Effect of synthetic ground motions on the liquefaction induced settlements

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ABSTRACT: In the methodology of the Pacific Earthquake Engineering Research (PEER) Center in the performance-based earthquake engineering, four stages must be studied: the hazard analysis, the structural analysis, the damage analysis and the loss analysis. Each stage has its own relation with the design model. The liquefaction apparition leads to several disastrous damages that are divided into four levels based on the crest settlement of the embankment and the peak ground acceleration (PGA) of the input signal. In this work, the effect of soil liquefaction-induced failure to a levee due to varied earthquake loading was assessed. A 2D finite element model of an embankment founded on a layered soil/rock profile was considered. An elastoplastic multi-mechanism model was used to represent the soil behavior. To account for the natural hazards, both real and synthetic input motions were used. To quantify the damage induced of the embankment the relative crest settlement was calculated and fragility curves were drawn in order to study a level of performance and to analyze the ground response.

The Pacific Earthquake Engineering Research (PEER) Center is a federally funded earthquake engineering research centers that developed a performance-based methodology for risk assessment. This methodology addresses the performance of the engineering model in terms of risk of collapse, repair costs and post-earthquake loss (Porter, 2003; Causse et al., 2014, among others). It aims at estimating the frequency of a particular performance to exceed various level of the design. It takes into account a probabilistic and a deterministic approach that are treated in four stages: the hazard analysis, the structural analysis, the damage analysis and the loss analysis. Each stage has its own relation with the design model via a performance parameter that is linked to the previous stage. Moreover, nowadays liquefaction is considered as a disastrous phenomenon that causes damages to the soil and the structures, in addition to human and economic losses. It is defined as the loss of the soil of its shear strength due to the excess of pore water pressure (Castro et al., 1982; Ishihara, 1993; Kramer, 1996, among others). The soil particles will lose their bonds and will behave like liquids. Moreover, the damage quantification due to liquefaction depends on the type of the studied structure. For an embankment for example, the crest settlement is the mode of failure to consider in order to conduct a damage analysis (Wu, 2014). Based on literature, for the case of an embankment, the liquefaction apparition has been divided into four damage levels based on the crest settlement of the embankment that is linked to the peak ground acceleration (PGA) of the input signal (Swaisgood, 2003). Hence, the effect of the ground motions is necessary to identify specially for the case of non linear behavior of the soil.

In the scope of the performance-based engineering methodology, the selection of the input ground motion is mandatory. This selection will help to determine the response of the structure in terms of probability distribution functions of the engineering demand parameter (Yamamoto and Baker, 2013). In addition, available data resources are sometimes inadequate to characterize the models due to several problems (i.e.ground motions from very large magnitude earthquakes, near-fault ground motions, basin effects) (Stewart et al., 2002). For this reason, a reference to artificial or synthetic earthquakes is conducted based on several methods (i.e. stochastic ground motion model, the composite source method, among others). These methods should be well chosen in order to represent particular conditions (Yamamoto and Baker, 2013). Synthetic motions are useful when real motions are not available.

The following paper aims to assess the effect of soil liquefaction-induced failure to a levee due to earthquakes loading and comparing the types of input motions, namely, real or synthetic. A 2D finite element model of an embankment founded on a layered soil/rock profile was considered. An elasto-plastic multi-mechanism model was used to represent the soil behavior. The methodology of the performance-based engineering was developed in this paper, three stages were considered: the hazard, structural and damage analysis. First, the real and synthetic ground motions were compared in terms of the intensity measure which was chosen to be the peak ground acceleration. The relative crest settlement of the embankment was chosen to be the engineering demand parameter. It was calculated for the tested ground motions in order to identify their effect on the ground response. And finally, the quantification of the damage was assessed based on fragility curves.

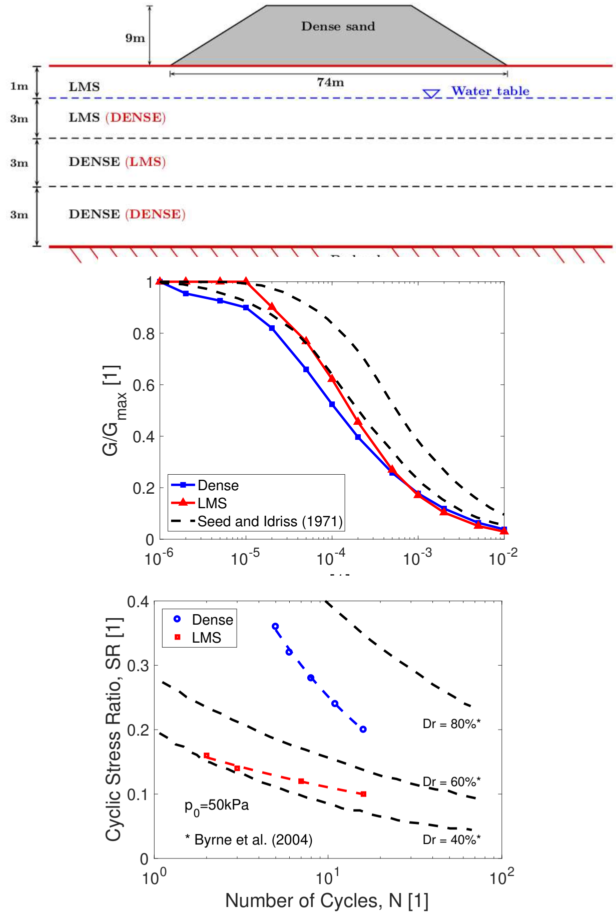

Figure 1: Gemotry and behavior of the soils used for the numerical model (Lopez-Caballero and Khalil, 2018)

1. MODEL DESCRIPTION

1.1. Geometry and FE model

The geometry of the model, as shown in Figure 1, consisted of an embankment of 9 m high composed of dry dense sand. The soil foundation consisted of a loose sand of 4 m at the top of a dense sand of 6m. The bedrock at the bottom of the dense sand is 5 m and has the shear wave velocity Vs = 1000 m/s. The water table is situated at 1m below the base of the dam and the dam was kept dry. The dam's inclination is a slope of 1:3 (vertical: horizontal).

A 2D coupled finite element modelling with GEFDyn Code (Aubry et al., 1986) is carried out using a dynamic approach derived from the $\underline{u} - p_w$ version of the Biot's generalized consolidation theory (Zienkiewicz, 1991). The FE model is composed of quadrilateral isoparametric elements (3.7 m x 1 m for the embankment with the foundation beneath and 4 m x 1 m for the free field) with eight nodes for both solid displacements and fluid pressures. The FE analysis is performed in three consecutive steps: i) a computation of the initial in-situ stress state due to gravity loads; ii) a sequential level-by-level construction of the embankment and iii) a seismic loading analysis in the time domain.

For the boundary conditions of the static phase, the horizontal displacement is blocked at the lateral surface of the meshing whereas the vertical displacement is allowed. For the base of the meshing, only the vertical displacement is not allowed. Concerning the dynamic phase, only vertically incident shear waves are introduced into the domain and as the response of an in-finite semi-space is modeled, equivalent boundaries have been imposed on the nodes of lateral boundaries. For the half-space bedrock's boundary condition, paraxial elements simulating "deformable unbounded elastic bedrock" have been used (Modaressi and Benzenati, 1994).

1.2. Input Ground Motion

In order to analyze the non linear behavior of the soil, and in the scope of the performance-based design, large number of input ground motions should be selected. In this study, synthetic ground motions are generated and the obtained FE model response is compared to the one using real recorded ground motions.

Based on literature, several methods exist for calibrating synthetic motions for specified earthquake scenarios. These calibrations are used to adjust recorded ground motions to make them more representative of the analysis conditions or where actual recordings are sparse (Stewart et al., 2002; Yamamoto and Baker, 2013, among others). For the scope of this study, the stochastic simulation technique, conducted via different types of codes, is the one used to generate synthetic ground motions. This technique tends to directly simulate the recorded ground motions with varied characteristics including the variability of the ground motion (Rezaeian and Der Kiureghian, 2012; Yamamoto and Baker, 2013, among others). In addition that it requires few parameters and is less expensive than other methods. For the sake of brevity, the details of each stochastic model are omitted, it is recommended to refer to each cited paper for more information. Three codes were used for this study to develop the stochastic simulation technique: the one conducted by Yamamoto and Baker (2013), nominated as "BKx" in this paper, the one of Rezaeian and Der Kiureghian (2012), nominated as "RZx" and finally the code of Zentner and Poirion (2012) nominated as "CAx" and "CAy". The BKx code, consists of the method of wavelet packet transform (WPT) to generate artificial ground response compatible with a target pseudovelocity response spectrum, and having non-stationary time-frequency (Yamamoto and Baker, 2013). This method requires the use of 13 parameters that are linked through regression analysis to the characteristics of the earthquake motion, such as the magnitude and distance. As for the RZx code, the method consists of rotating the recorded ground motion pairs into their principal axes. The parameters of the model are identified by fitting to each recorded pair in the new database (Rezaeian, 2010; Rezaeian and Der Kiureghian, 2012, among others). It should be noted that only the strong component was chosen. Concerning the CAx and CAy code, the method consists of generating ground motion time histories that have statistical properties compatible with the recorded accelerograms based on the method of Karhunen-Loève (Zentner and Poirion, 2012).

Furthermore, to conduct the comparison, real ground motions used for this study are nominated as "RL" and "RM". RL are motions of a moment magnitude (M_w) of 7.0 and a hypocentral distance (R) of 40.0 km (Isbiliroglu, 2018). And "RM" are real motions of M_w = 7.0 and R = 16 km. Table 1 summarizes the types and numbers of the motions used and generated for this study. The tested ground motions were used to induce the damage of an embankment due to liquefaction. The methodology of the performance-based engineering is developed in this paper, in which an intensity measure was chosen, an engineering demand parameter was selected and the damage analysis was considered.

Table 1: The characteristics of the used ground motions for this study

Name	Type	Number	Faulting	$V_{s,30}$ (m/s)
RM	Real	296	Strike-Slip	100-600
RL	Real	88	Strike-Slip	100-600
BKx	Synth	50	Strike-Slip	700
RZx	Synth	50	Strike-Slip	760
CAx	Synth	50	Strike-Slip	800
CAy	Synth	50	Strike-Slip	800

2. RESULTS

According to the performance-based earthquake engineering, the distribution of the ground motion intensity measures (IMs) is linked to the engineering demand parameters (EDPs) through probabilistic approaches (Stewart et al., 2002; Porter, 2003, among others). In this study, the IM is the peak ground acceleration (PGA) and the EDP is the relative crest settlement of the embankment. As mentioned in Section 1.2, a series of real and synthetic ground motions was selected for the purpose of comparing the ground response. The choice of artificial ground motions should be consistent with the physical conditions and characteristics of the recorded ground motions (Yamamoto and Baker, 2013). Therefore, in this study, consistency was made in terms of the peak ground acceleration (PGA) of the motion distributions. Figure 2 shows the obtained cumulative distribution function (CDF) of the tested motions as function of their PGA. Thus, "real" motions in Figure 2 refer to RL and RM whereas "synthetic" motions refer to all the sets of synthetic motions used for this study. It can be seen from Figure 2 that the chosen real

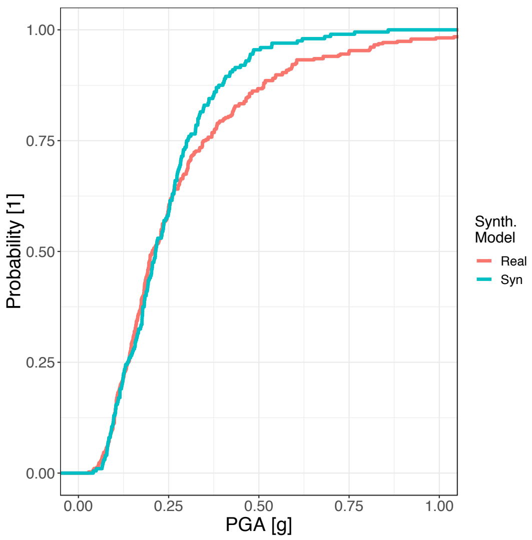

Figure 2: Cumulative distribution function of the real and synthetic ground motions

and synthetic motions are slightly different only for the case of large PGA, which means that they were well chosen. Hence, they are helpful to conduct the analysis of the damage induced of the embankment. First in this section, the intensity measure of the selected motions is studied statistically, then the chosen engineering demand parameter was calculated and compared between the different cases. Finally, fragility curves were drawn in order to quantify the damage analysis.

2.1. Intensity measure - PGA

The ground motion simulation consists of generating synthetic seismograms using analytic approaches (Stewart et al., 2002). Such simulations are important for the implementation of the performance-based engineering. The probabilistic distribution of the tested ground motions was drawn in order to assemble them according to their PGA. Figures 3 and 4 show the density and cumulative distribution functions of the different groups of real and synthetic ground motions.

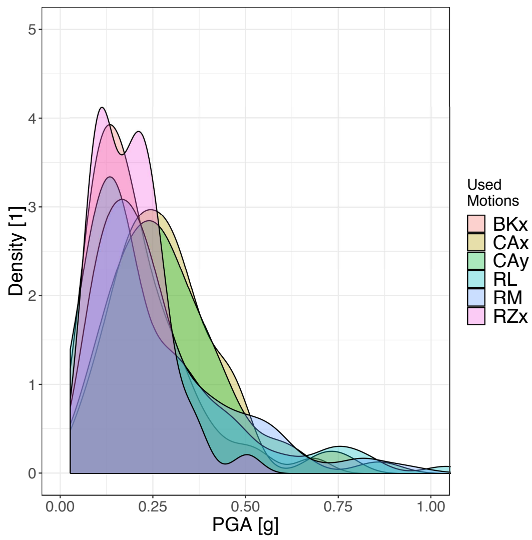

Figure 3: Density distribution function of the real (i.e. RM and RL) and synthetic ground motions

The density distribution function in Figure 3 can divide the synthetic motions into two groups that will serve for better analysis. These groups are: Baker and Rezaeian (BKx - RZx) and Code Aster (CAx - CAy). The real chosen motions are compatible between each other in addition to their compatibility with CAx- CAy. The cumulative distribution function in Figure 4 confirms this interpretation. After evaluating the ground motion intensity measures, the engineering demand parameter (EDP) at a particular set of IMs should be calculated (Stewart et al., 2002; Porter, 2003, among others). Hence, the EDP chosen for this study is

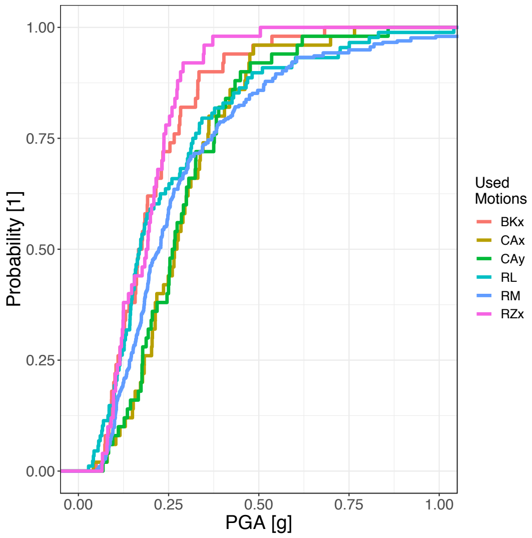

Figure 4: Cumulative distribution function of the synthetic ground motions

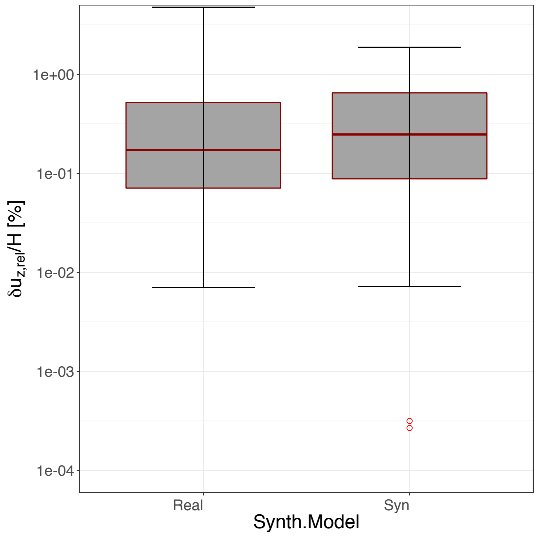

Figure 5: Box plot of the relative crest settlement for the tested real and synthetic motions

the relative crest settlement of the embankment and will be developed in the following section.

2.2. *Engineering demand parameter - crest settlement*

The structural analysis is the second stage of the performance-based engineering. This analysis consists of choosing an engineering demand parameter (EDP) to represent the response of the structure to the earthquake (Stewart et al., 2002; Porter, 2003; Lopez-Caballero and Khalil, 2018, among others). For dams under seismic activities, the mode of failure usually studied is the crest settlement because it is a quantifiable measurement. In this study, the crest settlement is chosen to be the EDP. Swaisgood (2003) analyzes a historical database on the performance of dams during earthquakes and found that the crest settlement is directly related to some input ground motion characteristics (i.e. the peak ground acceleration and magnitude). The percentage relative crest settlement is $\delta u_{z,rel}/H$ where $u_{z,rel}$ is the crest settlement and H is the height of the dam with the foundation (Swaisgood, 2003) (i.e. 19 m in this case). The relative crest settlement was calculated and compared for the real and synthetic tested motions in order to identify if the similarity in the response exits. The results are shown as box plot in Figure 5. It can be seen that there is no big difference in the median value of the percentage relative crest settlement of the real and synthetic ground motions (Figure 5). However, more dispersion was found for the real case. So for the case of this study, the damage induced on the embankment is the same if it was a real or a synthetic ground motion.

In order to ensure this interpretation, the relative crest settlement was drawn for each group of motions identified in the conducted hazard analysis (Section 2.1). Figures 6 and 7 show this variation.

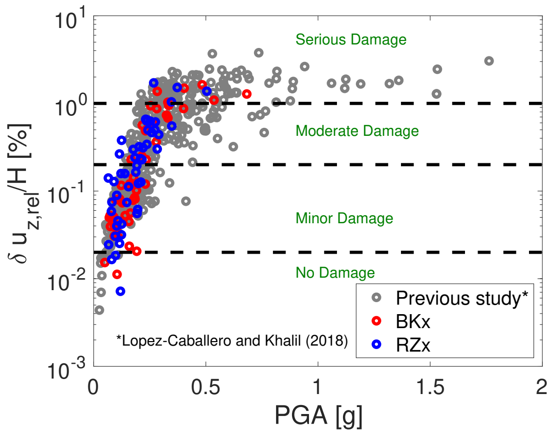

Figure 6: Percentage crest settlement of the real and synthetic ground motions selected

In order to understand the general response of the embankment, a reference to a study conducted by Lopez-Caballero and Khalil (2018) was held and is shown in Figure 6. Concerning the overall re-

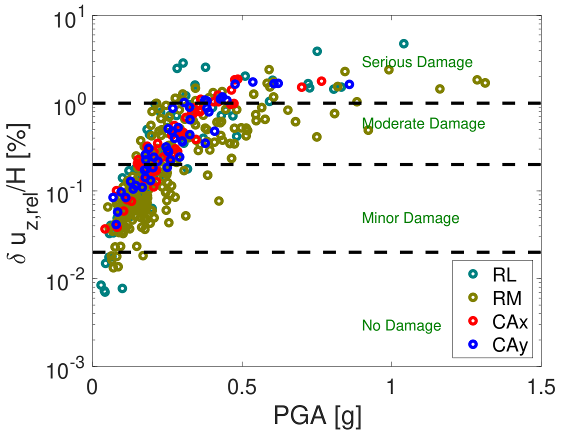

Figure 7: Percentage crest settlement of the real and synthetic ground motions selected

sponse, the percentage crest settlement increases when the PGA at the outcropping bedrock increases. This is also valid for the synthetic ground motions. Based on Swaisgood (2003), the percentage crest settlement is divided into damage levels limited by dashed lines in Figures 6 and 7.

Concerning Figure 6, the synthetic motions BKx and RZx show somehow compatible results. They give the same damage level but not the same value of the crest settlement. Whereas for Figure 7, there is a clear compatibility in the results regarding the synthetic motions. Which is normal because the difference between CAx and CAy is only the coordinates. However, comparing them with the real similar motions, it can be seen that they give the same results regarding the damage levels. But the value of the relaytive crest settlement they represent is larger than the one presented for the real cases. Hence, it can be partially concluded that for the case of this study, the chosen synthetic ground motions give similar ground response as the real motions. But in order to better quantify the structural damage of the embankment, fragility curves should be drawn in order to specify the response of the structure for a certain level of performance.

2.3. Damage analysis - fragility functions

In the context of the performance-based engineering, the damage analysis, which is the third stage of this methodology, is a procedure to quantify the structural damage (Porter, 2003; Lopez-Caballero and Khalil, 2018, among others). It consists of setting fragility functions in order to find the conditional probability of the design to exceed a certain level of performance for a given seismic input motion parameter. The performance level for this case is the “Moderate" damage so when $\delta u_{z,rel}/H$ is equal to 0.2%. Similar to the previous analysis, the fragility curves were drawn for the two selected group of motions as function of the acceleration at the outcropping $a_{max,out}$. They are shown in Figures 8 and 9.

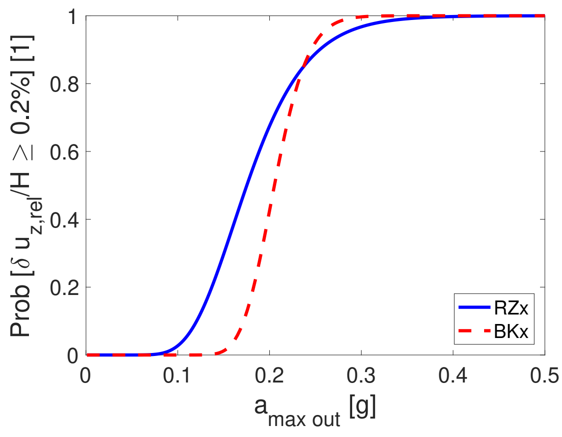

Figure 8: Fragility curves of the synthetic motions of codes Baker (BKx) and Rezaeian (Rzx)

From Figure 8, there is a small difference in the structural response between the two synthetic motions for high values of acceleration. On the contrary, for lower values of $a_{max,out}$ an important difference is identified even if there was no difference in their distribution function in Figure 4. For example, for an acceleration $a_{max,out}$ equals to 0.3g, there is 100% chance that the damage occurs based on BKx signals whereas it is an 90% chance for RZx signals. This difference in value should be taken into consideration because any change in the conditions of the model can generate a change in the structural response.

From Figure 9, as expected, it can be seen that there is an overestimation of the structural response between the real and synthetic ground motions. For an acceleration $a_{max,out}$ of 0.25g for example, and based on the real motions, there is 50% chance to generate damage above 0.2% whereas based on the consistent synthetic motions, the probability is almost 90%. From an engineering point of view, the

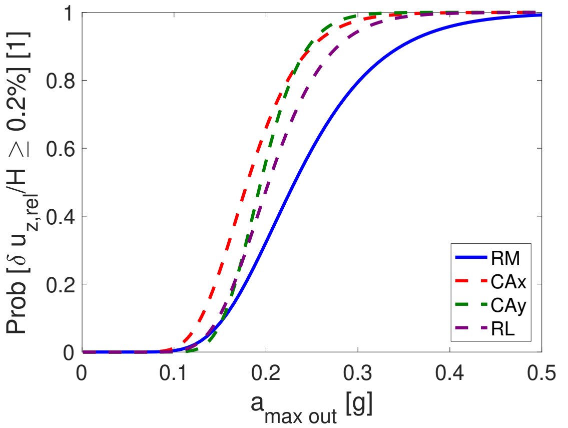

Figure 9: Fragility curves of the synthetic motions of Code Aster (CAx,CAy) and the real compatible motions

overestimation of the design is good because it will allow the improvement during the construction of the structure and it will generate higher safety factor. But for a cost analysis study, which is the last stage of the performance-based methodology, the over estimation of the design is not efficient and expensive. On the other hand, it can be seen that there is a difference regarding the response of the two real motions (i.e. RM and RL). It is due to the number of values used to compute the fragility curves; for RM there is 296 motions whereas for RL there is 88 motions (refer to Table 1) (Sáez et al., 2011).

3. CONCLUSION

The soil liquefaction induced settlement for an embankment dam due to real and synthetic earthquakes was assessed numerically in this paper. An elastoplastic multi-mechanism soil behaviour model was used with the help of a 2D finite element code (GEFDyn). The performance-based earthquake engineering methodology was investigated through three stages.

First, to account for the natural hazards, varied ground motions were chosen. The consistency between the real and synthetic motions was identified in terms of the peak ground acceleration (PGA). The chosen synthetic motions were based on the stochastic method to simulate artificial earthquakes. The results show that the chosen synthetic motions were compatible with the real ones. Two groups of synthetic motions were identified and analyzed.

As to quantify the damage subjected to the embankment, the induced relative crest settlement was chosen to be the engineering demand parameter. It was calculated for the tested motions. It was shown that there is a similarity in the global response between the real and synthetic motions; the relative crest settlement increases with respect to the peak ground acceleration for both motions. Also, they both give the same result in terms of the damage levels. Whereas, when the study consists of the level of performance, there was a discrepancy in the results between the two types of motions. For a damage level that is high then 0.2%, some synthetic motions give an over estimation of the response which is not very recommended for cost analysis.

Finally, when the recorded ground motion data are very sparse, it is a good idea to chose synthetic motions. But a good care should be made on the choice in order to represent the real case scenarios. Structure specific ground motions are more representative of the structural response than site specific motions. Only for specific cases, the synthetic motions give the same ground response as the real ones.

4. ACKNOWLEDGMENT

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

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