Evaluation of the Seismic Performance of Small Earth Dams

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Abstract. The evaluation of the seismic response of earth dams is a nontrivial task, which generally requires the use of advanced soil models able to accurately reproduce the material behaviour under dynamic loadings. The reliability of the numerical simulations is however constrained by the level of knowledge of the geotechnical model of the dams. The issue is particularly relevant when small earth dams, characterized by a reduced height and a limited reservoir volume, are considered. Such structures indeed frequently lack proper characterization of the materials constituting the dam body and its foundation. The implementation of dynamic numerical analyses is therefore limited in the common practice and the seismic performance of the dams are frequently assessed through simplified empirical methods. This study investigates the seismic behaviour of two small earth dams for which a reliable geotechnical model, based on both laboratory and *in situ* tests, is available. The seismic responses of the dams are analyzed through fully coupled effective stress dynamic analyses. The analyses are developed within the context of the ReSba European project for the French-Italian Alps area. The results have allowed comparing the seismic performance of the structure as predicted by simplified and advanced approaches in terms of stability conditions and seismic-induced settlement of the crest.

Keywords: Earth dam, Seismic Response, Seismic Performance, Effective stress analysis, Earthquake.

1 Introduction

The evaluation of the risk associated with existing dams is a major issue in high seismicity regions. The seismic-induced inertial forces may compromise the stability of the embankment, leading to the accumulation of permanent crest settlement and reducing, in turn, the freeboard. As a consequence, several studies have been devoted in the past to the assessment of the seismic performance of earth dams (e.g. [1]). Such an assessment can be carried out by employing different approaches, namely pseudo-static numerical analyses, simplified empirical relationships, displacement-based approaches derived from Newmark's rigid block method [2], and more accurate dynamic numerical simulations. These methods vary from simplified to highly sophisticated and, thus, require different levels of knowledge of the geotechnical model of the site.

According to the Italian National Code [3], the seismic response of existing dams should be assessed by following the principle of *gradualness*, thus selecting the method of analysis according to the amount of available information. Sophisticated methods may be suitable for studying the performance of *large dams* (i.e. dams with a height >15 m or/and a reservoir volume >10^6 m^3 [3]) for which a comprehensive characterization is usually available. Conversely, *small dams*, characterized by modest height and reservoir volume, frequently lack proper geotechnical information, therefore limiting the implementation of advanced numerical simulations. As a consequence, a first-level screening is usually carried out by using empirical relationships to identify the small dams for which more refined analyses are required. Despite their limited size, the risk associated with the potential failure of the small dams is indeed considerable due to their proximity to populated areas.

The ReSba (Resilience of Dams) project, funded by the European fund for regional development (Interreg-ALCOTRA), fits in this context intending to improve our ability to assess the natural risks associated with dams located in the French-Italian Alps area. Within the project, a simplified methodology has been firstly developed to classify the embankments according to the associated seismic risk [4]. The methodology has been then applied to select the small earth dams classified from medium to high seismic risk [5]. This study presents the results of the dynamic numerical simulations performed to analyze the seismic performance of two of them, namely the Arignano and Briaglia dams. The fully coupled analyses are carried out with due consideration of the possible pore water pressure build-up due to the coupled shear-volumetric response of granular soils. The results of the simulations are used as a benchmark for testing the ability (and the drawbacks) of the simplified empirical relationship proposed by Swaisgood (2003) to estimate the seismic-induced settlements of dams.

2 Geophysical and geotechnical survey

The Arignano and the Briaglia dams are two small earth dams located in the Piedmont region in Italy. Both the dams have been the subject of an extensive geophysical and geotechnical survey carried out within the ReSba project. In the following, the main results of this survey are briefly summarized. Further details can be found in [5].

2.1 Arignano dam

The dam was built in the second half of the 19th century in Arignano (Turin, Italy) and dikes the course of the Rio del Lago stream forming a reservoir with a volume of about 640,000 m^3. The embankment has a trapezoidal cross-section with a crest 380 m long and 5 m wide, and a height of about 7 m.

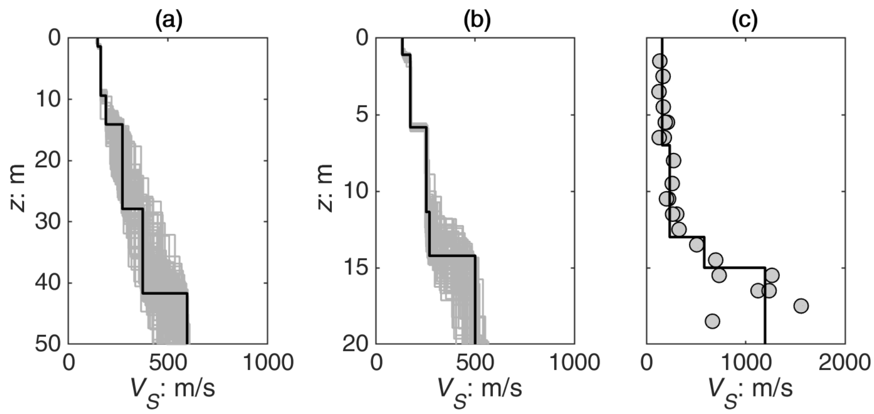

Fig. 1. Results of the MASW tests carried out along the crest of the Arignano (a) and Briaglia (b) dams (the thick lines are the minimum misfit profiles); (c) results of the DH test performed on the Briaglia dam.

A first geotechnical survey was conducted in 2004, comprising 12 Standard Penetration tests, 5 boreholes, and 4 Lefranc variable head permeability tests. Additionally, direct simple shear, unconfined undrained triaxial and oedometric tests were performed on undisturbed samples retrieved from the boreholes. The geophysical characterization was subsequently carried out within the framework of the ReSba project, involving the execution of active Multi-Channel Analysis of Surface (MASW) tests and single-station Horizontal-to-Vertical Spectral Ratio (HVSR) tests. Passive 2D array measurements were also carried out downstream from the dam. The experimental data were processed in the *f-k* domain using the Surface Wave. The main results of the geophysical tests performed along the dam crest are presented in Fig. 1a in terms of a statistical sample of the shear wave velocity V_S profile.

A geotechnical model of the site was built based on the results of the laboratory and *in situ* tests. According to the model, the embankment is constituted by a 2 m thick layer of sand and silt overlying 5 m of plastic clays and silts. The foundation consists instead of 2 m of clayey silts, followed by 10 m of fine silty sand overlying two layers of soft-to-hard clays with stiffness increasing with depth. The seismic bedrock is a deep layer of marlstones with sandstone inclusions. A summary of the mechanical parameters of the materials is given in Table 1.

Table 1. Geotechnical parameters adopted for the numerical model of the Arignano dam.

Layer	z: m	V_S: m/s	k: cm/s	φ': °	c': kPa	MRD curves	Finn-Byrne parameters
Dam 1	0.0 ÷ 2.0	150	$4{\cdot}10^{-7}$	21	12	Seed et al. (1986)[6]	C_1=0.3÷0.7 C_2=0.6÷0.3
Dam 2	2.0 ÷ 7.0	160	$4{\cdot}10^{-7}$	21	12	Sun et al. (1988)[7]	-
Found 1	7.0 ÷ 9.0	160	$4{\cdot}10^{-7}$	21	12	Sun et al. (1988)[7]	-
Found 2	9.0 ÷ 19	165	$8{\cdot}10^{-7}$	21	12	Seed et al. (1986)[6]	C_1=0.09÷0.5 C_2=0.4÷2.2
Found 3	19 ÷ 40	350	10^{-9}	30	1	Sun et al. (1988)[7]	-
Found 4	40 ÷ 51	600	10^{-9}	30	1	Sun et al. (1988)[7]	-
Bedrock	>51	1050	10^{-9}	-	-	-	-

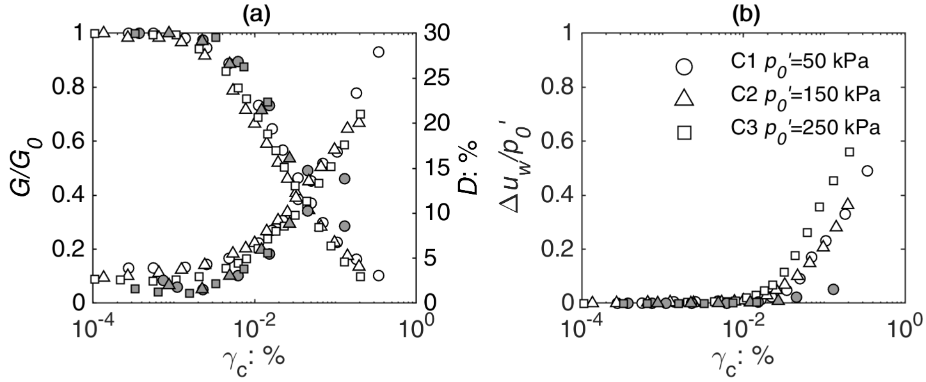

Fig. 2. MRD curves from RC/TS tests (a, the filled markers are the TS tests) along with r_u (b).

2.2 Briaglia dam

The dam is located within the municipality of Briaglia (Cuneo, Italy) and it was built at the beginning of the 1990s to retain a reservoir volume of about 59,500 m^3 from the Rio del Frocco river. The embankment is characterized by a height of about 11 m and a crest 88.5 m long and 4 m wide.

No prior information was available regards the foundation and structure material, the site was therefore object of an extensive geotechnical and geophysical survey, which included: a borehole on the crest of dam instrumented to conduct Down Hole tests; active MASW and passive HVSR tests; a seismic cone penetration (SPCTU) test and a flat-dilatometer (DMT) test. The main results of the geophysical investigations are presented in Fig. 1b in terms of a statistical sample of the V_S profiles from MASW test, whereas Fig. 1c present the results of the DH test.

Laboratory tests were also conducted on undisturbed samples. Specifically, isotropically consolidated undrained triaxial tests and oedometric tests were carried out to define the mechanical properties of the materials. In addition, combined Resonant Column and Torsional Shear (RC/TS) tests were performed to investigate the soil dynamic behaviour. The results of the tests are presented in Fig. 2a in terms of Modulus Reduction and Damping ratio (MRD) curves along with the pore water pressure build-up normalized to the initial confining pressure $r_u = \Delta u_w/p_0'$ (Fig. 2b).

Table 2. Geotechnical parameters adopted for the numerical model of the Briaglia dam.

Layer	z: m	V_S: m/s	k: cm/s	φ': °	c': kPa	MRD curves	Finn-Byrne parameters
Dam 1	0.0 ÷ 4.5	160	10^{-9}	32	1	RC test - C1	C_1=0.09 C_2=2.2
Dam 2	4.5 ÷ 6.8	190	10^{-10}	32	1	Darendeli (2001)[8]	C_1=0.08 C_2=2.4
Dam 3	6.8 ÷ 7.6	230	10^{-8}	38	1	Rollins et al. (1998)[9]	-
Dam 4	7.6 ÷ 14	270	10^{-8}	35	1	Seed et al. (1986)[6]	C_1=0.10÷0.12 C_2=1.7÷2.0
Found 1	14 ÷ 20	500	$5\cdot10^{-10}$	38	1	RC test - C2	-
Found 2	20 ÷ 30	800	10^{-11}	34	1	RC test - C3	-
Bedrock	>30	1500	10^{-11}	-	-	-	-

The embankment is constituted by silts and clayey silts over a thin layer of weathered sandstones overlying a 6.4 m thick layer of clayey silty sand. The dam is built on a 6 m thick layer of medium-hard clays overlying a stiffer deposit of marlstone. The latter constitutes the seismic bedrock, reaching high V_S values (≈1500 m/s) at a depth of about 30 m. The geotechnical parameters of the dam are reported in Table 2.

3 Numerical modelling

The fully coupled effective stress analyses are performed using the Finite-Difference code Flac 2D (Itasca C.G.). The soil stress-strain behaviour is modelled through an elastic-perfectly plastic constitutive model with a Mohr-Coulomb failure criterion, coupled with a hysteretic formulation to capture the nonlinear soil response. The parameters of the hysteretic model are calibrated based on the results of RC/TS tests (Fig. 2) and widely-used empirical models. Additionally, a small amount (<1%) of Rayleigh viscous damping is added to consider the energy dissipation at small strains.

The simulations are carried out by taking into account the eventual pore water pressure build-up due to the coupling between shear and volumetric strains above the volumetric shear strain threshold. Such feature is considered for the sandy and non-plastic silty soils by employing the empirical model proposed by Byrne [10], with constitutive parameters defined according to the following relationships:

$$C_1 = 7600 D_r^{-2.5} \quad C_2 = 0.4/C_1 \tag{1}$$

being D_r the relative density of the soil, defined through the CPT and SCPT tests.

The numerical field is discretized through four-sided mesh elements with sizes defined according to the Kuhlemeyer and Lysmer [11] criteria to ensure the accuracy of the wave propagation phenomena. The geotechnical models developed to study the Arignano and Briaglia dams responses are presented in Fig. 3. The simulations comprise three numerical steps: (i) definition of the geostatic stress-state; (ii) hydro-mechanical seepage analysis; (iii) fully coupled dynamic analysis. During the static steps, elementary boundary conditions are employed. Conversely, free-field lateral conditions and base absorbing dashpots are introduced under dynamic conditions to avoid undesired waves reflections [12].

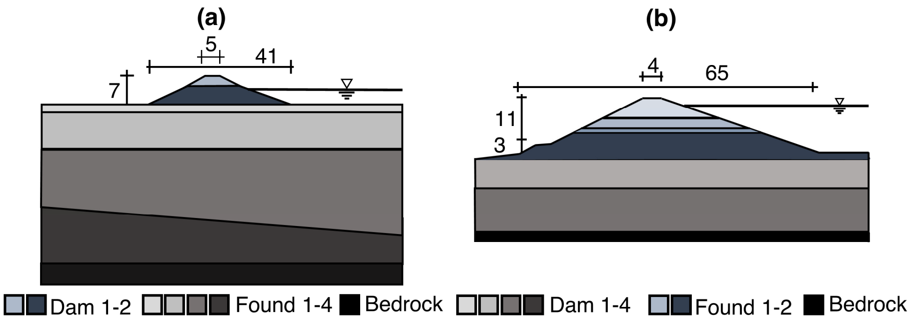

Fig. 3. Geotecnical models for the Arignano (a) and Briaglia dams (b) (dimensions in m).

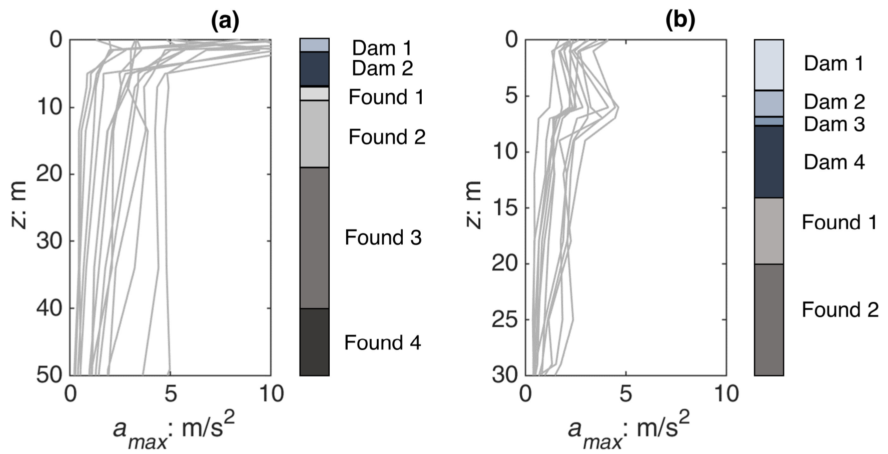

Fig. 4. Maximum acceleration profiles recorded along the axes of the (a) Arignano and (b) Briaglia dams.

The dynamic analyses are performed by applying input strong motions recorded on reference rock-like outcropping conditions. The latter were selected from the ITACA archive [13] in compliance with the seismicity of the Piedmont region and, specifically, of the sites of the dams. Fig. 4 reports, as an example, the maximum acceleration a_{max} profiles recorded along the axes of the models for some selected input motions. For the Arignano model, the acceleration profiles slightly increase up to a depth of about 7 m, after which there is an abrupt increase of a_{max} due to the 2D topographic effects induced by the dam body (Fig. 4a). Conversely, a reduction of a_{max} can be observed within the Briaglia dam (Fig. 4b) as a consequence of the large amount of energy dissipated by the first 6 m of soil characterized by significant pore water pressure build-up.

4 Empirical relationships for predicting the crest settlement

The results of the numerical simulations are here employed to assess the performance of the empirical relationship proposed by Swaisgood [14] to estimate the permanent settlement of the crest w normalized to the height of the dam H_d plus the thickness of the alluvium H_f in percentage:

$$w\left(\%\right) = \frac{\Delta H}{H_d + H_f} = e^{6.07 \cdot PGA + 0.57 \cdot M - 8} \tag{2}$$

The numerical w are compared to Swaisgood's equation [14] in Fig. 5a as a function of the PGA. The empirical relationship seems to be in good accordance with w obtained for the Arignano dam, although it slightly overpredicts w at low PGA (Fig. 5a). Conversely, the equation significantly underestimates w for the Briaglia dam. Similar trends can be observed by looking at the comparison between numerical w and predicted w_{pred} settlement in Fig. 5b.

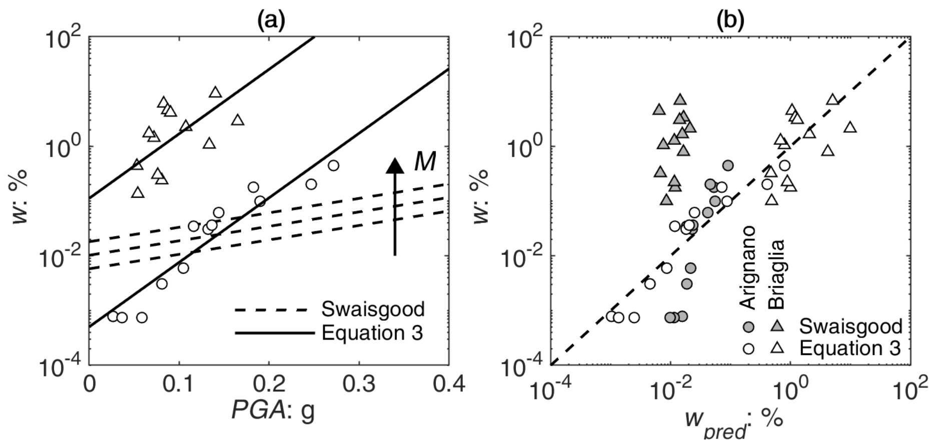

Fig. 5. (a) Simulated permanent settlement versus *PGA* and (b) simulated versus predicted settlement as obtained from Swaisgood's equation [14] and Eq. 3.

The data does not follow the diagonal of the plot, revealing a systematic bias coming from the application of Eq. (2). Figs. 5 also report the predictions from relationships calibrated through a nonlinear least-squares procedure on the results of the simulations. In particular, the equations here adopted are expressed in the following, general, form:

$$w(\%) = A \cdot e^{\alpha \cdot PGA} \tag{3}$$

where A and α are calibration parameters. It is worth noting that although the calibrations were initially performed separately for the two dams, the obtained values of the parameter α, defining the slope of the lines, resulted to be in very good accordance. It was therefore decided to adopt the same value, equal to 27.1, for the two dams. Conversely, the value of A (equal to $5 \cdot 10^{-4}$ and 0.11, respectively for the Arignano and Briaglia dams), seems to be characteristic of each dam. The differences in this parameter are attributed mainly to the specific geometric features of the dams, as well as to the different thicknesses of the soil layers affected by pore water pressure build-up. Further parametrical analyses on dams characterized by different geometric features are thus required to derive an empirical relationship for linking A to the specific characteristic of the structure.

5 Conclusions

This study has investigated the ability of the most used empirical equations for predicting the seismicity-induced permanent settlement of the crest of small earth dams. For this purpose, fully coupled dynamic numerical simulations were performed to analyze the seismic response of two embankments for which reliable geotechnical models were available based on laboratory and *in situ* tests.

The results of the analyses have shown that care should be exercised when such equations are employed in the common practice, as the seismic damage of the dams may be significantly underestimated. Nevertheless, the results of the analyses also suggest that the estimate may be substantially improved by explicitly including in the formulations of the empirical relationships the influence of the geometry of the dam and the thickness of the soil layers prone to pore water pressure build-up.

6 Acknowledgement

This study was supported by the European Union for regional development (Interreg-ALCOTRA) for the French-Italy Alps, within the ReSba project, for which funding is gratefully acknowledged. The Auhors thank the Piedmont Region Administration for their availability and support. The Authors wish also to thank F. Passeri, PhD, and L. Fazio, MSc, who worked actively on the project.

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