have a history of earthquake loading.

- 279 • The embankment is not subjected to any repairs during its lifetime.
- 280 • The constitutive model does not take into account the secondary con-
281 solidation or compression after each seismic loading.
- 282 • The pore water pressure have dissipated after each seismic loading. It is
283 ensured by adding a time-gap (recovery time) between each mainshock.
284 This assumption is evaluated later in the paper.

285 5. Input Ground Motions

286 The seismicity of the site requires the knowledge of the geographical lo-
287 cation, the site characteristic and the magnitude-frequency distribution of
288 the earthquakes. The seismic hazard analysis involves the quantitative es-
289 timation of the ground motion characteristic at a particular site with the
290 help of deterministic or probabilistic approaches. The *Probabilistic Seismic*
291 *Hazard Analysis (PSHA)* sets a predictive relationship for each ground mo-
292 tion parameter in each source. This method combines the uncertainties in
293 the earthquake characteristics to obtain the probability that any IM will be
294 exceeded at a particular time period. The hazard curve is used to identify
295 the ground shaking level or the mean annual rate of exceedance (λ_{IM}) [44].
296 Once the main aspects that characterize the local seismic hazard are defined,
297 it is possible to proceed with the selection of time histories. For this purpose,
298 calibrations are used to adjust recorded ground motions and make them more
299 representative of the analysis conditions [57, 61, among others].
300 Concerning the present work, the response of the embankment based on se-
301 quential seismic loading is the major focus of this paper. The site of concern

302 is a valley in Mygdonia that has an epicentral distance located about 30 km
303 to the NE of the city of Thessaloniki in northern Greece. The magnitude M_w
304 in this area is between 4.5 to 7.8. The fully probabilistic hazard analysis is
305 adopted from the study of Aristizábal et al. [2] on the same site. They gener-
306 ated a long catalog from 500 years to 50,000 years (equivalent to a probability
307 of exceedance of 1% in 50 years). Hence, the magnitude-frequency distribu-
308 tion for 50,000 years catalog is shown in Figure 3a. More details regarding
309 the PSHA are presented in the work of Aristizábal et al. [2].

310 In the case of regions with lower seismicity, it is not easy to know with
311 a higher level of accuracy, the expected ground motion scenarios. In prac-
312 tice, the effect of various GMPE's is studied. In addition, to generalize (to
313 a certain extent) this work, two synthetic ground motion models were used:
314 Rezaeian and Der Kiureghian [45] and Boore [7]. They are designated as
315 *Mod.R* and *Mod.B* accordingly. *Mod.R* uses an NGA database [10] and for
316 *Mod.B*, Aristizábal et al. [2] adapted the Akkar et al. [1] GMPE which pro-
317 vides a good representation of the European context. The hazard curve built
318 from the generated catalog of the two synthetic ground motion model, is
319 shown in Figure 3b. It can be seen from this figure that the 10% of ex-
320 ceedance for 100 years ($\lambda_{IM} = 0.001$ 1/year) is 0.2 g for *Mod.B* and 0.5
321 g for *Mod.R*. Concerning the methods of each model, both are based on
322 stochastic simulations. They tend to directly simulate the recorded ground
323 motions with varied characteristics including the variability of the ground
324 motion [45, 61, among others]. For *Mod.R*, the method consists in rotat-
325 ing the recorded ground motion pairs into their principal axis and choosing
326 only the strong component. As for *Mod.B*, the stochastic method consists

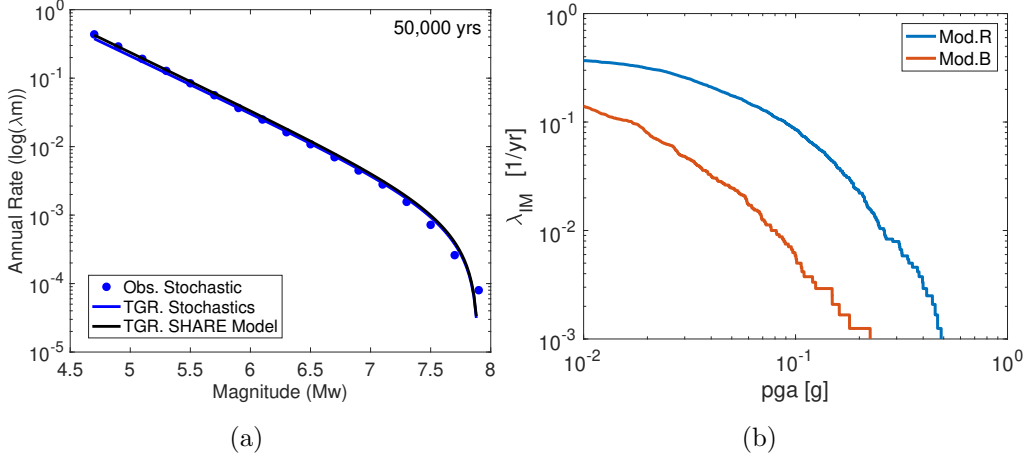


Figure 3: a) The magnitude-frequency distribution curve for 50,000 years catalog [2] along with b) the PGA hazard curve of the generated synthetic ground motions

327 in distributing randomly the energy over a duration equal to the inverse of
 328 the low frequency corner. The different ground motion parameters can be
 329 obtained by using the random vibrating theory [8]. For the sake of brevity,
 330 the deeper details and equations of each stochastic model are omitted, it is
 331 recommended to refer to each cited paper for more information. The spectral
 332 response of the two models is drawn in Figures 4a and 4b. It is clear that
 333 the spectral acceleration of *Mod.R* is higher than *Mod.B*.

334 Because it is difficult to understand the complexity of the earthquakes from
 335 one parameter [30], and based on Kawase [29] there exists a proportional
 336 relation between the outcrop acceleration and the equivalent predominant
 337 frequency which is $T_{v,\alpha} = \alpha \cdot PGV / a_{max,out}$ with $\alpha = 4.89$. These three par-
 338 ameters are represented in Figure 5. *PGV* is represented as dashed lines.
 339 Although the two models represent the same site, they use different relation-
 340 ships to determine their parameters and in consequence the corresponding

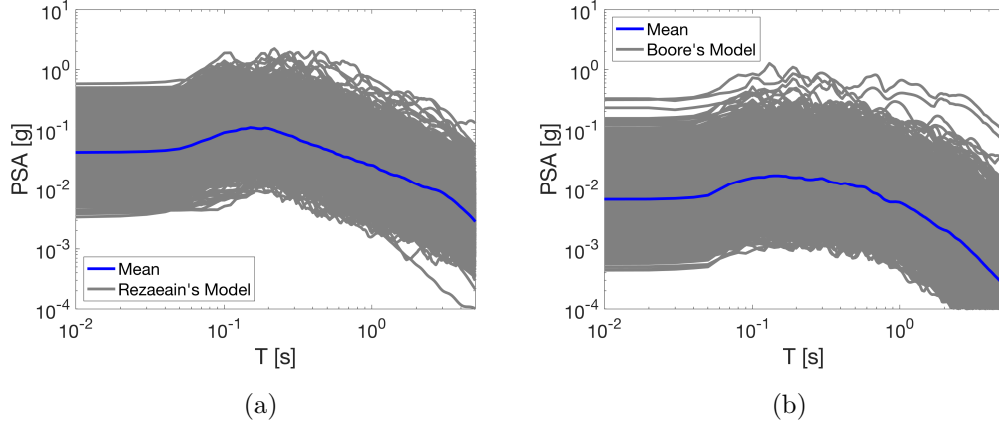


Figure 4: The response spectral of a) *Mod.R* and b) *Mod.B*

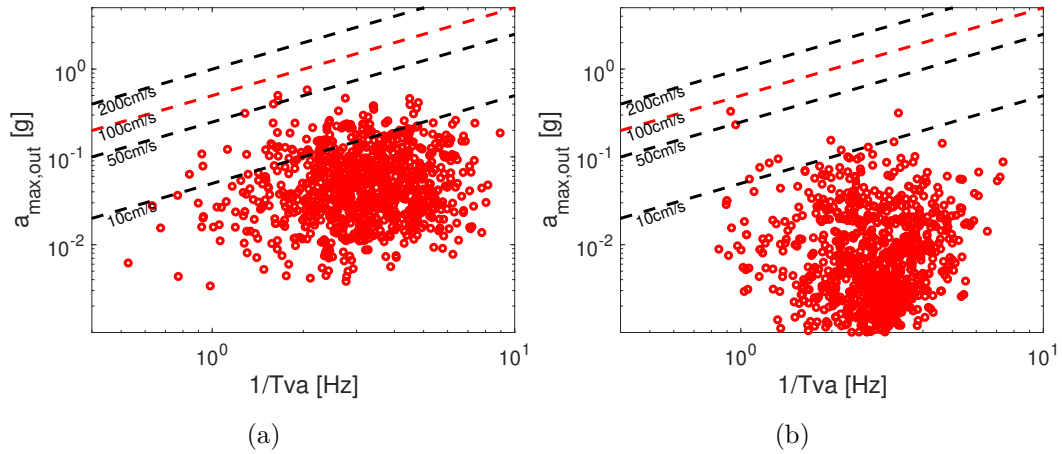


Figure 5: The distribution of some ground motion parameters of the two models: a) *Mod.R* and b) *Mod.B*

341 IMs. This proves the difference in the results shown in Figure 5. They are
 342 compatible in their frequency interval but not in their peak acceleration;
 343 *Mod.R* has a higher acceleration than *Mod.B*. Also, the majority of the motions
 344 of *Mod.B* have a *PGV* less than 10 cm/s which is not the case for

345 *Mod.R.*

346 After identifying and presenting the different synthetic ground motion
347 models used in this study, a site-specific seismic analysis will take place. The
348 response of the embankment will be calculated after a set of unsequenced
349 ground motion records.

350 **6. Site-Specific Seismic Analysis**

351 In Section 5, the used synthetic ground motions were presented. Two
352 models were used (*Mod.R* and *Mod.B*). In this section, the response of the
353 embankment based on each seismic load will be developed. Since the crest
354 settlement is the mode of failure normally studied in case of embankments,
355 it will be the parameter for the damage quantification. It is calculated by
356 considering each ground motion as a single event. The percentage relative
357 crest settlement as calculated by Swaisgood [58] is the ratio of the vertical
358 displacement of the crest to the height of the dam with its corresponding
359 foundation: $\delta u_{z,rel}/H$, given that H in this study is 19 m. The relative crest
360 settlement is divided into damage levels [58, 34]. The limit values of these
361 levels is still debatable but the ones chosen for this study are shown as dashed
362 lines in Figure 6. When $\delta u_{z,rel}/H \leq 0.02\%$, there is *No* damage, if 0.02%
363 $< \delta u_{z,rel}/H \leq 0.1\%$, the damage is *Minor*, if $0.1\% < \delta u_{z,rel}/H \leq 1\%$, the
364 damage is *Moderate* and finally if $\delta u_{z,rel}/H > 1\%$, the damage is *Serious*.
365 Figure 6 shows the relative crest settlement obtained using the two models:
366 *Mod.R* and *Mod.B*. As expected and as seen in Figure 6, the relative crest
367 settlement $\delta u_{z,rel}/H$ increases with the peak ground acceleration for both
368 models. Since the acceleration of *Mod.R* is higher than *Mod.B* (proved in

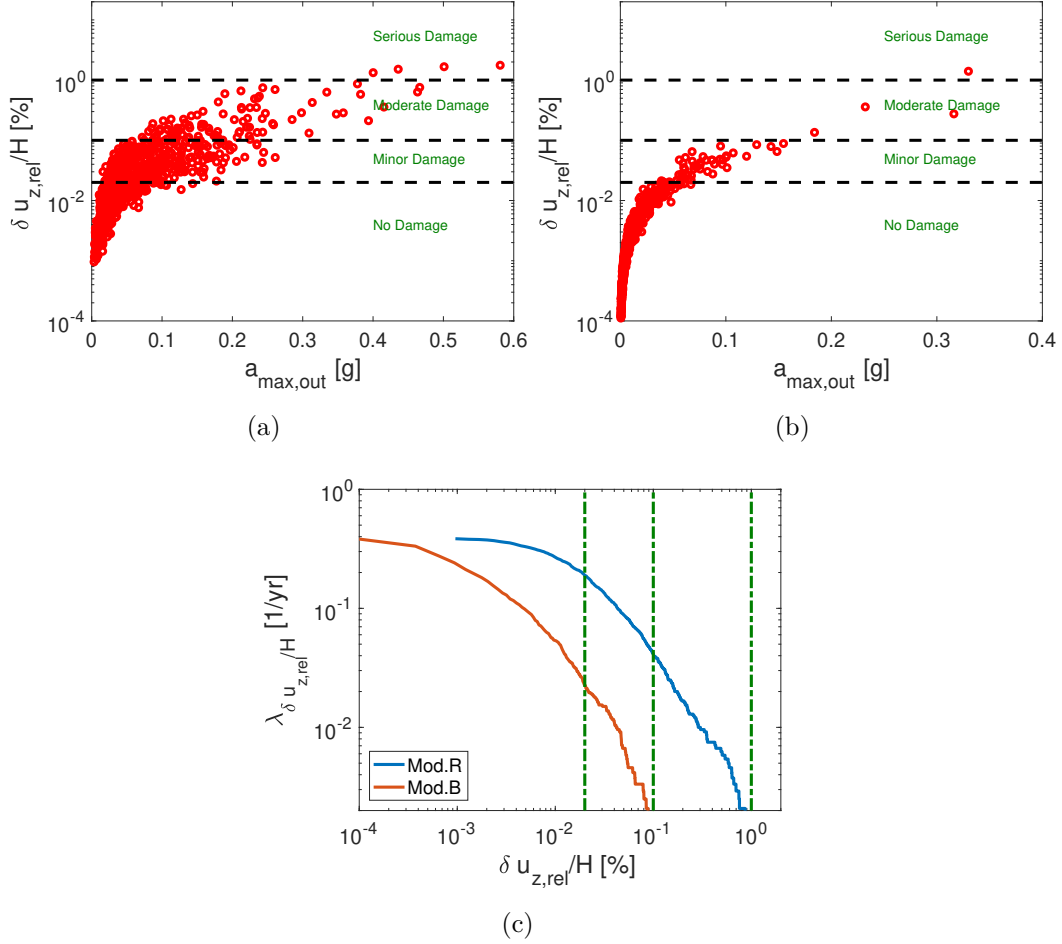


Figure 6: The variation of the crest settlement with respect to the outcrop acceleration of the two models: a) *Mod.R* and b) *Mod.B* and c) the obtained crest settlement hazard curve.

369 Section 5), *Mod.R* induces more damage than *Mod.B*. Comparing the damage
 370 levels, it can be seen that *Mod.B* in majority, did not show any damages;
 371 few values of $\delta u_{z,rel}/H$ were in *Minor* damage and only one shows *Serious*
 372 damage. Whereas *Mod.R* shows more variability in the damage levels that

373 lies in majority in *Minor* and *Major* damages. But on the contrary of Figure
374 6b, only four ground motions showed *Serious* damage for *Mod.R*. In addition,
375 a hazard curve for the crest settlement was obtained using the two stochastic
376 models (Figure 6c). The damage levels are also represented as dashed green
377 lines in this figure. It can be seen for example that with a 10% of exceedance
378 in 100 years, the obtained co-seismic settlement using *Mod.R* corresponds
379 to the *Serious damage* level. Whereas for *Mod.B*, for the same probability,
380 *Moderate damage* is the corresponding level.

381 It was seen in this section that the two synthetic ground motion models show
382 different results in terms of the embankment performance. These results
383 highlight the importance of the choice of the ground motion model.

384 7. Survival Analysis of the Levee

385 Also in the scope of the PBEE methodology, the lifetime of the structure
386 is the length of time until failure occurs. In order to calculate it, the degra-
387 dation of the structure over time should be considered. Thus, the study
388 of its performance due to sequential loading is required. In Section 6, it
389 was shown that the two synthetic ground motion models gave different re-
390 sponses and that *Mod.R* induced more damage. Thus, this model will be
391 used to compute the seismic sequential loads as well as the survival life of
392 the tested embankment. Assuming that the working life of the embankment
393 (i.e. T_{window} in Figure 1) is 100 years, the occurrence rate of event during
394 this life corresponds to 44 acceleration time histories (i.e. $N_{shock} = 44$) which
395 means $\lambda_{earthquake} = 0.44$ events/year. In order to be statistically representa-
396 tive, a large number of subsets must be used. In this work, 21 subsets (i.e.

397 $N_{subset} = 21$ in Figure 1) compatible with the seismic hazard of the tested
398 site were used. The 44 events were permuted randomly 10 times (i.e. $k =$
399 10 in Figure 1) for each subset. In total, 210 sequences of ground motions
400 were created for a serviceable life of 100 years. It should be reminded that
401 for the sequential computation, the storage of the history variable of the i th
402 computation will be used as initial state of the $i+1$ computation.
403 This section is divided into two parts. The first part develops the quantifi-
404 cation of the relative crest settlement of the embankment for each sequence.
405 The second part develops its survival analysis.

406 *7.1. Crest settlement in the sequences*

407 Based on Iervolino et al. [25], the damage accumulation is due to either the
408 aging of the material or the sequential earthquakes. The first degradation
409 model is called the deterioration-based: the system degrades progressively
410 due to internal factors such as aging, corrosion in steel or wear [40, 42].
411 The second degradation model is the shock-based model where the system
412 is subjected to a sudden decrease in its performance due to an earthquake
413 [40, 42]. In this work, the aging is not taken into consideration because
414 it needs a deeper understanding of the material origin, resistance and age.
415 As explained in the introduction of this section, the event of interest is the
416 occurrence time of different mainshocks sequences.

417 In order to ensure that the pore water pressure is dissipated completely after
418 each mainshock (the sixth assumption in this study), a recovery time of 30
419 seconds is considered. It was chosen in a way to ensure the dissipation of Δp_w
420 without the generation of expensive computational time. In order to validate
421 if this recovery time is enough for the dissipation of Δp_w , one input ground

422 motion is considered and a post-seismic loading is applied. The results are
 423 shown for a sample located at 3 m depth under the center of the embankment
 (Figure 7a) and at free field (Figure 7b). It is interesting to note that after

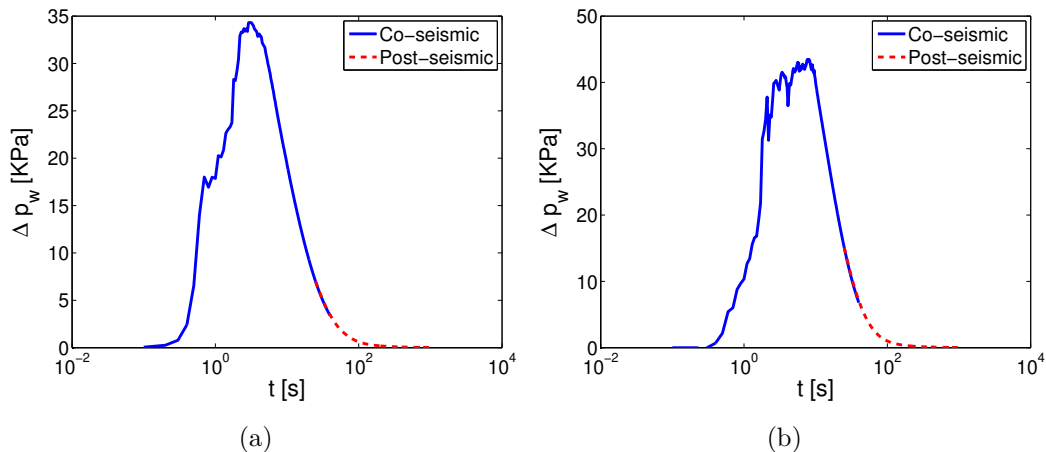


Figure 7: The excess pore water pressure of one mainshock a) under the center of the embankment and b) at free-field

424
 425 the co-seismic loading, the excess pore water pressure is near zero in the two
 426 locations. The post-seismic loading, where the embankment is returned to
 427 its static case, proves that chosen recovery time can be enough since the two
 428 curves overlapped and follow the same path.

429 Concerning the damage quantification of each sequence, the relative crest
 430 settlement $\delta u_{z,rel}/H$ is calculated based on the method developed in Section
 431 6. Figure 8a shows $\delta u_{z,rel}/H$ and the corresponding damage levels for all the
 432 seismic sequences. To better visualize the results and conduct the analysis,
 433 Figure 8b shows $\delta u_{z,rel}/H$ in the form of box-plots (aka Speaker style). A
 434 small explanation about this form of graph is discussed herein before pro-
 435 ceeding in the discussion of the results. The median of the tested data is

436 represented by the small bars in the boxes. The boxes represent 50% of the data
 437 data and there boundaries considers 25th and 75th percentile of the data
 438 respectively. The two extremes are 1.5 times the distance between the two
 439 percentiles, and the red dots above or below the box-plots are the outliers.

Back to the analysis of the relative crest settlement of *Mod.R* sequences

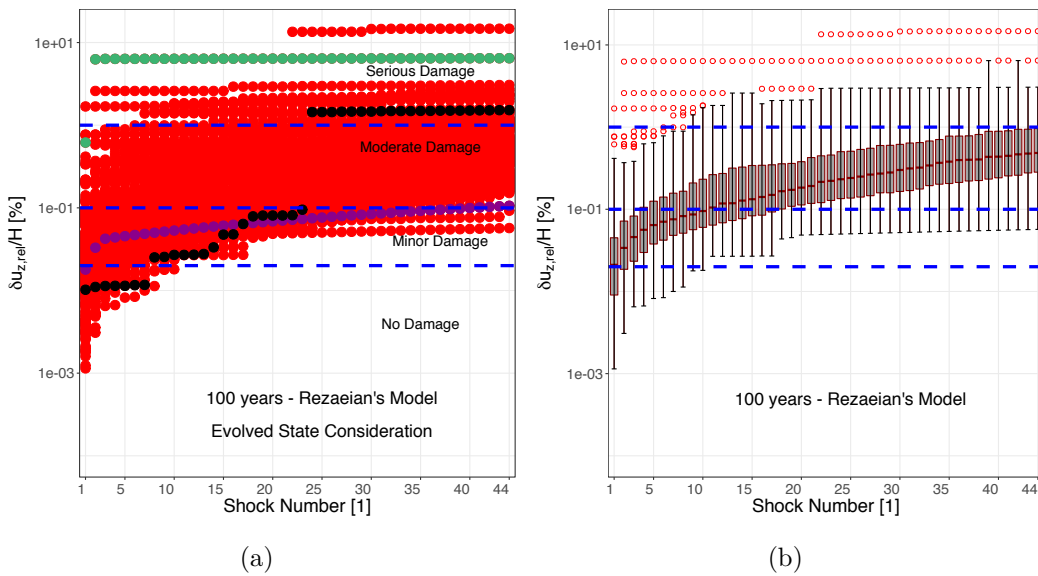


Figure 8: The relative crest settlement of the sequential signals a) for all the sequences and b) in form of box-plots

440

441 and as expected, $\delta u_{z,rel}/H$ increases when the number of shocks increases
 442 (Figures 8). Based on these results, the increment was either progressive
 443 during the sequential load, or sudden after few mainshocks. For this pur-
 444 pose, three curves are selected for interpretation. For example, the purple
 445 curve in Figure 8a shows that after the first shock, the embankment had
 446 *No* damage. The relative crest settlement kept increasing during this se-
 447 quence but in a small manner. At the end of this sequence, the embankment

448 showed a damage level located in the interface between *Moderate* and *Minor*
449 damage. This case can be considered “safe”, in the point of view that the
450 cumulative damage did not lead to failure. On the opposite, the green path
451 shows a *Moderate* damage directly after the first shock. The embankment
452 was not able to resist the load repetition and hence it failed drastically after
453 the second shock till the end. For the path in black, the damage was pro-
454 gressive. The embankment had *No* damage until the seventh shock. After it,
455 $\delta u_{z,rel}/H$ increased progressively until after the 23rd shock, the embankment
456 was not able to resist and the induced damage was drastic. It should be
457 mentioned that previously, in Section 6, only four ground motions showed
458 *Serious* damage. The occurrence of these motions in the sequence, either led
459 to a good densification of the soil and hence less damage or it increased the
460 crest settlement to a severe case.

461 Another statistical way to visualize the results is found in Figure 8b.
462 Based on the median value, $\delta u_{z,rel}/H$ increases with the shock increment.
463 For the first shock, 50% of the data showed *Minor* damage. But after the
464 44th shock, 50% of the data showed *Moderate* damage. The upper quarter
465 of the data showed *Serious* damage. The outliers are presented in the upper
466 part of the graph which proves that the embankment fails completely for
467 some cases at the beginning of the sequences. An associated explanation is
468 also represented in Figure 9 with the empirical Complementary Cumulative
469 Distribution (CCDF). This figure shows the distribution of the relative crest
470 settlement after a specified number of shocks. The damage levels are also
471 represented by blue dashed lines. For the damage line 0.1% for example,
472 the distribution of the relative crest settlement increases when the shock

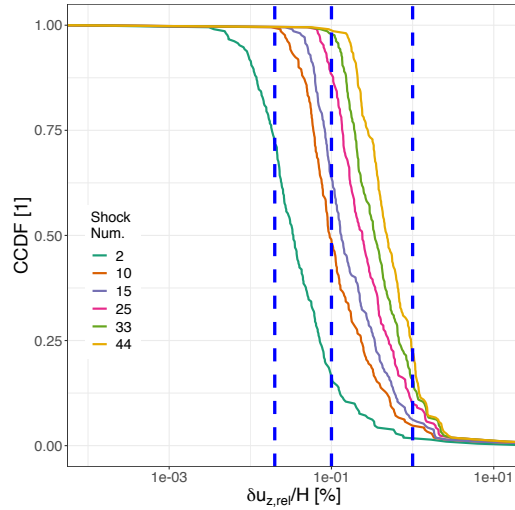


Figure 9: The complementary cumulative distribution function (CCDF) of the relative crest settlement

473 number increases. After the 10th shock, 50% of the tested sequences shows
 474 that the embankment will be deteriorated in a *Moderate* damage. Usually
 475 in engineering practices, the green curve in Figure 9, is similar to the ones
 476 conducted in Section 6 where the response of the embankment is calculated
 477 after the occurrence of independent earthquakes. It was shown so far in this
 478 work, that this curve can be misleading and may not represent the behavior
 479 of the embankment for a long term.

480 On the other hand, since the ground motions sequences are chosen for
 481 a working life of 100 years, the change in the damage levels between the
 482 years is interesting to identify. Figure 10 shows the Damage Level (DL) of
 483 the selected ground motion as function of the magnitude and the source-
 484 to-site distance. Since the acceleration time histories during the lifetime are
 485 permuted randomly, it is evident that the ground motions are not in the same

486 position in the first, 50 and 100 years. The damage levels are numerated from
 1 to 4 to represent *No* damage to *Serious* damage respectively. From Figure

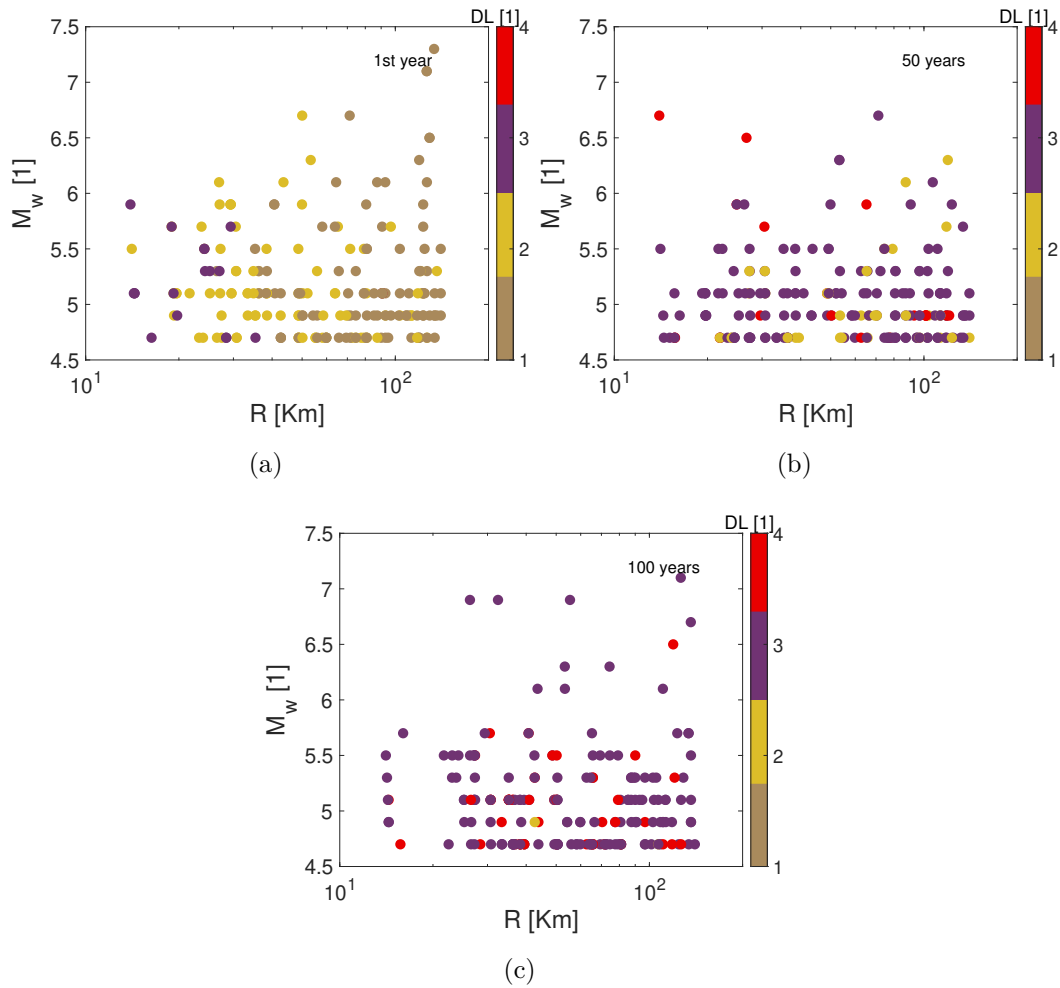


Figure 10: The damage levels of *Mod.R* with respect to the magnitude and distance of the seismic motions

487

488 [10a](#), after the first ground motion, very few responses show damage level of
 489 class 3. The other responses were either 1 or 2. After 50 years of sequential
 490 ground motions (Figure [10b](#)), it is clear that there is no damage level of

491 class 1 and very few of class 2. Finally, at the end of the chosen working
492 life (i.e. 100 years), more damage levels of class 4 are shown (Figure 10c),
493 which means that the embankment have a high risk to fail at the end. This
494 synthesis is similar to the one found previously in this section with a different
495 type of analysis.

496 As a partial conclusion of this section, it is noted that the loading history
497 of the embankment will definitely affect its behavior in the long term. Even
498 if the embankment was intact after few shocks, the failure can be reached in
499 the working life. In addition, some sequences showed that the embankment
500 was drastically damaged after few shocks.

501 7.2. Survival analysis

502 The survival analysis is the analysis of data involving time needed for an
503 event of interest to happen. It is also known as the time-to-event data. De-
504 tailed explanation about this analysis were developed in Section 2. The event
505 of interest in the scope of this study, is the occurrence of mainshocks during
506 a working life of 100 years. For this purpose, the survival function is calcu-
507 lated based on the Kaplan-Meier estimator [27]. The estimated probability
508 is usually a step function and is shown in Figure 11. The three damage levels
509 are represented in this figure. An important quantity that can be derived
510 from this figure, is the Mean Time To Failure (MTTF, the expected time to
511 failure for a non-repairable system) that is also represented. After two years
512 (i.e. $MTTF = 2.27$ years), 50% of the cases survived the damage level of
513 class 2 (i.e. *Minor* damage). Whereas the MTTF of class 3 (i.e. *Moderate*
514 damage) is 25 years. At 100 years, the survival probability of this damage
515 level is zero. For *Serious* damage, 25% of the cases were not able to survive.

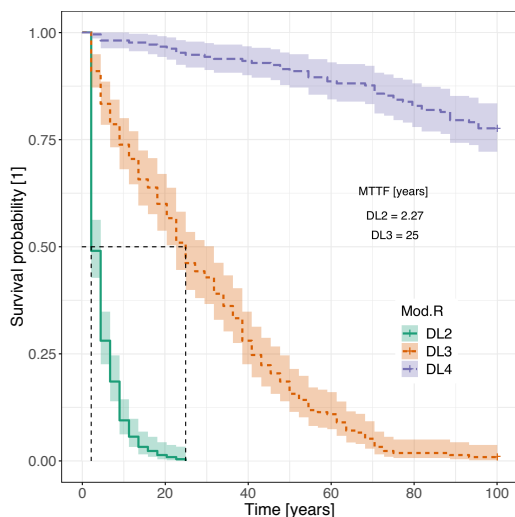


Figure 11: The survival probability of the levee for *Mod.R*

516 In all the preceding, the survival function was conducted based on a
 517 non parametric approach. It is important to mention that even when the
 518 parameters of the distribution are unknown, distributions such as Weibull,
 519 lognormal, exponential or others, can be fitted from results obtained from
 520 non parametric methods. The accuracy of this fitting is ensured by the Kol-
 521 mogorov–Sminorv (KS) test [32]. It was carried out for each empirical dis-
 522 tribution of the two critical damage levels. Both, the Weibull and its special
 523 case the exponential, could be considered according to the KS test. It should
 524 be noted that the distribution of the survival function in case of Weibull is
 525 $S(t) = \exp(-\alpha t^\gamma)$ and in the case of exponential is $S(t) = \exp(-\lambda t)$. Ta-
 526 ble 1 shows the obtained parameter of each damage level from the survival
 527 function found in Figure 11. The interpretation of these results are pointing
 528 on the idea that the Weibull distribution is more appropriate to represent
 529 the slow deterioration overtime whereas the exponential distribution can be

530 used to represent the deterioration after sudden shocks. Since the *Moderate*
 531 damage was mainly due to the progressive increase of the crest settlement, its
 532 distribution can be approximated to follow a Weibull distribution. Whereas
 533 *Serious* damage is more likely to follow the exponential distribution because
 the failure was sudden in most of the cases. Thus, the results obtained for

	Weibull		Exponential
	α	γ	λ
<i>With Recovery Time (Section 7.2)</i>			
DL3	0.058	0.909	0.037
DL4	$1.9 \cdot 10^{-3}$	1.057	$2.5 \cdot 10^{-3}$
<i>No Loading History (Section 8.1)</i>			
DL3	0.065	1.344	0.014
DL4	$5.77 \cdot 10^{-5}$	2.387	0.018
<i>No Recovery Time (Section 8.2)</i>			
DL3	0.024	1.175	0.044
DL4	$9.19 \cdot 10^{-5}$	1.961	0.007

Table 1: The distribution parameters of *Moderate* and *Serious* damages for the three tested cases of this work

534

535 both distributions confirm the biases that could be introduced in a paramet-
 536 ric approach when the survival function is not well known.

537 8. Influence of various parameters on the survival analysis

538 To this point of this article, it was represented an analysis of the relative
 539 crest settlement of an embankment after both, a single event and sequential

540 events of mainshocks. This later considered sequences of 44 mainshocks
541 with a recovery time between each. In addition, the survival function was
542 calculated based on a non parametric approach. In this section, answers on
543 the following questions will be discussed respectively:

- 544 • What if the loading history was not considered? Technically, what will
545 happen if the embankment was returning to its initial state after each
546 ground motion of each sequence.
- 547 • What if there was no recovery time between the GM which means that
548 the mainshocks are sequenced in a back-to-back form?
- 549 • How the response will differ if the analysis was conducted based on a
550 different synthetic ground motion model (e.g. *Mod.B*)?

551 8.1. Effect of the loading history

552 In soil precisely, the history of loading plays a major role for its future
553 behavior. Many previous studies have been conducted to check the effect of
554 past histories on the response of geo-structures (i.e. Sica et al. [56], Lopez-
555 Caballero et al. [36]). Moreover, it was seen in the previous section, that
556 even if the embankment was intact after few years, it might be damaged in a
557 long time. Its state evolves based on the years of loading. Considering now
558 the sequences are formed by simply adding the responses found in Figure
559 6a based on their occurrence, Figure 12a will be obtained. Based on this
560 figure, the results did not show too much dispersion and few outliers, on the
561 contrary of the results in Figure 8b. The embankment started its cycle of life
562 with a *Minor* damage, to attend a *Serious* damage after half of its lifetime.

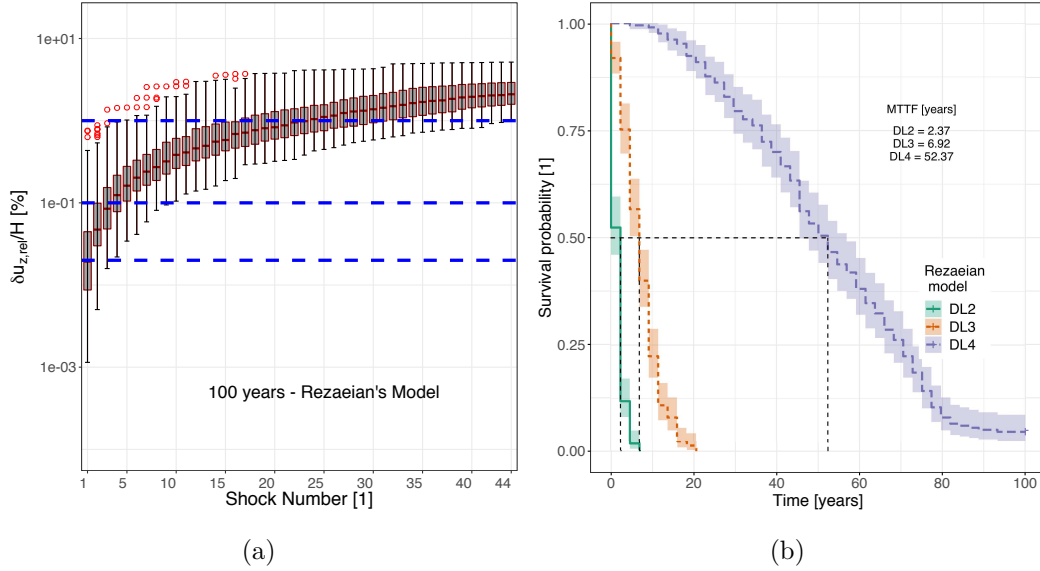


Figure 12: The relative crest settlement with No loading history a) in the form of box-plot and b) its survival analysis

563 The difference between the response of the embankment with consideration
 564 of its evolved state (Figure 8b) and that presented in Figure 12a is that the
 565 last one over estimates the results. The embankment settles less when in its
 566 history, it was subjected to many loads: this is due to the consolidation of
 567 the soil and the evolution of its properties.

568 Concerning the survival analysis of this case (Figure 12b), after approxi-
 569 mately 7 years (i.e. MTTF = 6.92 years), the embankment will moderately
 570 survive the applied sequential loads. Whereas after almost half of its service-
 571 able life (i.e. MTTF = 52.37 years), it will be damaged drastically. These
 572 results are very different from the ones obtained in Section 7.2. The param-
 573 eters of the survival function distributions are shown in Table 1.

574 Maybe this analysis satisfies the decision makers as they consider it “preven-

575 tive” for security reasons, but besides the fact that it is not realistic, it could
576 mislead the design and may generate additional useless costs.

577 8.2. Importance of the recovery time between the sequences

578 Based on codes and literature, a system is considered to fail when its cu-
579 mulative damage due to shocks exceeds its capacity of resistance [16, 25, 42,
580 among others]. On the opposite of the structural systems, geotechnical sys-
581 tems have in majority, recovery times that if not considered in the analysis, it
582 may lead to severe and over estimated damages. In this section, a comparison
583 of the survival function of three different types of shock-based approaches is
584 considered. The approaches are i) the consideration of the the recovery time
585 between each shock (Section 7.2), ii) taking the shocks in a back-to-back form
586 and iii) the consideration of the loading history (Section 8.1). The results are
587 shown in Figure 13. The survival probability was analyzed for two damage
588 levels: *Moderate* damage ($0.1 < \delta u_{z,rel}/H < 1$) in Figure 13a and *Serious*
589 damage ($\delta u_{z,rel}/H > 1$) in Figure 13b.

590 For both damage levels, it can be seen that the loading history plays a
591 major role in the response of the embankment: without its consideration,
592 the relative settlement was over estimated. This over estimation may be
593 beneficial if the region was of low seismic activity. But for some cases, it can
594 be a result of high cost of construction/reparation. Considering the *Moderate*
595 damage (Figure 13a) and comparing the orange and the green curves, it can
596 be noticed that there are five years of survival life that are not taken into
597 account if the recovery time was not ensured. In addition, it is clear, from the
598 different values of the MTTF, that loading history has an important effect.
599 Whereas for the *Serious* damage (Figure 13b), it is important to mention

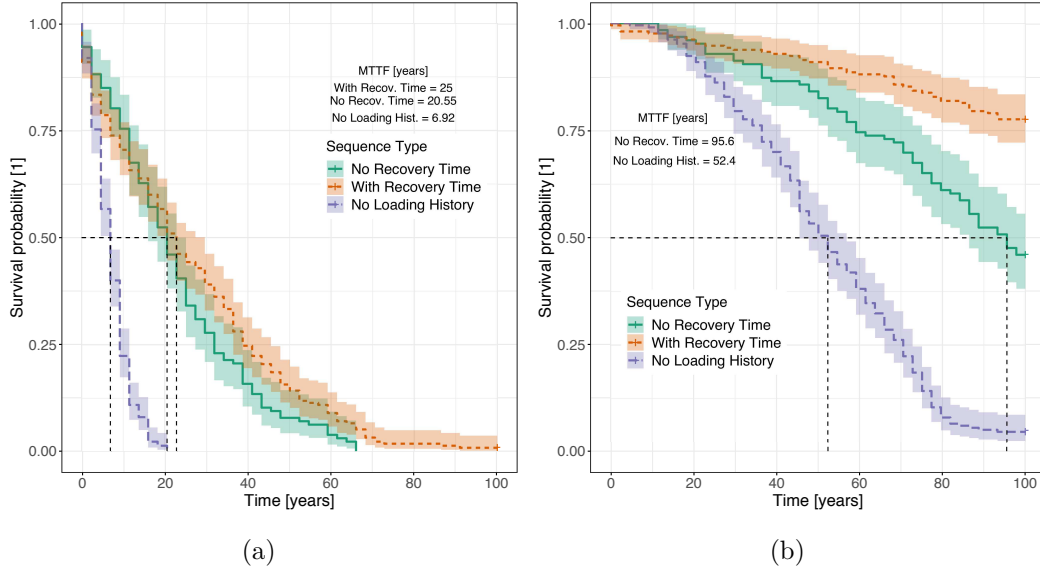


Figure 13: The survival analysis of the different sequence types for a) Moderate damage and b) Serious damage

600 that the embankment did not reach the MTTF when the recovery time was
 601 ensured. The parameters of the survival function distribution are shown in
 602 Table 1.

603 8.3. Effect of different synthetic ground motion models

604 The survival analysis in this paper was conducted based on the synthetic
 605 ground motion model called *Mod.R* since it showed more variety in the re-
 606 sponse of the embankment and its damage level (Section 6). This section will
 607 consider *Mod.B* in a back to back form (or with no recovery time) since this
 608 type of calculations needs less computational time. Adopting the same strat-
 609 egy developed in this paper to calculate the survival functions, the results
 610 are obtained in Figure 14 as function of two damage levels: *Minor* damage

611 (DL2) and *Moderate* damage (DL3). Also in this figure, a comparison of
 612 the survival function with *Mod.R* in back-to-back form is shown. It is clear
 613 how the survival function for *Mod.B* is very optimistic and shows that the
 614 embankment is able to resist the shocks during its lifetime. At 41 years for
 615 example, there is 50% chance that the embankment will survive a damage
 616 level of DL2 if it was subjected to *Mod.B*, whereas based on *Mod.R* it needs
 617 2 years (Figure 14a). Also for DL3, based on *Mod.B*, the embankment have
 618 high chances to survive this damage level whereas based on *Mod.R*, after 20
 years, it will have only 50 % chance to survive it (Figure 14b). It is evident

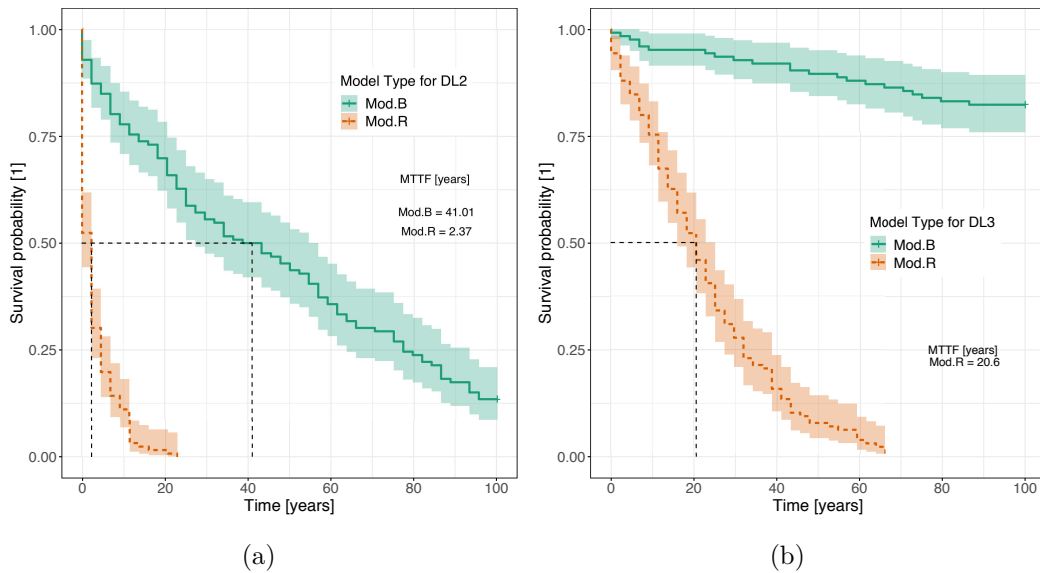


Figure 14: The survival analysis of *Mod.B* in back-to-back form for a) DL2 and b) DL3

619
 620 that the response varies with the ground motion model, which points on the
 621 importance of the choice of the stochastic method to generate the ground
 622 motions.

623 9. Conclusions

624 This paper presents the survival analysis of a liquefiable embankment
625 subjected to sequential earthquakes. First, it started with a site-specific
626 seismic analysis where the damage was quantified after a set of unsequenced
627 ground motion records. To generalize this work, two synthetic ground motion
628 models were used. They were extracted from the studies of Rezaeian and
629 Der Kiureghian [45] and Boore [7] and were designated in this paper as
630 *Mod.R* and *Mod.B* respectively. Then, one synthetic model (e.g. *Mod.R*)
631 was chosen for the analysis. Sequential mainshocks were created accordingly,
632 and the damage was calculated after each sequence. A total of 210 sequences
633 composed of 44 acceleration time histories each, were generated in order
634 to represent a lifetime of 100 years. Finally, the survival function and its
635 corresponding Mean Time To Failure (MTTF) were calculated. For these
636 purposes, an elastoplastic multi-mechanism soil behavior model was used
637 with the help of a 2D finite element code (*GEFDyn*).

638 The conclusions that this paper have found are the following:

- 639 • For the site-specific seismic analysis, *Mod.R* showed a large variety of
640 the response than *Mod.B*. The relative crest settlement of this model
641 was mainly between *Minor* and *Moderate* damage levels.
- 642 • The cumulative damage in this study showed two responses for the
643 embankment: i) a progressive deterioration that did not necessarily
644 lead to drastic damages or ii) a sudden deterioration after few years.
- 645 • Based on the survival function, it was shown that after 25 years (MTTF
646 = 25 years), the embankment have 50% chance to present *Moderate*

647 damage. Whereas it will not show *Serious* damage during its lifetime.
648 These results prove that classical or short-term analysis are not always
649 a good idea to know the global performance of the embankment.

650 • The distribution of the survival function for the *Moderate* damage level,
651 follows a Weibull distribution whereas that of the *Serious* damage level
652 is more likely to be represented by an exponential distribution. In
653 addition, parameters of each distribution were calculated for each type
654 of analysis.

655 • The consideration of the loading history for geo-structures is very im-
656 portant since the MTTF changed from 6 years to 25 years. In addition,
657 a recovery time between each ground motion is essential in order to en-
658 sure the dissipation of the excess pore water pressure and it was shown
659 in this study, that the MTTF is also affected.

660 • It is very important to pay attention to the chosen synthetic ground
661 motion model since the performance might highly be affected. The
662 MTTF for *Mod.B* was under estimated comparing to *Mod.R*.

663 It should be noted that the above conclusions are based on the results corre-
664 sponding to soil behavior models and stochastic GM models adopted in this
665 paper.

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