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► **To cite this version:**

Christina Khalil, Fernando Lopez-caballero. Lifetime response of a liquefiable soil foundation-embankment system subjected to sequences of mainshocks and aftershocks. *Soil Dynamics and Earthquake Engineering*, 2023, 173, pp.108107. 10.1016/j.soildyn.2023.108107 . hal-04149459

HAL Id: hal-04149459

<https://hal.science/hal-04149459>

Submitted on 5 Jul 2023

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Lifetime response of a liquefiable soil foundation-embankment system subjected to sequences of mainshocks and aftershocks

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Abstract

During their typical design working life, structures are subjected to multiple sequential earthquakes that are divided into clusters of mainshocks and aftershocks. In consequence, the induced seismic damage accumulates due to these several events in each cluster. On the other hand, the Performance Based Earthquake Engineering (PBEE) methodology presents limitations in accounting for the damage issued from multiple events and the fragility curves represent the failure probability only in one event of a given intensity.

For this purpose, this work studies the response of a given structure subjected to sequences of mainshocks - aftershocks during its lifetime. The used time histories (mainshocks) were stochastically generated from a synthetic ground motion model, whereas the aftershocks were generated from the Branching Aftershock Sequence (BASS) model. The cumulative damage measure of the

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embankment is calculated after the tested sequences. More importantly, its lifetime distribution is estimated and compared with previous results. Finally, to be compatible with the PBEE methodology, the fragility evolution of the embankment over its lifetime was calculated as to emphasize its evolution with the loading history in the soil.

Keywords: clusters, cumulative damage, lifetime distribution, fragility evolution

1 **1. Introduction**

2 In earthquake-prone regions, structures are exposed to seismic sequences
3 that are composed of mainshocks, aftershocks and foreshocks. Aftershock
4 events are usually triggered by the mainshocks due to the change in the static
5 and dynamic stresses during the earthquake process [46]. It was shown that
6 the aftershocks exacerbate the damage generated from the triggering main-
7 shock and may be the reason of collapse even though they have a smaller
8 magnitude than the triggering mainshock [69, 72, 47, 67, 57, among oth-
9 ers]. Otherwise, the structures are designed to have a useful serviceable
10 lifetime. But when they are subjected to multiple events over their lifetime,
11 the induced potential damage can be interpreted as the accumulation of the
12 damages due to all the occurred earthquake shocks [52, 20, 32].

13 In practice, from the Performance Based Earthquake Engineering (PBEE)
14 methodology, structures are designed to resist the first damaging earthquake
15 scenario. Hence, the classical fragility curves represent the failure probab-
16 ility of one event of a given intensity only. This means that this framework
17 neglects the cumulative damage and the evolution of the material proper-

18 ties resulted from multiple events [19]. In addition, the commonly used
19 Probabilistic Seismic Hazard Analysis (PSHA) to select the corresponding
20 ground motions, is intended to evaluate the hazards from discrete indepen-
21 dent events. Thus, empirical scaling laws (i.e. Gutenberg-Richter, modified
22 Omori’s law, Bath’s law) have been proposed to generate the aftershocks
23 occurrence [55]. These laws are based on parameters that depend on the
24 statistical properties of particular seismic sequences [55, 46, 61].

25
26 Previous works on buildings subjected to mainshock-aftershock events
27 have been conducted [30, 13, 48, 46, 12, 72, 47, 31, 11, 20, 57, 40, 21, 54,
28 among others]. In this context, researchers, have been recently interested
29 in conducting methodologies to assess the fragility exposure of structures
30 subjected to mainshock-aftershock events [41, 19, 9, 66, 73, among others].
31 Therefore, evaluating the seismic performance of structures subjected to se-
32 quential seismic events, specially when dealing with the aftershock occur-
33 rence, requires explicit consideration of the uncertainty in the state of the
34 structure [57]. For example, the known *Incremental Dynamic Analysis (IDA)*
35 that assesses multiple realizations to capture record-to-record response vari-
36 ability requires the scaling of the used seismic inputs [58, 65, 20, among
37 others]. Then, failure is identified once the structure exceeds a threshold
38 limit. However, in liquefaction related problems, several aspects are impor-
39 tant to take into account. For example, the multi-physical aspects of the soil,
40 its history of loading and its correlation with several intensity measures, are
41 all important aspects that makes the IDA approach unrepresentative of the
42 overall response of the geo-structure [59, 6, 36, 26].

43

44 Otherwise, several analysis exist in order to calculate or estimate the life
45 cycle of the structure [69, 52, 48, 20, 45, 51, 32, among others]. The *Survival*
46 *Analysis* has been a used approach in geotechnical engineering to identify
47 the relative risks of various earthquake [39, 32, 26]. It is defined as the time
48 length of the structure until the occurrence of an event of interest (i.e. equip-
49 ment failure, damage, complex system) [15, 39, 7, 10, 32, 26]. It accounts for
50 a set of statistical methods to analyze data that has the occurrence time of an
51 event as the outcome. It calculates the probability of survival of concerned
52 test data and estimates its Mean Time to Failure (MTTF).

53

54 In the work of Khalil and Lopez-Caballero [26], the lifetime distribution
55 of an embankment subjected to mainshock sequences was estimated . These
56 later were extracted from several subsets compatible with the seismic hazard
57 of a site of concern. The site seismicity as well as the occurrence rate of
58 mainshocks were extracted from the work of Aristizábal et al. [2]. Each
59 subset was also in accordance with the event rate of the mainshocks and
60 the generated time histories were permuted many times in order to take into
61 account the randomness of the events. As a continuity of the work of Khalil
62 and Lopez-Caballero [26] and for the same geometry and numerical model,
63 this study will try to answer the following significant questions:

- 64 a) Is the cumulative damage of the embankment influenced by the se-
65 quence type (i.e. mainshocks sequences or mainshock-aftershock se-
66 quences) ? (Section 6)
- 67 b) Does the aftershock occurrence affects the lifetime distribution of the

68 embankment, as well as its MTTF ? (Section 7)

69 c) How do the fragility curves evolve after multiple loading histories ?
70 Does this evolution depend on the type of the applied load ? (Section
71 8)

72 In order to answer these questions, it should be mentioned that the
73 methodology and the PSHA of the site of concern are similar to the ones
74 presented in the work of Khalil and Lopez-Caballero [26]. A large number
75 of time histories is generated using stochastic simulations from the synthetic
76 ground motion model of Rezaeian and Der Kiureghian [44]. The model used
77 to generate the aftershocks is based on the probabilistic version of the Branch-
78 ing Aftershock Sequences (BASS) model and is inspired by Turcotte et al.
79 [63] and Hu et al. [16]. Also in this paper, an interpretation of the potential
80 failure of the embankment is developed in order to understand what happens
81 globally in the embankment, and locally in the soil.

82 **2. Geometry and Numerical Model**

83 *2.1. Geometry*

84 The model's geometry is a levee of 9 m height composed of dry dense
85 sand. The foundation is formed of 4 m loose to medium sand (LMS) on the
86 top of a 6 m dense sand. The bedrock is located under the dense sand. The
87 water table starts 1 m below the surface to keep the embankment dry. The
88 inclination of the levee is a slope of 1:3 (vertical: horizontal). The geometry
89 in this work is inspired by Rapti et al. [43], Lopez-Caballero and Khalil [33],
90 and is detailed in Figure 1.

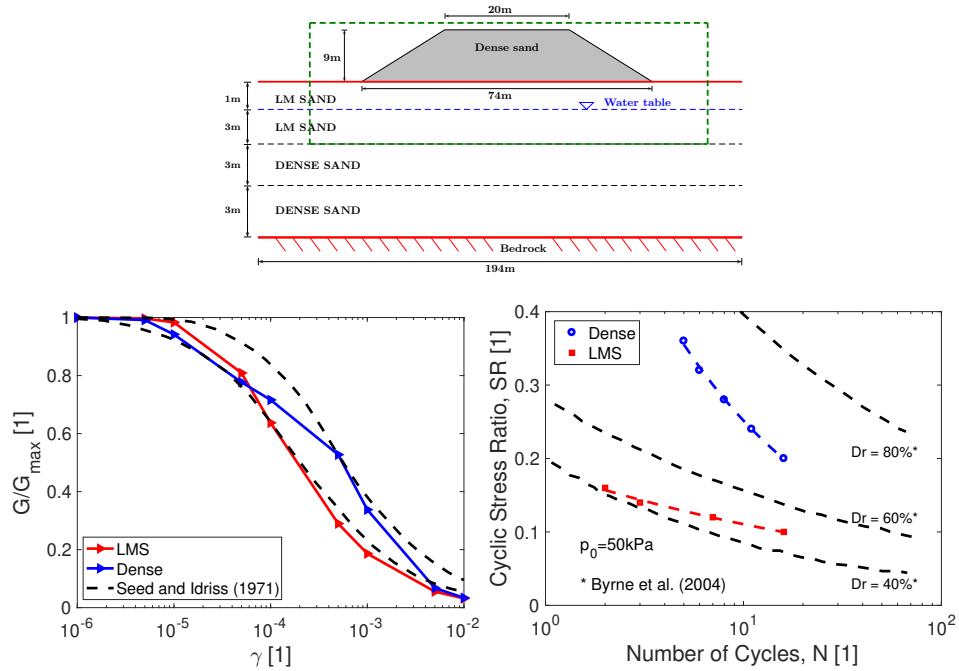


Figure 1: Geometry and behavior of the soil [33]

91 *2.2. Soil Constitutive Model*

92 As for the constitutive model, the *Ecole Centrale Paris (ECP)* elastoplas-
 93 tic multi-mechanism model (also known as *Hujeux* model) is the one chosen
 94 for this study and is written in terms of effective stress. The non-linearity
 95 of this model is represented by four coupled elementary plastic mechanism:
 96 three plane-strain deviatoric plastic strain mechanism in three orthogonal
 97 planes (k - planes) and an isotropic plane to take into account normal forces.
 98 The model follows a Coulomb type failure criterion, contemplate the existe-
 99 nce of dilatancy/contractancy phenomena, and use the critical state con-
 100 cept. The cyclic behavior is taken into account by a kinematical hardening
 101 that is based on the state variables at the last load reversal. The model is
 102 written in the concept of the incremental plasticity which divides the total

103 strain into an elastic and a plastic part. Refer to Aubry et al. [4], Hujeux [18]
 104 and Lopez-Caballero and Modaressi-Farahmand-Razavi [34], among others,
 105 for further details about the ECP model. For the sake of brevity, only some
 106 model definitions will be developed in the following. Considering the well-
 107 known sign convention in soil mechanics which sets the positive sign to the
 108 compression forces, the yield surface of this numerical model is written in
 109 the k plane as follows:

$$f_k(\sigma, \varepsilon_v^p, r_k) = q_k - \sin \phi'_{pp} \cdot p'_k \cdot F_k \cdot r_k, \quad (1)$$

110 where p'_k and q_k are the effective mean and deviatoric values of the stress ten-
 111 sors and ϕ'_{pp} is the friction angle at the critical state. The parameters that
 112 control the behavior of the soil are F_k , which controls the isotropic harden-
 113 ing associated with the plastic volumetric strain and r_k , which controls the
 114 isotropic hardening generated by the plastic shearing. These two parameters
 115 represent progressive friction mobilization in the soil. At perfect plasticity,
 116 the product $F_k \cdot r_k$ reaches unity, and the Mohr-Coulomb criterion will be
 117 satisfied. Therefore, in order to measure the “distance to reach the criti-
 118 cal state”, the variable r_{apt} is calculated based on the adopted elasto-plastic
 119 numerical model and an apparent friction angle ϕ'_{apt} is defined by:

$$\sin \phi'_{apt} = \frac{q_k}{p'_k \cdot F_k} \quad (2)$$

$$r_{apt} = \frac{\sin \phi'_{apt}}{\sin \phi'_{pp}} \quad (3)$$

120 2.3. Finite Element Model

121 The computations were conducted using the coupled FE modelling code
 122 GEFDyn [3], using a dynamic approach derived from the $\underline{u} - p_w$ (i.e. \underline{u} being

123 the displacement tensor and p_w being the pore water pressure) version of
124 the Biot's generalized consolidation theory [74]. The FE model is composed
125 of quadrilateral isoparametric elements ($3.5 \text{ m} \times 1 \text{ m}$) with eight nodes for
126 both solid displacements and fluid pressures. An implicit Newmark numer-
127 ical integration scheme with $\gamma = 0.625$ and $\beta = 0.375$ was assumed in the
128 dynamic analysis [29]. The FE analysis is performed in three consecutive
129 steps: i) a computation of the initial in-situ stress state due to gravity loads;
130 ii) a sequential level-by-level construction of the embankment and iii) a se-
131 quential seismic loading analysis in the time domain. For the computation of
132 the sequential seismic loading, for the first motion precisely, its final effective
133 stresses, pore-water pressures and model history variables are stored to be
134 used as initial state for the computation of the second ground motion. The
135 storage of the history variable of the i th computation will be used as ini-
136 tial state of the $i+1$ computation. More details regarding the calculation
137 procedures are developed in each section.

138 *2.4. Boundary Conditions*

139 In the analysis, equivalent boundaries have been imposed on the nodes
140 of lateral boundaries (i.e., the normal stress on these boundaries remains
141 constant and the displacements of nodes at the same depth in two opposite
142 lateral boundaries are the same in all directions). They are the response of
143 a modeled infinite halfspace. Hence, only vertically incident shear waves are
144 introduced into the domain. The model is wide enough (194 m) to ensure that
145 the effect of the boundaries on the response of the model can be neglected
146 and also to satisfy the free field condition at the lateral boundaries. For
147 the half-space bedrock's boundary condition, paraxial elements simulating

148 deformable unbounded elastic bedrock have been used [38]. The incident
149 waves, defined at the outcropping bedrock are introduced into the base of
150 the model after deconvolution.

151 **3. Assumptions for this study**

152 For the following study, some assumptions are taken:

- 153 • The effect of external uncontrolled conditions (i.e. weather, rain, wind)
154 is not considered.
- 155 • The material aging is not taken into account since it is connected to
156 its resistance, origin and age.
- 157 • The numerical computations start after the construction phase so when
158 the embankment is in its original state. This means that the embank-
159 ment does not have a history of earthquake loading.
- 160 • The embankment is not subjected to any repairs during its lifetime.
- 161 • The constitutive model does not take into account the secondary con-
162 solidation or compression after each seismic loading.
- 163 • The cluster earthquakes consist of mainshocks and aftershocks only.
164 The effect of foreshocks is not considered in this study.
- 165 • The stochastic ground motion model to generate the mainshocks and
166 aftershock databases is based on the model proposed by Rezaeian and
167 Der Kiureghian [44]. For the sake of simplicity, the same database was
168 used to generate both types of motions.

169 • Only shear body waves have been used in this work. Nevertheless, any
170 other hypothesis about the complexity of the incident wave field can be
171 applied in the scope of this proposed methodology (e.g. [68, 62, 1, 22,
172 among others]).

173 4. Model to generate Aftershocks

174 It is known that an earthquake event does not consist of only mainshocks,
175 but also the occurrence of aftershocks and foreshocks. A cluster earthquake
176 is in general composed of one mainshock with its corresponding aftershocks.
177 The Epidemic Type Aftershock Sequence (ETAS) model has been widely
178 used to model the statistics of seismicity [63, 16, 61, among others]. An
179 essential feature of the ETAS model is the magnitude dependent branch-
180 ing (parent-daughter) ratio. In this context, Turcotte et al. [63] introduced
181 Branching Aftershock Sequence (BASS) model as the self-similar limit of
182 the ETAS model. Both approaches are based on empirical laws describing
183 the distribution of earthquakes in magnitude, time and space. Four scaling
184 relations are required in order to generate and identify the aftershocks dis-
185 tribution: 1) the Gutenberg-Richter (GR) frequency-magnitude scaling, 2)
186 the Bath's law (or the modified version) for maximum-magnitude scaling,
187 3) Omori's law for power-law seismicity rate decay and 4) a spatial form of
188 Omori's law. Both approaches utilize the concept of primary, second-order
189 and higher-order aftershocks. The primary difference between the ETAS and
190 the BASS model is in their use of the Bath's law to estimate the a -value of
191 the GR relation; the BASS model uses the modified form of Bath's law in-
192 stead of the productivity relation used in the ETAS model [63]. In addition,

193 the branching statistics in the BASS model are identical to the self-similar
194 Tokunaga statistics of drainage networks [63, 70].

195 The following section will develop the theoretical model used to gener-
196 ate the corresponding aftershocks of the tested mainshocks in the work of
197 Khalil and Lopez-Caballero [26]. It should be recalled that the used PSHA
198 (which is based on the disaggregation of hazards) is extracted from the work
199 of Aristizábal et al. [2] and the mainshocks were generated from the stochas-
200 tic ground motion model of Rezaeian and Der Kiureghian [44]. As for the
201 aftershocks generation (M_w , R), it is based on the BASS model to compute
202 the distribution in magnitude and time of the aftershocks and is inspired by
203 Turcotte et al. [63] and Hu et al. [16]. As for the location of the aftershocks,
204 and due to the point source of the seismic propagation (i.e. no directivity
205 effects), an Aftershock PSHA (or APSHA) is used [20]. It should be men-
206 tioned that only primary order of aftershocks is considered in this study.
207 The theoretical equations will be developed first, and at the end of that, a
208 summary of the used scaling relations for this study and their parameters
209 will be presented.

210 4.1. *Distribution of the magnitude-frequency*

211 Similar to any seismic hazard model, the frequency-magnitude distribu-
212 tion of each sequence aftershock should satisfy the Gutenberg-Richter (GR)
213 relation (Equation 4):

$$\log_{10}[N_d(\geq m_d)] = a_d - b_d m_d, \quad (4)$$

214 where m_d is the magnitude of the aftershocks, $N_d(\geq m_d)$ is the number of
215 the aftershocks with magnitude greater than or equal to m_d . Parameters a_d

216 and b_d are constants (or known as *a-value*, *b-value* of the GR distribution).
 217 It should be noted that in practice, the magnitude of the largest aftershock
 218 Δm^* should be less than that of the triggering mainshock m_p . So

$$N_d(\geq (m_p - \Delta m^*)) = 1. \quad (5)$$

219 It must be emphasized that this Δm^* is not the magnitude difference between
 220 the mainshock and the largest aftershock [63, 14, 70, 16, among others]. Now
 221 considering both Equations 5 and 4, The GR relation for aftershocks will be:

$$\log_{10}[N_d(\geq m_d)] = b_d(m_p - \Delta m^* - m_d), \quad (6)$$

222 In order to terminate the sequence of aftershocks, it is necessary to spec-
 223 ify a minimum magnitude of the aftershocks m_{min} . The total number of
 224 aftershocks N_{dT} based on Equation 6 is:

$$N_{dT} = N(\geq m_{min}) = 10^{b_d(m_p - \Delta m^* - m_{min})}. \quad (7)$$

225 Hence, the cumulative distribution function P_{Cm} for the magnitude of the
 226 aftershocks, is deduced from Equations 6 and 7. It is a random value between
 227 0 and 1 and is given by:

$$P_{Cm} = \frac{N_d(\geq m_d)}{N_{dT}} = 10^{-b_d(m_d - m_{min})}. \quad (8)$$

228 In this study, the empirical parameters to calculate the aftershocks magnitude
 229 are $m_{min} = 4.5$, $b_d = 1$ and $\Delta m^* = 1.3$.

230 4.2. Distribution of the occurrence time

231 Knowing the magnitude of each occurring aftershock in the sequence from
 232 Equation 8, the time delay t_d until the occurrence of each aftershock after the

233 triggering mainshock should be known. It should be mentioned that t_d will
 234 allow to know the sequential position of each occurring aftershock. Following
 235 the generalized form of Omori's law, the rate of aftershock occurrence $R(t_d)$
 236 would be:

$$R(t_d) = \frac{dN_d}{dt} = \frac{1}{\tau(1 + \frac{t_d}{c})^p}, \quad (9)$$

237 where τ , c and p are given parameters. $N_d(\geq t_d)$ is the number of aftershocks
 238 that occurred after a time t_d :

$$N_d(\geq t_d) = \int_{t_d}^{\infty} \frac{dN_d}{dt} dt' = \frac{c}{\tau(p-1)(1 + \frac{t_d}{c})^{p-1}}. \quad (10)$$

239 Setting $t_d = 0$ in Equation 10, the total number of aftershocks will be ob-
 240 tained. Hence, the cumulative distribution function P_{Ct} for the occurrence
 241 time of the aftershocks will be:

$$P_{CT} = \frac{N_d(\geq t_d)}{N_{dT}} = \frac{1}{(1 + \frac{t_d}{c})^{p-1}}, \quad (11)$$

242 P_{Ct} is a random value between 0 and 1 and the time of occurrence of the
 243 aftershock is deduced from Equation 11. The parameters used in this paper
 244 to calculate t_d are $\tau = 0.001$ and $p = 1.25$.

245 4.3. Location of the aftershocks

246 Concerning the seismogenic zone of the aftershock, it is assumed that each
 247 mainshock has its corresponding aftershocks located within an area around
 248 its epicenter [64, 20]. The size of this area depends on the magnitude of the
 249 triggering aftershocks as follows:

$$S_A = 10^{m_d - m_{min}}. \quad (12)$$

250 Within this area, considered a square, the aftershocks occur on a smaller
 251 lattice. Table 1 summarizes the parameters values of the scaling relations
 252 used for this study. Having the magnitude, distance and the time delay until
 253 the occurrence of every daughter earthquake, the generation of aftershocks
 254 time histories is feasible.

Scaling relations	Parameters values	References
<i>GR-Bath Parameters:</i>		
$N_{dT} = N(\geq m_{min}) = 10^{b_d(m_p - \Delta m^* - m_{min})}$	$b_d = 1$	Turcotte et al. [63]
$m_d = -(1/b_d) \cdot \log(P_{Cm}) + m_{min}$	$\Delta m^* = 1.3$	and Hu et al. [16]
<i>Temporal Omori:</i>		
$t_d = \tau \cdot (P_{Ct}^{-1/(p-1)} - 1)$	$p = 1.25$ $\tau = 0.001$	Turcotte et al. [63] and Hu et al. [16]
<i>Space APSHA:</i>		
$S_A = 10^{m_d - m_{min}}$	$m_{min} = 4.5$	Iervolino et al. [20]

Table 1: The scaling relations of the magnitude, time and space distribution of the used model to generate the aftershocks

255 4.4. Description of the generated aftershocks

256 In the following, it will be described the intensity measure (IM) of the
 257 generated aftershocks. The median response spectra (structural damping ξ
 258 = 5%) of the generated mainshocks and aftershocks are shown in Figure
 259 2a. It is clear that the spectral acceleration of the mainshocks is slightly
 260 higher than that of the aftershocks [63, 16]. Figure 2b shows a window of
 261 a sequence example. From this figure, three ideas can be validated: i) the
 262 aftershocks, if they exist, occur between two mainshocks, ii) the magnitude

263 of the aftershocks is less than that of the principal mainshock and iii) the
 264 mainshock of larger magnitude will generate more aftershocks comparing to
 the one with a small magnitude (Equation 8).

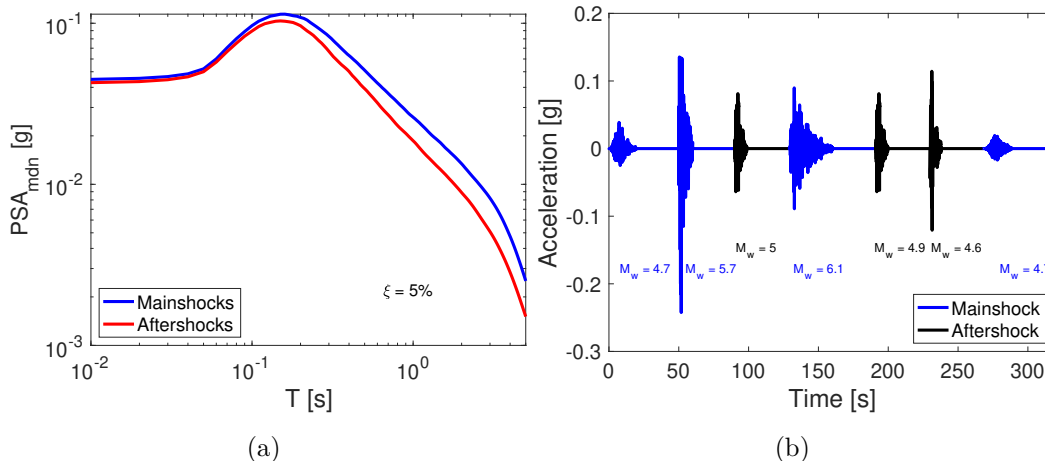


Figure 2: a) The obtained median response spectra of both the generated mainshocks and aftershocks and b) a section of a sequence

265

266 Another illustration of the mainshock-aftershock parameters is shown in
 267 Figure 3. This figure considers three mainshocks (in dashed circles) with their
 268 corresponding aftershocks. Each cluster earthquake is drawn in a different
 269 shape (circle, square and diamond). Figure 3a shows a scatter plot of the
 270 outcrop acceleration $a_{max,out}$, the equivalent predominant frequency $1/T_{va}$
 271 and the magnitude M_w . Based on Kawase [25], there exists a proportional
 272 relation between the peak ground velocity PGV , $a_{max,out}$ and $1/T_{va}$ and is
 273 represented as dashed lines. In addition to these parameters, Figure 3b shows
 274 also the variation of the source-to-site distance R . Thus, it can be seen that
 275 the corresponding aftershocks have close values of $a_{max,out}$ and $1/T_{va}$ but are
 276 in general less than those of the mainshock. In addition, the magnitude of the

277 mainshock is higher than that of the aftershocks, whereas the source-to-site
 278 distance of the aftershocks is close to that of the mainshocks.

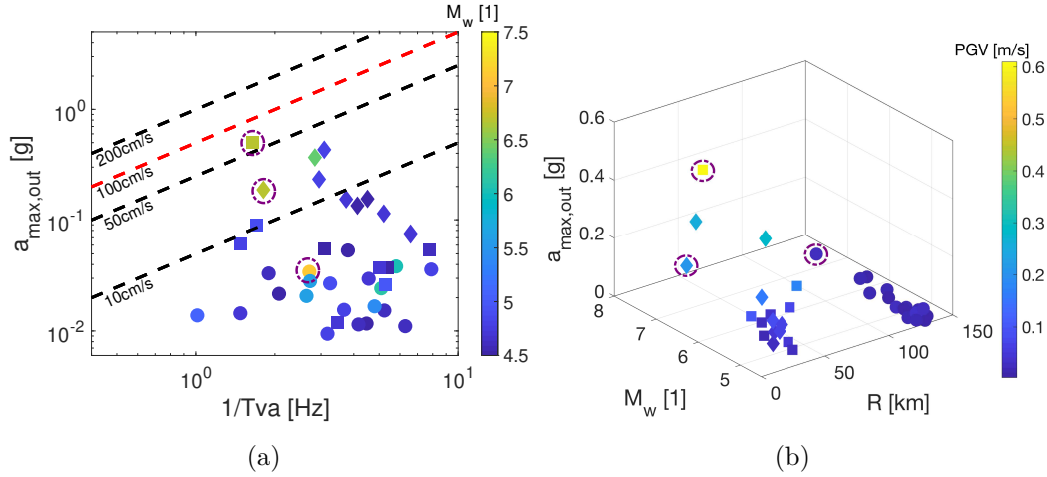


Figure 3: Scatter plots of some ground motion parameters of mainshocks and aftershocks with respect to a) M_w and b) M_w and R

278

279 In order to take into account all the generated Mainshocks (MS) and the
 280 Aftershocks (AFS), Figure 4 shows a matrix form of a logarithmic comparison
 281 of some ground motion parameters. The comparison is conducted in terms of
 282 the density functions located in the diagonal of the matrix, the scatterplots
 283 and the histograms that are both located to the bottom left of the diagonal
 284 and finally, the boxplots that are located to the right.

285 For the density distribution function of the chosen parameters, it can be
 286 seen that the distributions of MS and AFS overlap for almost all the tested
 287 parameters, except a small change in the effective duration (D_{595}). For the
 288 median value of each database, it can be seen from the boxplots, that it is
 289 almost close for the two ground motions. Concerning the scatterplots and
 290 the histograms, they are useful to have an idea about the precise values

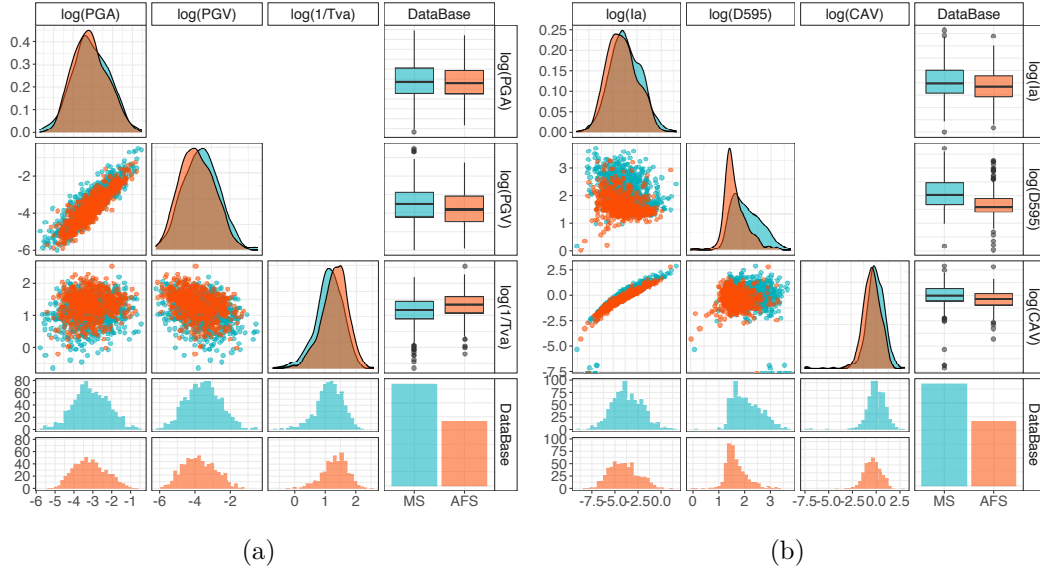


Figure 4: The logarithmic distribution of some ground motion parameters of the Mainshocks (MS) and the Aftershocks (AFS)

291 or the relation between each parameter. After the aftershock generation,
 292 it will be analyzed in the next section the possible damage measures of the
 293 embankment and their link to the soil mechanical behavior. For this purpose,
 294 an example ground motion will be analyzed.

295 5. Potential embankment failure and soil behavior

296 The assessment of seismic structural vulnerability has broadened the ob-
 297 jectives of seismic design so that not only the safety against collapse is the
 298 required criteria but also the long term resistance of the structures. A great
 299 effort has been made to improve the current earthquake-resistant design
 300 methods in order not only to avoid collapse under a destructive earthquake,
 301 but also to limit the damage under moderate earthquakes [8]. In engineering

302 practices, a stability analysis is required in order to define a safety index
303 above which the structure will reach failure. This later is commonly known
304 by the factor of safety FS . It is typically defined as the ratio of the avail-
305 able shear strength to the shear stress required to maintain the equilibrium
306 along the prescribed failure surface [17, 28, 37, among others]. The basic
307 idea for stability analysis, specially for soils, is to calculate at each point in
308 a domain of interest, the so-called local factor of safety LFS . It is defined as
309 the ratio of the potential mean stress to the current mean stress under the
310 Mohr-Coulomb failure criterion [37].

311 Furthermore, the damage indices used to measure the potential structural
312 failure are either defined for each structural element (local) or related to the
313 entire structure (global) [8]. In this study, the potential failure and damage
314 state of the embankment are going to be examined. For the sake of brevity
315 only, one example ground motion will be analyzed in details. The acceleration
316 time history at the bedrock for this example is shown in Figure 5a. Typically,
317 the global structural failure for any similar geo-structure (the embankment)
318 is to calculate the relative crest settlement $u_{z,rel}$. Thus, Figure 5b shows the
319 evolution of $u_{z,rel}$ and the excess pore water pressure ratio r_u (i.e. $= \Delta p_w / p'_0$)
320 for a point placed under the center of the embankment, at 3 m depth. It
321 should be mentioned that 30 seconds of zero values were added at the end
322 of the ground motion in order to ensure the recovery time (i.e. the Δp_w
323 dissipation) of the soil foundation. It is clear from Figure 5b that at the
324 beginning of the motion, $u_{z,rel}$ and r_u were zero. When the strong phase of
325 the motion starts the relative crest settlement decreases rapidly, in parallel,
326 the excess pore water pressure is generated which results in peak values for

327 r_u . After the strong phase, the crest settlement is constant and the excess
 328 pore water pressure dissipates. Thus, from Figure 5, it was shown that there
 329 is a link between the evolution of the vertical displacement with the excess
 330 pore water pressure dissipation.

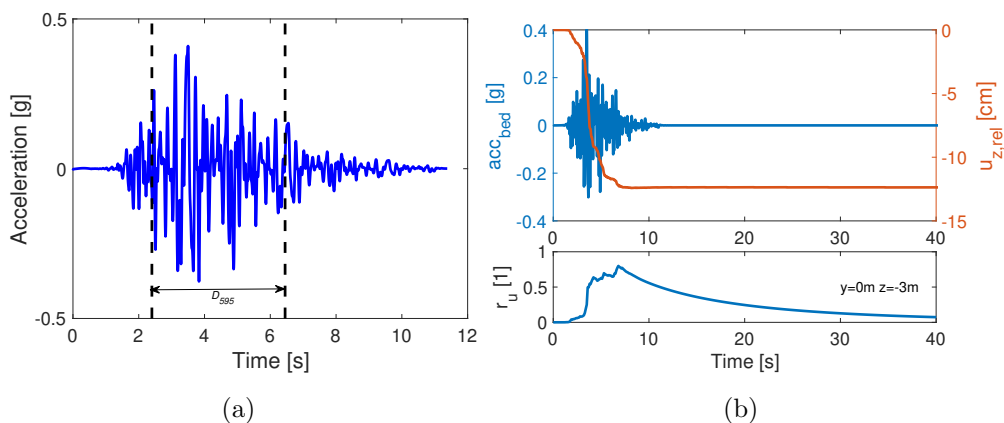


Figure 5: The example a) ground motion of this section and b) the evolution in time the relative crest settlement $u_{z,rel}$ and the excess pore water pressure ratio r_u (i.e. $= \Delta p_w / p'_0$)

331 In order to have a complete idea about the response of the soil foundation-
 332 embankment system, the distribution of the vertical co-seismic displacement
 333 at the end of the earthquake loading was calculated. Since the liquefaction
 334 induces settlement (as shown in Figure 5), the results of Figure 6 show that
 335 the damage of the soil foundation is located under the embankment towards
 336 the free field [28, 27, 49, 42, among others].

337 As for the potential failure of the embankment from a local damage index,
 338 a local safety factor LFS could be estimated by calculating the residual
 339 strength. The LFS discussed in this paper is the one proposed by Lopez-
 340 Caballero and Modaressi-Farahmand-Razavi [35] and Rapti et al. [43] and
 341 derived from the yield surface of the model (Equation 1). The parameter r_{apt}

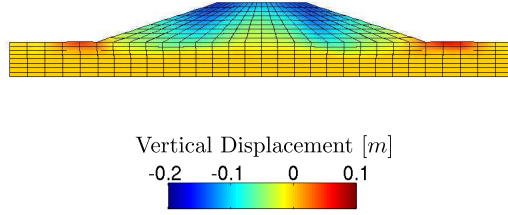


Figure 6: Enlarged view of typical vertical co-seismic displacement contours at the end of the shaking

342 (Equation 2) provides, for any soil state, a direct measure of the “distance
 343 to reach the critical state”.

344 It varies between 0 and 1 where perfect plasticity is reached and could
 345 be defined as the inverse of a local safety factor ($r_{apt} = 1/LFS$). Thus, the
 346 potential mean stress is evaluated from the friction angle at the critical state
 347 ϕ'_{pp} and the current mean stress from the apparent friction angle ϕ'_{apt} . A
 348 threshold value of the damage measure is taken as 0.75 (i.e. corresponds to
 349 $LFS \approx 1.33$). This value was chosen based on sensitivity analysis performed
 350 in previous works on the same geometry [35, 43]. The domain of interest in
 351 which the local damage r_{apt} is calculated is composed of the liquefied layer
 352 with the embankment (i.e. the green window in Figure 1). It was chosen
 353 vertically due to the depth of the liquefiable layer (4 m) and laterally because
 354 of the possible zones where the failure pattern could occur.

355 In order to explain the function of this local damage measure, Figure 7
 356 considers the threshold value of r_{apt} (i.e. 0.75). Thus, in this figure, for each
 357 integration point (IP) in the domain of interest, when $r_{apt} < 0.75$, red color
 358 is designated and when $r_{apt} \geq 0.75$, black color is attributed. It is clear that
 359 at the beginning of the ground motion, some instability points (black points)

360 are shown in the foundation layer. However, at 7 seconds, which is the end
 361 of the strong phase, additional instability points appear in the foundation
 362 and increase upwards towards the slope of the embankment (Figure 7b).

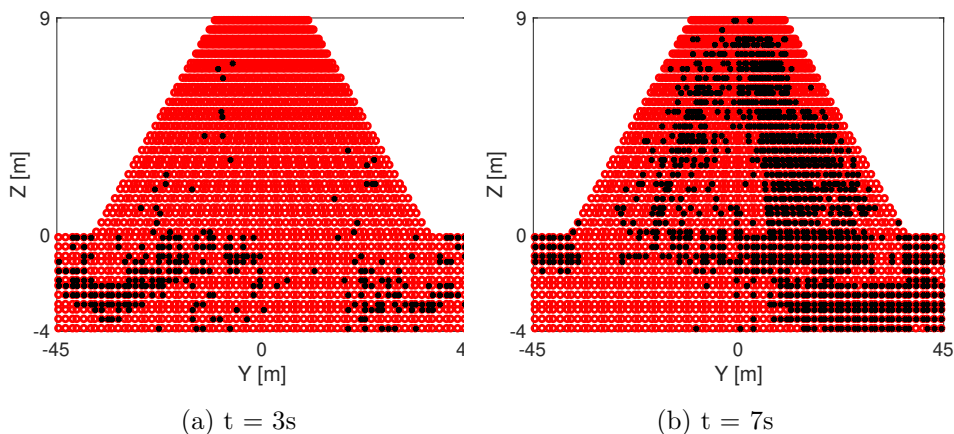


Figure 7: The distribution of $r_{apt} \geq 0.75$ from the IPs (black points) for the example ground motion

363 In order to statistically represent the values of r_{apt} during the co-seismic
 364 duration of the example ground motion, Figure 8a shows the empirical Com-
 365plementary Cumulative Distribution Function (CCDF) of this parameter.
 366 The limit value of 0.75 is also plotted as a dashed line in this figure. As
 367 expected, the CCDF of r_{apt} evolves with the increase in time. It may also be
 368 interpreted that the probability of exceeding the limit value of 0.75 increases
 369 with time. Because r_{apt} can be interpreted as a *LFS*, its global representa-
 370tion can be defined as the probability of exceeding the r_{apt} limit value, which
 371 is going to be introduced in this paper as the *failure density concept*. This
 372 later intends to give a measure of the failure extension in the embankment,

373 and is defined in Equation 13 such as:

$$\mathbb{P}(r_{apt}(t) \geq 0.75) = \int_{r_{apt} \geq 0.75} f(r_{apt}) \cdot dr_{apt} \approx \frac{N_{r_{apt} \geq 0.75}(t)}{N_T} = \rho_{r_{apt} \geq 0.75}(t) \quad (13)$$

374 $N_{r_{apt} \geq 0.75}(t)$ gives the number of IPs that surpassed the r_{apt} limit and N_T is
 375 the total number of IPs in the domain of interest (2484 in this case).

376 Therefore, having two global damage indices: $u_{z,rel}$ and $\rho_{r_{apt}}$, it will be
 377 interesting to compare them to understand the potential failure of the em-
 378 bankment. For this purpose, Figure 8b shows the variation in time of both
 379 $u_{z,rel}$ and $\rho_{r_{apt} \geq 0.75}$ and points out on the fact that they are linked. For ex-
 380 ample, around 5 seconds, the settlement increases when $\rho_{r_{apt} \geq 0.75}$ increases.
 381 Once the settlement reaches a maximum value, $\rho_{r_{apt} \geq 0.75}$ reaches a peak value
 382 of 0.4. This value means that 40% of the domain of interest had $r_{apt} \geq 0.75$.
 383 It is interesting to mention that around 10 seconds, even if $u_{z,rel}$ was constant,
 384 $\rho_{r_{apt} \geq 0.75}$ was decreasing. This can be interpreted as the dissipation of the
 385 excess pore water pressure at this time (Figure 5b) and a proof that the soil
 386 tries to densify. At the end of the strong phase of the motion, the soil tries
 387 to recover where its settlement and r_{apt} are constants. Thus, for practical
 388 purposes, $\rho_{r_{apt} \geq 0.75}$ at the end of the ground motion is able to represent the
 389 state of failure of the embankment.

390 It can be partially concluded in this section that the damage measure
 391 represented by the relative crest settlement $\delta u_{z,rel}/H$ is a global indicator of
 392 the potential failure of the embankment. In addition, the ratio of apparent
 393 to critical friction angle, designated as the parameter r_{apt} , provides a reliable
 394 measure of soil strength, which takes into account the loading history and
 395 can be used as a criterion for estimating the local state of soil during cyclic
 396 loads. More importantly, this section interpreted the link between the global

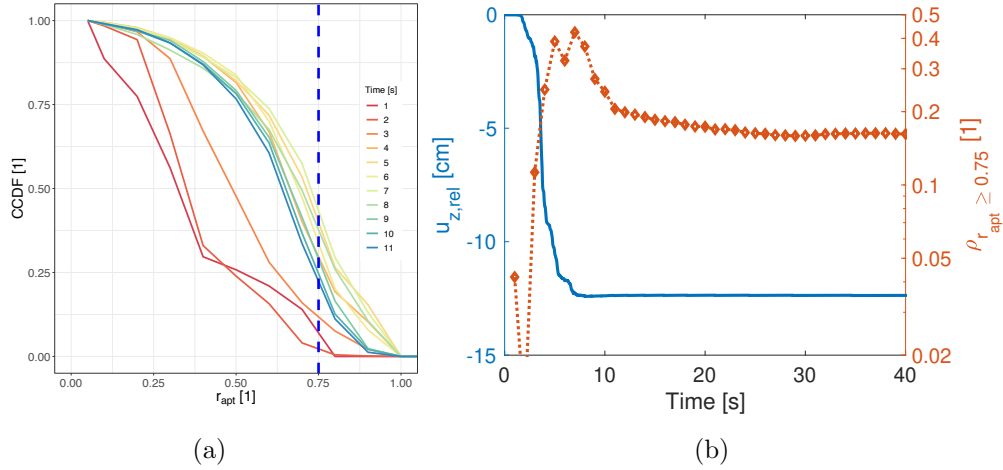


Figure 8: a) The CCDF of the parameter r_{apt} during the co-seismic duration of the example ground motion and b) the dependency of the parameter r_{apt} with the crest settlement of the embankment

397 damage measure with the soil mechanical behavior via the predefined *failure*
 398 *density concept*.

399 Hence, it was presented in this paper a model to generate aftershocks and
 400 the analysis to interpret the potential embankment failure. In the coming
 401 section, one of the objectives of this paper will be developed, as to quantify
 402 the potential failure for sequences of mainshocks and aftershocks.

403 6. Damage measures during sequential clusters

404 The PBEE methodology provides a probabilistic description of the system-
 405 level performance of structures. It requires the calculation of an engineering
 406 demand parameter that can be related to the intensity measure of the oc-
 407 curred ground motions. Therefore, it was presented in Section 5, the potential
 408 structural failure and its link to the soil mechanical behavior. However, for

409 life cycle assessment, it is necessary to account for the build-up of seismic
410 losses due to the damage issued from multiple events [11, 20, 19, among
411 others].

412 In this study, 210 sequential cluster events of mainshocks-aftershocks are
413 considered. It should be reminded that each sequence contains 44 acceler-
414 ation time histories of mainshocks that were permuted 10 times. In order
415 to be statistically representative of a large number of sequences, 21 subsets
416 compatible with the seismic hazard of the site of concern were considered. In
417 addition, as to take into account the recovery time of the embankment, 30
418 seconds are interposed between each mainshock and its corresponding first
419 aftershock. The choice of this recovery duration was validated in the work
420 of Khalil and Lopez-Caballero [26]. Since the used numerical model takes
421 into consideration the damage history of the embankment, in this section,
422 the evolution of the global response will be examined from the two damages
423 indices that were presented in Section 5: $u_{z,rel}$ and $\rho_{r_{apt}}$. All over this paper,
424 the notation *MS* designates *Mainshocks* and *AFS* designates *Aftershocks*.

425 6.1. Relative crest settlement for sequential earthquakes

426 The performance of the embankment during its lifetime is evaluated from
427 its cumulative global damage measure. In the case of embankments (or
428 dams), the damage is quantified from the percentage relative crest settle-
429 ment $\delta u_{z,rel}/H$ where $u_{z,rel}$ is the crest settlement and H is the height of
430 the embankment with the foundation (i.e. 19 m in this case) [60, 33, 26].
431 In order to classify the damage, levels are also attributed to the relative
432 crest settlement. When $\delta u_{z,rel}/H \leq 0.02\%$, there is *No* damage, if 0.02%
433 $< \delta u_{z,rel}/H \leq 0.1\%$, the damage is *Minor*, if $0.1\% < \delta u_{z,rel}/H \leq 1\%$, the

434 damage is *Moderate* and finally if $\delta u_{z,rel}/H > 1\%$, the damage is *Serious*.
 435 For the sake of brevity only, Figure 9 shows a comparison of $\delta u_{z,rel,cum}/H$ for
 436 two examples of MS and MS-AFS sequences in order to examine the effect
 437 of the aftershock occurrence. It should be mentioned that for each presented
 438 case in Figure 9, the permutation of the mainshock events is the same in
 both sequence types. The only difference is the aftershock occurrence.

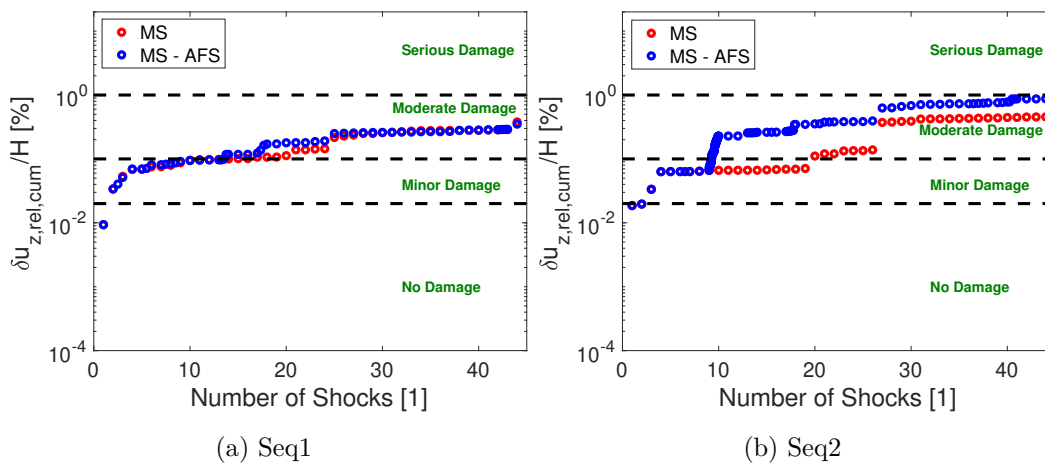


Figure 9: Comparison of the percentage relative crest settlement for examples of mainshock (MS) sequences and mainshocks - aftershocks (MS - AFS) sequences

439

440 Clearly, two typical behaviors can be identified. In Figure 9a, the global
 441 response during the lifetime of the embankment was almost similar for the
 442 two sequences. At a point in time, when the 17th shock occurred, a small
 443 peak in the damage appears for the MS-AFS sequences. It should be noted
 444 that this shock has 3 aftershocks. On the contrary, the damage for MS-
 445 AFS sequences in Figure 9b, has a rapid increase in the 9th shock so that
 446 it surpassed a damage level. It should be noted that this shock produced
 447 70 aftershocks. After it, the damage was higher for the MS-AFS sequences,

448 until the end of the lifetime when the soil densifies and was subjected to
 449 multiple types of motions, the level of damage is the same. Thus, it should
 450 be remarked that the existence and the high number of aftershocks have a
 451 major importance on the lifetime behavior of the embankment due to the
 452 aging effect.

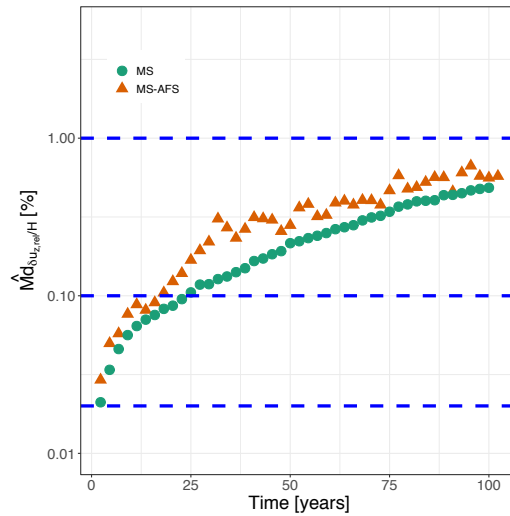


Figure 10: Median value of the crest settlement for the MS and MS-AFS sequences

453 In order to take into consideration all the tested sequences, the median
 454 estimator value of the cumulative relative crest settlement is shown in Figure
 455 10. A comparison with the MS sequential type appears also in this figure.
 456 Clearly, the damage during the lifetime of the embankment is not the same,
 457 whereas at the beginning and the end of the lifetime, the cumulative crest
 458 settlement is slightly different between both sequences. However, for the level
 459 of damage, no remarkable difference is noted (i.e. at the end of the lifetime,
 460 *Moderate* damage was reached for MS and MS-AFS sequences).

461 It can be deduced from the comparison of the MS and MS-AFS sequences,

462 that the lifetime damage of the embankment could be affected by the occur-
463 rence and number of aftershocks. As for the level of damage, similarity
464 between both sequences is identified. In order to understand the reason be-
465 hind this behavior, a close examination of the damage index related to the
466 soil residual strength is conducted in the next section.

467 6.2. Damage index from the soil residual strength

468 It was presented in Section 5 the parameter r_{apt} that provides, for any
469 soil state, a direct measure of the “distance to reach the critical state”. It
470 varies from 0 and reaches perfect plasticity at 1. It could be defined as the
471 inverse of a local safety factor ($r_{apt} = 1/LFS$). A threshold value of the
472 damage measure is taken as 0.75 (i.e. corresponds to $LFS \approx 1.33$) [35, 43].
473 In addition, in the same cited section, the failure density concept represented
474 by $\rho_{r_{apt} \geq 0.75}$ at the end of the ground motion, was presented. It was shown its
475 ability to represent the state of failure of the embankment. In this section,
476 similarly to Section 5, the link between the two global damage measures is
477 shown in Figure 11 for an example of MS and for the same sequence with
478 added aftershock occurrence. It should be mentioned that for the sake of
479 clarity, the notation $\rho_{r_{apt} \geq 0.75}$ means $\rho_{r_{apt} \geq 0.75}(t = t_{end})$. The response of
480 the embankment for this sequence example can be divided into three time
481 intervals: before 30 years, between 30 and 60 years, and after 60 years. A
482 discussion on each sequences will be developed and then a comparison of
483 both types will proceed.

484 For the mainshock sequence in Figure 11a, before 30 years, $\rho_{r_{apt} \geq 0.75}$
485 slightly increases when $\delta u_{z,rel}/H$ was also increasing. The global damage
486 for this duration is still acceptable. It should be remarked that for one shock

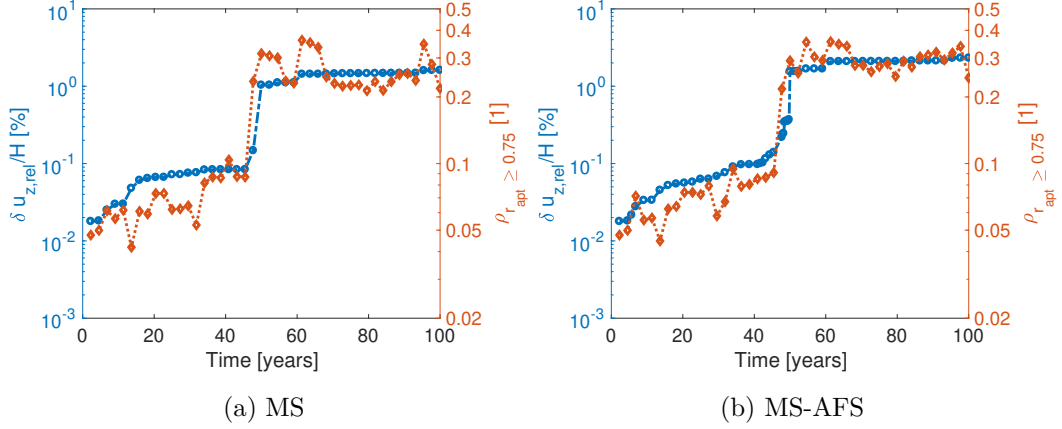


Figure 11: The time evolution of $\delta u_{z,rel}/H$ and $\rho_{r_{apt} \geq 0.75}$ for a a) MS sequence and b) a MS-AFS sequence

487 at 18 years, the crest settlement increases whereas $\rho_{r_{apt} \geq 0.75}$ decreases, this
 488 means that this shock did not induce many local instabilities, or on the
 489 contrary it helped the soil to dissipate the residual excess pore water pres-
 490 sure Δp_w . Between 30 and 60 years, the behavior was almost similar to the
 491 previous time interval, however, at 50 years, $\delta u_{z,rel}/H$ increases rapidly to
 492 reach severe damages which induced instabilities in 30% of the domain of
 493 interest (i.e. $\rho_{r_{apt} \geq 0.75} = 0.3$). After 60 years, both damage measures are
 494 constants. For the aftershock occurrence in this example sequence (Figure
 495 11b), no change in the response for two time intervals: before 20 years and
 496 after 60 years. However, between 30 and 60 years, the aftershocks prevented
 497 the rapid increase in the crest settlement and in $\rho_{r_{apt} \geq 0.75}$. It is interesting
 498 to mention that for both MS and MS-AFS sequences, at the end of the life-
 499 time duration, the level of damage is the same. This result confirms the one
 500 deduced from the lifetime evolution of the relative crest settlement and also

501 verifies the link between both damage indices.

502 As to take into consideration more example sequences and the lifetime
 503 evolution of $\rho_{r_{apt} \geq 0.75}$, Figure 12 shows this parameter for two example se-
 504 quences of MS and their corresponding sequences with aftershock occurrence.
 505 As expected, two typical behaviors can be identified: a similarity in the re-
 506 sponse during time, for example the sequence in Figure 12a, and a slight
 507 difference in the damage response specially after a strong shock (Figure 12b).

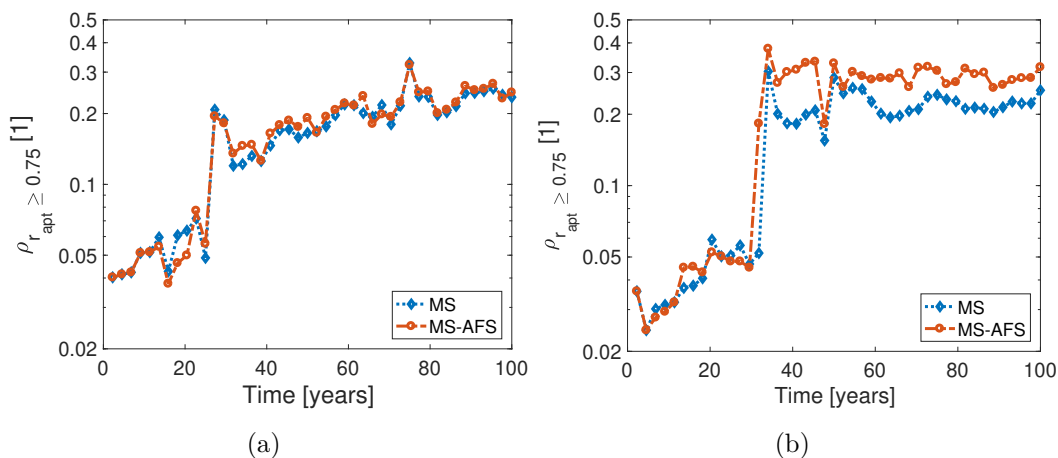


Figure 12: The time evolution of $\rho_{r_{apt} \geq 0.75}$ for two examples of MS or MS-AFS sequences

508

509 It can be partially concluded from this section, that the lifetime embank-
 510 ment failure, evaluated from two damage indices slightly differs between MS
 511 and MS-AFS sequences. The occurrence of AFS increases the damage during
 512 time, but, at the end of the working life, the damage of both sequences is
 513 slightly close and the level of damage is the same. The coming section will
 514 try to answer the second question of the Introduction section about the effect
 515 of the aftershocks occurrence on the lifetime distribution of the embankment.

516 7. Survival Functions of the tested sequences

517 Survival analysis is the time needed for an event of interest to occur. It is
518 also called the time-to-event data [5, 24, 53, among others]. Survival analysis
519 is represented by survival functions or hazard functions. They are both
520 inversely proportional, so when the hazard increases the survival function
521 decreases. This later estimates the lifetime distribution of a test model and
522 more importantly its Mean Time To Failure (MTTF, the expected time to
523 failure for a non-repairable system). Several approaches exist in order to
524 calculate the survivors [15]. The approaches can be parametric and non-
525 parametric. The commonly used method is the *Kaplan-Meier* estimator [23].
526 It is a non parametric method that does not need any assumption for the
527 distribution of the survival time, or the relationship between the covariates
528 and the survival time (please refer to Khalil and Lopez-Caballero [26] for
529 more details).

530 Section 6 analyzed the damage measure of example sequences of MS and
531 MS-AFS. In order to take into account all the tested sequences and evaluate
532 their survival probability, a survival analysis is conducted to estimate the
533 useful working life of the embankment. The survival functions for *Moderate*
534 (DL3) and *Serious* (DL4) damage levels are shown in Figure 13. The MTTF
535 and the *p-value* are indicated in this figure. Typically, if the *p-value* $\leq 5\%$,
536 the null hypothesis H_0 is rejected, and vice versa. The null hypothesis H_0
537 in the case of this study is to state that there is no difference in the lifetime
538 distribution of the embankment between a MS or a MS-AFS sequence. In
539 addition, an informative risk table shows the number of sequences that did
540 not reach the precised damage level in a specific period of time.

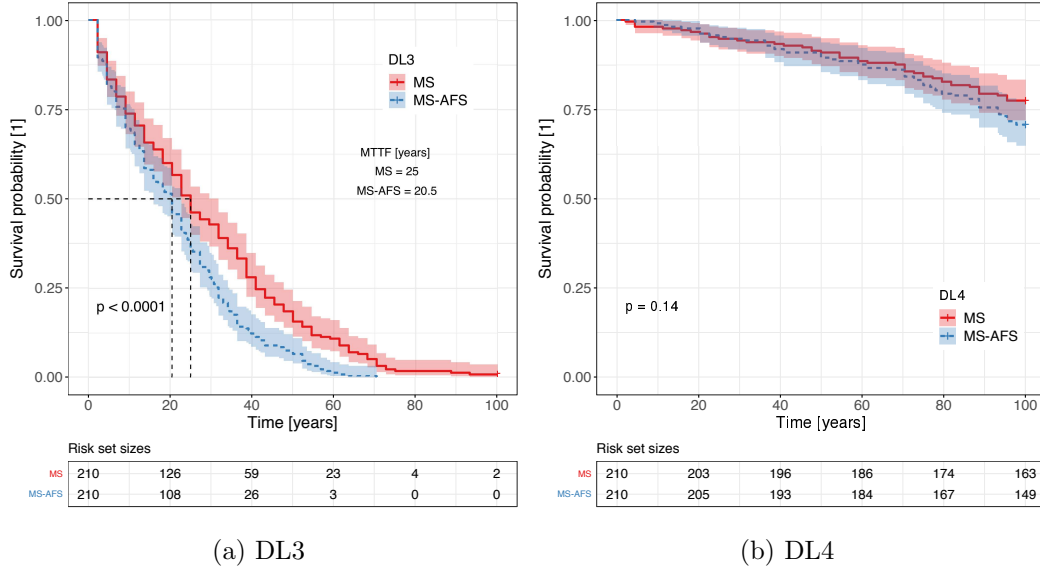


Figure 13: Survival functions of the two types of sequences used in this study for a) *Moderate* damage (DL3) and b) *Serious* damage (DL4)

541 Concerning the *Moderate* damage (or DL3, Figure 13a), the MTTF of
 542 both MS and MS-AFS sequences occurs within the useful life of the embankment (i.e. 100 years). However, the occurrence of aftershocks decreased the
 543 MTTF of the embankment for 5 years (i.e. 20.5 years instead of 25 years
 544 in the case of MS sequence). The risk table shows also that more sequences
 545 have reached *DL3* when the aftershocks occurred. For example, at 20 years,
 546 126 MS sequences resisted this level of damage comparing to a decrease to
 547 108 MS-AFS sequences that resisted it. Moreover, the p -value $\leq 5\%$, which
 548 means that H_0 is rejected and thus, there is a difference between the life-
 549 time distribution of the MS and MS-AFS sequences for *Moderate* damage
 550 level. As for the *Serious* damage (or DL4, Figure 13b), the embankment
 551 maintains its initial performance and then starts to degrade. The survival
 552

553 function did not reach its MTTF and the null hypothesis is accepted, thus,
554 the two distributions are alike for this damage level. However, after 70 years,
555 the lifetime distribution for DL_4 in the case of MS-AFS sequences is slightly
556 different from that of the MS sequences. This remark can also be seen from
557 the risk table. Thus, for DL_4 , the embankment might reach its MTTF for
558 a time window greater than 100 years (e.g. 475 years). This means that for
559 the same event rate of mainshocks, the number of shocks should be greater
560 than 44 input motions in one sequence. Clearly, the survival probability is
561 site and structure specific because it is controlled by the ground motions
562 characteristics and the working life of the concerned structure.

563 It can be partially concluded that for the considered working life of the
564 embankment, the aftershocks occurrence is important for small damage lev-
565 els whereas it is not the case for high damage levels. This aspect was seen
566 because the embankment for DL_3 , started to degrade directly after the first
567 shock, however for DL_4 , it maintained its initial performance and then de-
568 graded with time.

569 Since the survival analysis can be interpreted as a probability of failure,
570 the coming section will develop the classical way to calculate this proba-
571 bility (i.e. fragility curves) and evaluate it during the working time of the
572 embankment.

573 **8. Fragility Curves Evolution**

574 From Section 7, it was found that for *Moderate* damage level, the life-
575 time distribution of the MS and MS-AFS sequences is not the same, on the
576 opposite to *Serious* damage level. In this section, and because the fragility

577 curves are commonly used for seismic loss estimation and risk management,
 578 their time evolution will be assessed. Classical fragility curves represent the
 579 failure probability of one event of given intensity [56, 50, 71, 19, among oth-
 580 ers]. However, for life cycle assessment, it is necessary to account for the
 581 build-up of seismic losses because of the damage in multiple events. In addi-
 582 tion, the fragility functions are developed from *independent and identically*
 583 *distributed* observations. Hence, it is difficult to conduct the fragility curves
 584 for the cluster earthquakes since the aftershocks depend on their correspond-
 585 ing mainshock. More importantly, the history of loading in the soil plays a
 586 major role in its future behavior, which emphasizes the importance of the
 587 sequential analysis [26]. In order to overcome these challenges, the evolution
 588 of the fragility curves during the lifetime of the embankment proposed in this
 589 study is summarized in Figure 14.

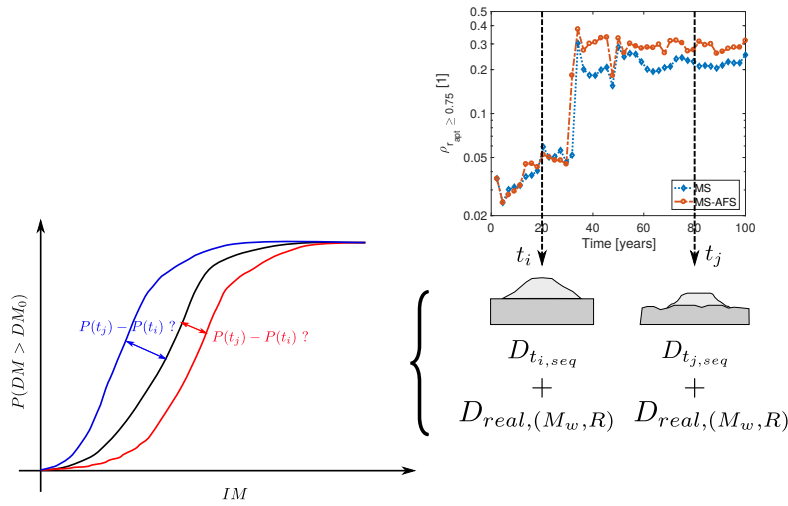


Figure 14: The methodology presented in this section

590 For different time intervals during a MS-AFS sequence, the damage state

591 of the embankment is not the same. For each corresponding state that is
592 considered as a new initial state (site and structure specific), the damage
593 is calculated after a number of unchained real motions (structure-specific)
594 in order to calculate the fragility curves. Thus, the total induced damage
595 is the one produced from the cumulative damage at $t = t_i$ resulted from
596 the MS-AFS sequence added to the one produced from unchained ground
597 motions (i.e. $D_{tot} = D_{cum_{ms-afs}(t_i)} + D_{real}$). According to that, and given
598 the IM corresponding to the real ground motions, the evolution in time of
599 the fragility curves is calculated. In the case of this work, the real ground
600 motions are events ranged between 5.2 and 7.6 in magnitude and a site-
601 to-source distances from 15 to 50 km. The MS-AFS sequence taken as an
602 example is shown in Figure 15 and the years interval after which the fragility
curves will be calculated are 0, 11, 22, 45 and 100 years (dashed lines).

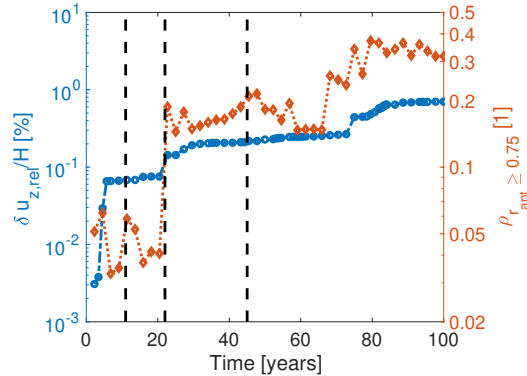


Figure 15: The MS-AFS sequence example and the time interval after which the fragility curves will be calculated

603

604 First, in order to examine the evolution of the global response of the
605 embankment after each time interval, its damage measure is drawn in Figure
606 16a. It should be noted that in this figure, “NH” means “no history” and

607 “WH” means “with history”. Taking as a reference the damage at $t_0 =$
608 0, which means when the embankment did not have a loading history, the
609 evolution of $\delta u_{z,rel}/H$ shows that the crest settlement increases when the
610 time increases. It is interesting to mention that for longer t_0 , the history
611 of loading delays the arrival to high values of $\delta u_{z,rel}/H$ that are compatible
612 with the referenced values (at $t_0 = 0$).

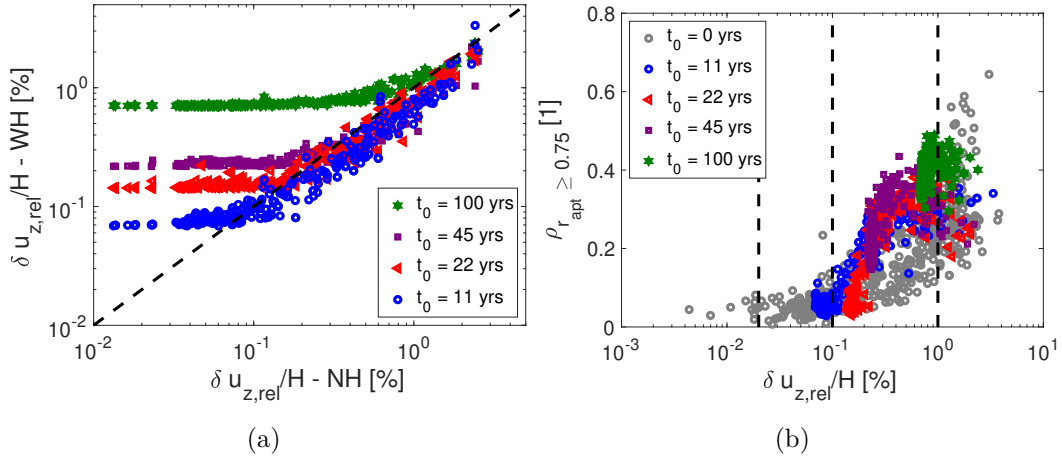


Figure 16: The time evolution of the crest settlement $\delta u_{z,rel}/H$ and the density parameter $\rho_{r_{apt}}$

613 Concerning the second damage index related to the soil’s residual strength,
614 $\rho_{r_{apt} \geq 0.75}$, and its link with $\delta u_{z,rel}/H$, Figure 16b shows that for a short life-
615 time duration (i.e. $t_0 = 11$ or 22 years), $\delta u_{z,rel}/H$ and $\rho_{r_{apt} \geq 0.75}$ were still
616 acceptable. In addition, the results overlap with the ones of $t_0 = 0$. This indi-
617 cates that sometimes, the short term analysis may be enough to represent
618 the performance of the embankment. However, for higher values of t_0 , both
619 $\delta u_{z,rel}/H$ and the density parameter $\rho_{r_{apt} \geq 0.75}$ showed important values with
620 a tendency to regain the initial values at $t_0 = 0$. This result emphasizes the

621 need to consider sequential analysis to understand the global performance of
 622 the structure during its lifetime.

623 As for the calculation of the fragility curves, two-parameter log-normal
 624 distribution functions are used (Equation 14) and are estimated from the
 625 maximum likelihood method [56, 50]. In addition, the quality of fit and the
 626 estimation of the confidence intervals are conducted from the methodology
 627 presented in the work of Sáez et al. [50]. Considering a threshold damage
 628 measure DM_0 , an IM of level im and the “initial” damage $D_{t_0,seq}$ issued from
 629 the MS-AFS sequence, the fragility curve has the following form:

$$P(DM > DM_0 | IM = im_{(M_w,R)}, D_{t_0,seq}) = \phi \left[\frac{\ln im - \ln \eta_{DM|IM}}{\beta_{DM|IM}} \right] \quad (14)$$

630 where $\phi[.]$ is the standardized normal distribution function. $\eta_{DM|IM}$ is the
 631 median threshold value of IM required to cause the damage and $\beta_{DM|IM}$ is
 632 the total lognormal standard deviation. One or more IMs can be used to
 633 represent the fragility curves [33]. In this study, the intensity measure is
 634 the outcrop acceleration $a_{max,out}$, and the damage levels are *Moderate* and
 635 *Serious*. The results are shown in Figure 17. The solid line in this figure
 636 shows the response of the embankment without a past history which means
 637 at $t_0 = 0$.

638 Clearly, from Figure 17, the fragility curves evolves with time. At 11
 639 years, the probability of exceeding the *Moderate* damage level depends on
 640 the severity of the ground motion that happened during this time (Figure
 641 17a). For example, for ground motions of accelerations less than 0.2 g, the
 642 fragility curve is lower than that at $t_0 = 0$. However, greater than 0.2 g and
 643 for this short time interval, the embankment reaches failure faster when it
 644 has a history of loading. It should be mentioned that the other time intervals

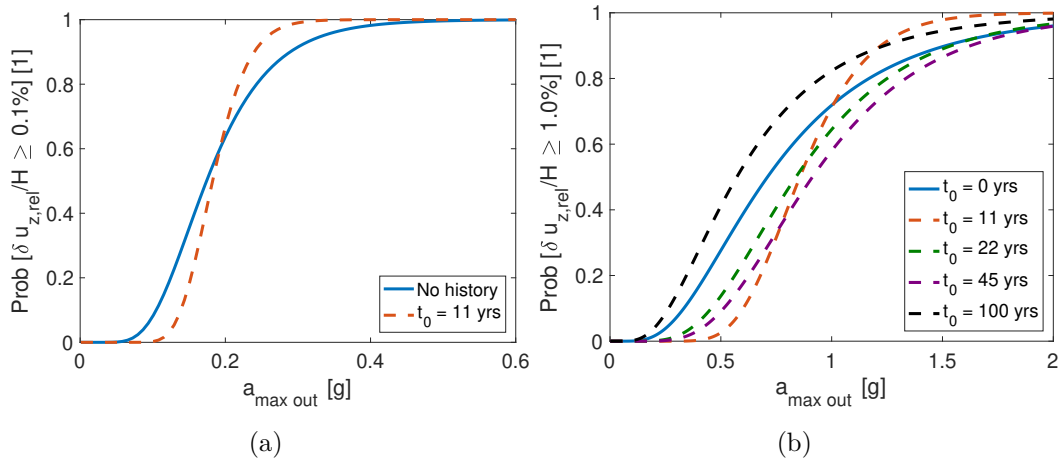


Figure 17: The evolution in time of the probability of exceeding the a) *Moderate* and b) the *Serious* damage level

645 (i.e. 22, 45 and 100 years) are not drawn for this damage level, since their
 646 probability of failure is 1. As seen in Figure 15, the new “initial” state of the
 647 embankment after those time intervals has already surpassed the *Moderate*
 648 damage level. Concerning the *Serious* damage level in Figure 17b, also for t_0
 649 = 11 years, the probability of failure depended on the severity of the ground
 650 motion. However, at 22 and 45 years, the fragility curves is lower than that
 651 at $t_0 = 0$. Thus, it can be deduced that the embankment resisted the applied
 652 load better than the beginning of its lifetime. This results points out the
 653 importance of the soil history of loading that affects its future behavior and
 654 may result in soil densification. Nevertheless, at 100 years, the probability
 655 of failure increases, and the embankment resistance to the amount of loading
 656 decreases.

657 In order to take into account different sequence example, Figure 18 shows
 658 the fragility curve of the *Serious* damage evolution for two other example se-

659 quences having different permutations of their mainshock occurrence. First,
 660 it can be seen that the result of this case is different from that in Figure 17b.
 661 For the same IM, in the case of sq14, the probability of failure decreases at
 662 the beginning of the lifetime, indicating a soil densification, after that an
 663 increase starts at 43 years. However, for higher time intervals, the probab-
 664 ility of exceeding this damage level increases. It is interesting to notice that
 665 sq14 and sq48, at almost close t_0 (i.e. purple and yellow curves), showed
 666 completely opposite behavior. sq14 indicates a probability of failure close to
 667 that at $t_0 = 0$, however, sq48 shows an increase in this probability.

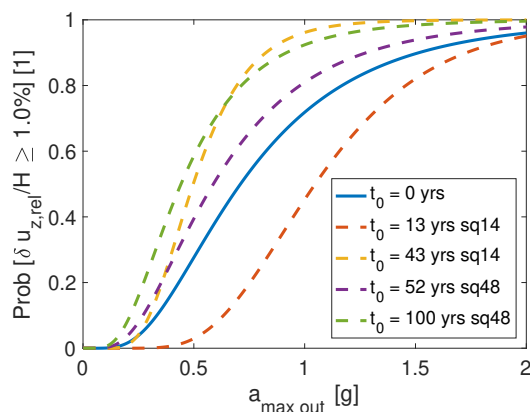


Figure 18: The evolution in time of the probability of exceeding the *Serious* damage level for different sequence examples

668 Thus, it can be deduced from these results, that the initial state of the
 669 embankment for each selected time interval is site-specific and is affected by
 670 the history of loading. Otherwise, the fragility curve computation is somehow
 671 a structure-specific approach. The probability of collapse decreases if the
 672 embankment tends to densify due to its history of loading, or the opposite
 673 if its initial state was already excessively damaged. It should be reminded

674 that the embankment during its lifetime was not subjected to any type of
675 reinforcement or amelioration.

676 **9. Discussions and Conclusions**

677 In the present paper, the work focuses on the effect of the aftershock
678 occurrence on the liquefaction-induced settlement on the embankment. It
679 presented a model to generate the aftershocks which is based on the prob-
680 abilistic version of the BASS model and is inspired by Turcotte et al. [63]
681 and Hu et al. [16]. Moreover, this work developed a methodology to take
682 into account the fragility curve evolution during the lifetime of the studied
683 embankment that analyses its potential failure and the link with the soil
684 mechanical behavior. For the sake of simplicity, the same database was used
685 to generate both MS and AFS ground motions (i.e. the stochastic ground
686 motion model proposed by Rezaeian and Der Kiureghian [44]).

687 It is important to mention that any other approaches could be imple-
688 mented, if needed, under the proposed methodology of this paper. Since the
689 proposed model is highly flexible, some aspects can be refined or modified if
690 there is a need to capture specific characteristics of the seismic environment.
691 In addition, the embankment was build on a layered homogeneous model.
692 However, in practice, significant material heterogeneities may exist.

693 As a conclusion of this work, the presented findings are based on the
694 results corresponding to the soil behavior model and the stochastic ground
695 motion model adopted in this work. Thus, the answers to the questions
696 presented in the introduction are cited correspondingly:

697 a) During sequential earthquakes, the cumulative damage of the embank-

698 ment is affected by the sequence type. It was shown in this paper that
699 the damage is higher for sequences with aftershocks (i.e. MS-AFS)
700 than the ones without the aftershocks occurrence (i.e. MS).

701 b) The survival functions with their MTTFs values, showed that the con-
702 sideration of the aftershocks events is important for moderate levels of
703 damage. However, the embankment survived high damage levels for a
704 working life of 100 years.

705 c) The evolution of the fragility curves depends on the sequence type, and
706 more importantly the initial soil state. It should be mentioned that the
707 fragility functions were not computed for the sequential cluster events
708 themselves due to some challenges (i.e. dependency of the mainshock
709 and aftershock events, the randomness of the aftershock occurrence
710 and the soil history of loading). Instead, it was evolved for some time
711 intervals of a sequence example, after a bundle of unchained recorded
712 ground motions. It was shown that the probability of failure is unpre-
713 dictable because it either increases or decreases from the very initial
714 probability of failure at $t_0 = 0$.

715 **Acknowledgement**

716 This work, within the ISOLATE project, benefited from French state
717 funding managed by the National Research Agency reference under program
718 Mobility and Sustainable Urban Systems (DS06) 2017 reference No. ANR-
719 17-CE22-0009. The research reported in this paper has been supported in
720 part by the SEISM Paris Saclay Research Institute.

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