Coupled hydro-mechanical modelling of the seismic response of a zoned earth dam

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ABSTRACT

Global climate change is leading to prolonged periods of low rainfall followed by intense thunderstorms, causing severe droughts in vital water resources like rivers and reservoirs. Additionally, heavy rainfall events are causing landslides and significant damage in many countries. Dams and artificial water reservoirs are crucial in mitigating these risks. Given the limited availability of suitable sites for constructing new reservoirs, existing dams must be safeguarded, although they also pose seismic vulnerability risks due to the uncontrolled release of water. Therefore, it is crucial to assess the seismic performance of earth dams and plan rehabilitation works accordingly. This paper focuses on the coupled hydro-mechanical modelling of the seismic response of a zoned earth dam. The model was calibrated using results from a geotechnical centrifuge test, simulating the impoundment phase and applying increasing levels of seismic input. The comparison between the predicted and observed behaviour demonstrated the efficacy of the finite element (F.E.) model in accurately capturing the key characteristics of the response exhibited by this complex geotechnical system when subjected to dynamic loads.

Keywords: earth dams, fully coupled hydro-mechanical approach, dynamic analyses, centrifuge test

1 INTRODUCTION

The increasing need for rational utilization of diminishing water resources and effective flood protection draws urgent attention to the construction of dams in Italy and worldwide. This involves completing interrupted projects and ensuring proper maintenance of existing structures, many of which are now over 50 years old on average. Currently, only 70% of Italy's 542 large dams are operating normally. By converting the structures currently in experimental operating service, the percentage of utilized water resources could increase from the current 47% to almost 80% of the total potential availability. Moreover, restoring dams that are under limited operating service can yield additional benefits at a relatively low cost. Therefore, focusing on existing dams rather than constructing new ones is strategically important. However, large dams also pose a significant risk in terms of seismic vulnerability, as the uncontrolled release of millions of cubic meters of water can cause extensive property damage and loss of human life. Therefore, it is of utmost importance to assess the seismic performance of existing earth dams and plan potential rehabilitation measures.

The study of the stability of earth dams under seismic conditions is particularly complex due to the dynamic nature of the forces involved. The impact of a seismic event on an earth dam results from both the inertial forces caused by the earthquake's accelerations and the reduction of shear resistance in the materials of the dam's body. Currently, coupled dynamic numerical analyses are considered the most reliable and promising method for investigating the behaviour of complex geotechnical systems like earth dams. These analyses accurately simulate the problem's geometry, boundary conditions, and dynamic soil behaviour. They enable to account for factors such as the partial saturation of the embankment and the complex coupled hydro-mechanical behaviour of soils.

The evaluation of the seismic performance of large earth dams through numerical analyses can be performed using two approaches: in the first one, a numerical model is created for an existing structure, based on a specific characterization of soil parameters and, when available, on-site measurements of real seismic events, usually of low amplitude; this model is then subjected to scenario earthquakes associated with increasing return periods, to evaluate the portions of the structure where damage occurs and the physical quantities that best represent this damage (e.g.: Rampello et al. 2009; Elia et al., 2011; Elia and Rouainia, 2013; Sica et al., 2008; Albano et al., 2015; Pelecanos et al., 2015; Han et al., 2016; Casablanca et al., 2022).

In the second approach, a conveniently simplified model is adopted to study the response of typical structural and geotechnical systems (e.g.: Papadimitriou et al., 2014; Andrianopoulos et al., 2014, Masini et al., 2016, 2019, 2021; Masini and Rampello, 2022). In this case, the adopted constitutive models are usually simpler and typological models of the dams are subjected to pseudo-static or time-domain dynamic analyses to assess the patterns of deformation in the dam body and to evaluate relevant quantities, such as permanent displacements, that should not be intended as a prediction of actual performance of the earth structure, but rather as an index of seismic performance.

However, the calibration of the constitutive models adopted and the definition of the initial values of the state parameters still involves significant uncertainties. In this context, dynamic tests carried out on large (e.g., Di Filippo et al., 2023) and small (e.g. Fioravante et al., 2021) scale physical models under laboratory-controlled conditions can provide a valuable insight into the system response and usefully serve as a mean of validation of the numerical analyses aimed at predicting the seismic performance.

This paper discusses some aspects of the coupled hydro-mechanical modelling of the seismic response of a zoned earth dam. The numerical model was calibrated based on the results of a geotechnical centrifuge test that simulated the impoundment phase and applied three seismic inputs of increasing intensity. The numerical analyses were conducted using the PLAXIS 2D F.E. code. The Hardening Soil with Small Strain constitutive model was utilized to describe the mechanical behaviour of the dam materials, which was calibrated using laboratory tests performed to characterize the materials' response under both fully and partially saturated conditions.

2 GEOTECHNICAL CENTRIFUGE TEST

As part of the PRIN REDREEF project (Risk Assessment of Earth Dams and River Embankments to Earthquakes and Floods), two geotechnical centrifuge tests were carried out on a zoned earth dam model at ISMGEO (Istituto Sperimentale Modelli GEOtecnici, Italy). The ISMGEO geotechnical centrifuge consists of a symmetrical rotating arm with a diameter of 6 m, a height of 2 m, a width of 1 m and a nominal radius of approximately 2.2 m to the model base. This centrifuge has a capacity of 240 g-tons, i.e. it has the potential to achieve accelerations of up to 600 g while holding a payload of 400 kg (Baldi et al., 1988). A shaking table is integrated into the centrifuge equipment, allowing it to operate under an artificial acceleration field of up to 100 g. This table can provide excitations at frequencies of up to 500 Hz, generating seismic accelerations of up to 50 g. This shaker can faithfully reproduce real strong motions at the model scale (Airoldi et al., 2016). The model is housed in an aluminium container measuring 800x395x330 mm. Figure 1a shows an image of the model housed within the container, and Figure 1b depicts the layout of the instrumentation. The geometric scaling factor of the physical models was N = 50, corresponding to a zoned earth-dam prototype with a height of 13 m, a width of 39 m and a slope of 35°. An upstream reservoir level of 11 m was simulated (220 mm at the scale of the model) to reproduce operational conditions. The model was equipped with accelerometers, pore pressure transducers (PPT), tensiometers (T) and displacement transducers to measure horizontal accelerations along the shaking direction, fluid pressure and displacements respectively. In addition, an accelerometer was attached to the base of the model container to record the seismic input from the shaking table.

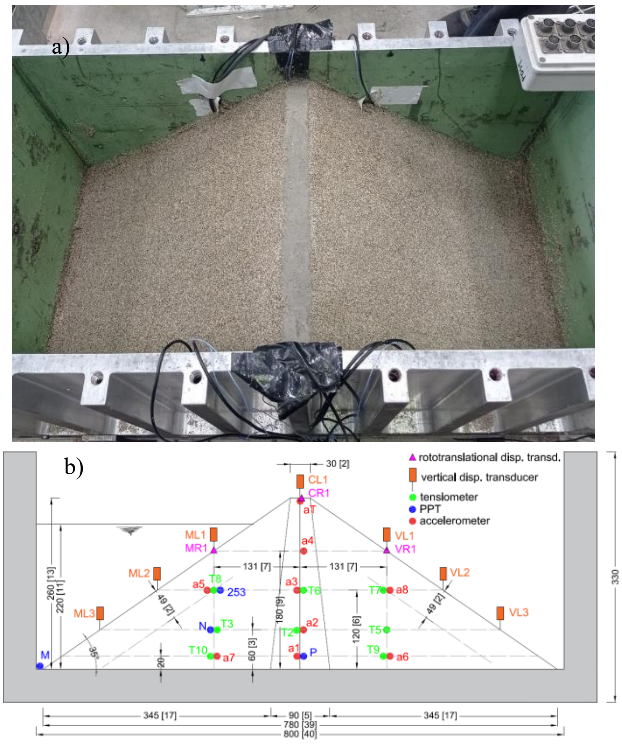

Fig. 1. Top-view of the dam model tested in the centrifuge (a), and layout of the instrumentation (b) – dimensions at the prototype scale are reported in the square brackets.

3 NUMERICAL MODEL AND CALIBRATION

The numerical model of the earth dam is shown in Fig. 2. The domain is discretised in 4523 triangular 15-noded elements, with a total of 37035 nodes and the analyses are performed in plane strain conditions. The soil of the shells is the Ticino sand, characterised by a unit weight γ = 20 kN/m^3 and void ratio e = 0.622, corresponding to a relative density Dr = 87%, while the core is constituted by the Pontida clay, with unit weight γ = 19.2 kN/m^3 and void ratio e = 0.404.

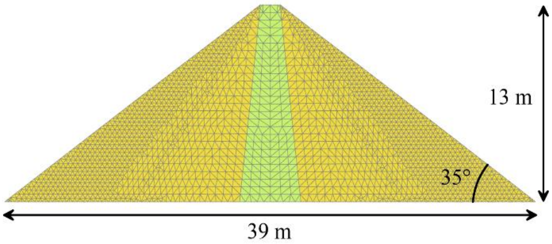

Fig. 2. Finite element model of the earth dam.

Table 1. Parameters of the HSSmall model

Parameter	Ticino sand	Pontida clay
c' (kPa)	1.0	14
φ' (°)	41	34
ψ (°)	0	0
E'_{ref} (MPa)	396	228
E'/E'_{50}	20	26.8
E'_{50}/E'_{oed}	1.4	1.55
$E'_{50,ref}$ (MPa)	19.8	8.5
$E'_{oed,ref}$ (MPa)	14.1	5.5
E_{ur} (MPa)	59.4	27.1
G_{0ref} (MPa)	165	95
$\gamma_{0.7}$	1.7e-4	1.7e-4
m	0.5	0.98

The mechanical behaviour of the soils is described through the *Hardening Soil with Small Strain Stiffness* (HSSmall) constitutive model (Schanz et al., 1999), whose parameters are summarised in Table 1.

The shear modulus reduction curve has been calibrated on a series of experimental data available in the literature (Fioravante and Giretti, 2016; Fioravante and Jamiolkowski, 2005; Fioravante, 2000) as shown in Fig. 3, while the damping ratio is a response of the HSSmall model. The remaining parameters have been calibrated on a series of conventional oedometer and triaxial compression tests carried out in the geotechnical laboratory of University of Naples Federico II as a part of the project PRIN REDREEF. To account for the partial saturation of the soils, the Van Genuchten model has been adopted, with the parameters reported in Table 2.

Fig. 4 shows the comparison between the retention curves predicted by the Van Genuchten model and the experimental data by Ventini et al. (2021) in terms of degree of saturation and relative permeability (i.e. the ratio between the current permeability and that for fully-saturated soil).

The numerical analyses have been performed according to the following phases, consistent with the centrifuge test model: (1) initialisation of the model stress state through the instantaneous activation of the model's weight; (2) impoundment phase, progressing in three stages until the water level reached 11 meters; (3) dynamic phase involving the application of four artificial accelerograms at the base of the model, with seismic parameters reported in Table 3.

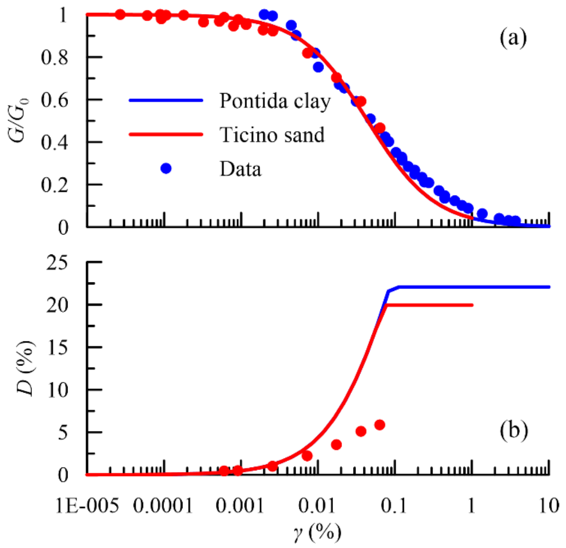

Fig. 3. Shear modulus reduction curves (a) and damping ratio (b) for the tested soils

Table 2. Parameters of the Van Genuchten model

Parameter	Ticino sand	Pontida clay
k_{sat} (m/s)	1.72e-3	2.83e-10
S_{res}	0.156	0.33
S_{sat}	1	1
g_n	5	1.16
g_a (m^{-1})	6.38	0.45
g_c	-0.8	-0.137
g_l	0.5	-4.63

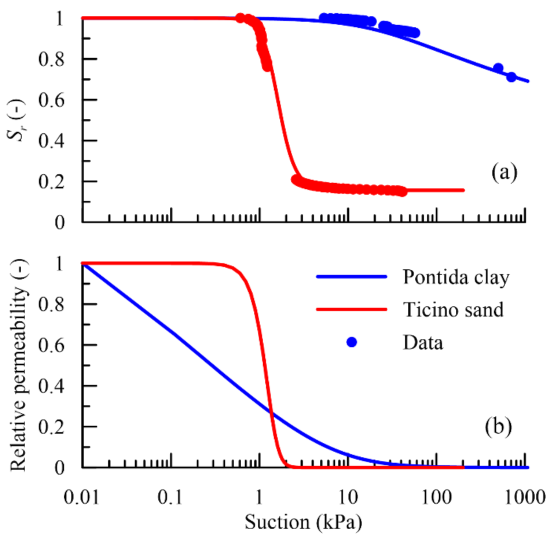

Fig. 4. Retention curves in terms of degree of saturation (a) and relative permeability (b) for the tested soils.

4 RESULTS OF THE NUMERICAL ANALYSES

4.1 Static impoundment phase

The impoundment phase was numerically simulated by a fully coupled analysis to account for the soil skeleton and fluid coupling during reservoir filling. A time-dependent variation of the external water level at the upstream boundary of the dam was imposed to schematically model the three reservoir levels reached during the centrifuge test. Figure 5 shows the contours of the calculated pore water pressure in the dam body at the end of the impoundment stage. The positive values reported in the figure indicate suction, according to Plaxis 2D sign convention.

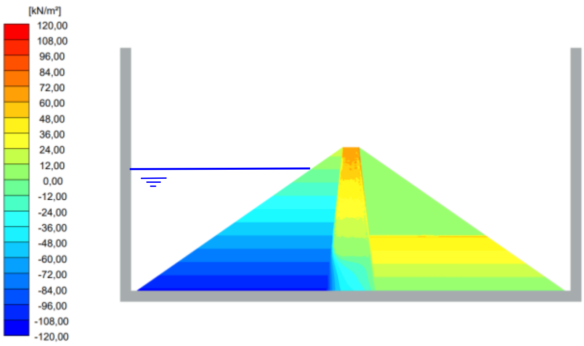

Fig. 5. Contour of pore water pressure computed at the end of the impoundment phase.

Suction values close to 70 kPa were calculated at the top of the clayey core, while the maximum suction calculated in the downstream sandy flank is about 40 kPa.

Figure 6 shows the comparison between the calculated and measured time histories of pore water pressure during the impoundment phase at some relevant points. In the following, negative values indicate suctions, according to the commonly adopted geotechnical convention. The T10 tensiometer, at the base of the upstream shell (Figure 1), shows a prompt response as the reservoir level rises, following the trend of the PPT M at the bottom of the reservoir. The time history calculated at the same point agrees well with the observed response. The pore pressures measured by tensiometer T3 and PPT N, located at the same height on the upstream side, are significantly different in the early phase of the impoundment. This may be partly due to the inability of the PPT to measure suction. However, both instruments reach approximately the same value when the maximum reservoir level is reached. The FE model shows a faster response when the reservoir level is changed, while the final value is close to the measured values. The time history computed at the location of tensiometer T2 inside the core fairly agrees with the observed behaviour, although some time-lag can be observed. The same trend is observed at the location of the tensiometer T9. As a general remark, the FE model is able to reproduce with fair approximation the observed behaviour in terms of pore water distribution in the embankment.

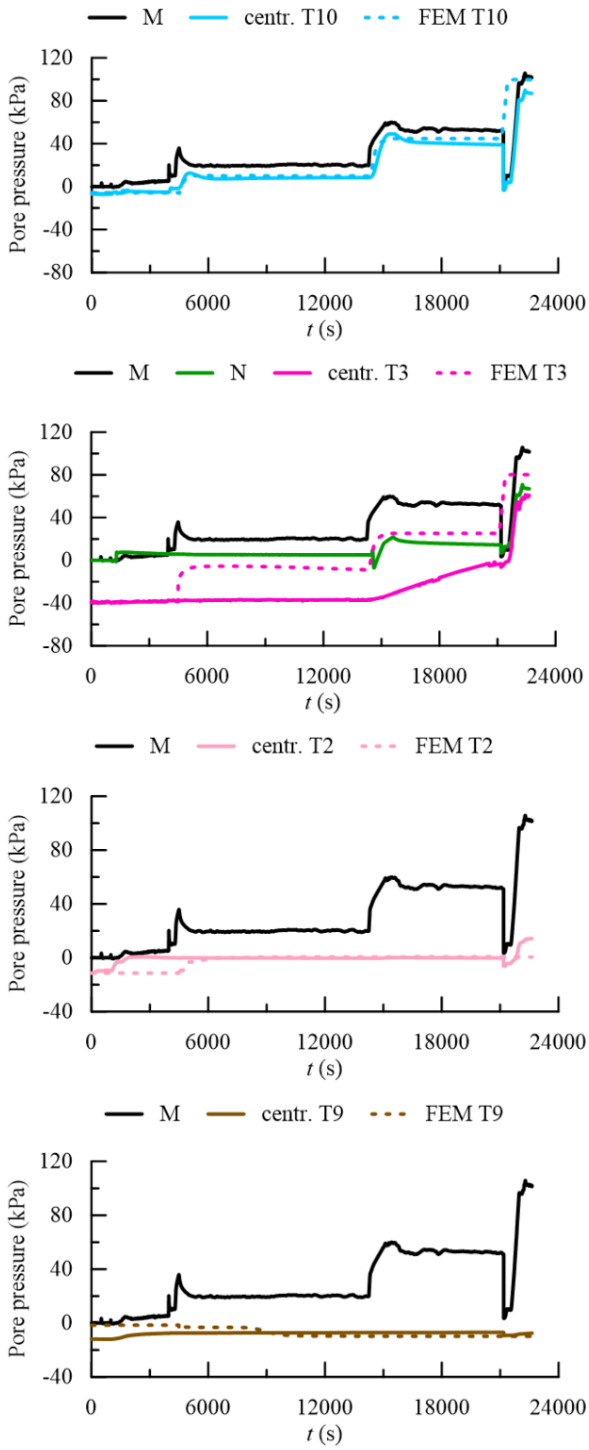

Fig. 6. Calculated (FEM) and measured (centr.) time histories of pore water pressure during the impoundment phase at the position of the PPTs and tensiometers.

4.2 Dynamic phase

The dynamic phase was simulated by performing a coupled consolidation time domain dynamic analysis by applying the same real time history of acceleration four times in succession with increasing peak acceleration

(PGA, Table 3). For the sake of brevity, only the results related to the application of the most severe input (#4) are presented. Fig. 7 compares the calculated and measured time histories of horizontal acceleration a_x (Fig. 7a) and the corresponding Fourier spectrum (Fig. 7b) at the dam crest (accelerometer a_T in Figure 1b). A good agreement can be found in terms of the maximum acceleration amplitude of the two signals. Specifically, the maximum horizontal acceleration measured during the test in a_T is approximately 0.81g while the numerical analysis leads to a 0.83g. Different considerations arise from an inspection of the plot in Fig. 7b; although the frequency band where amplification occurs show similar bounds, different values of spectral amplitude have been found, indicating that in the experimental model, higher amplification occurs than in the numerical model. This different behaviour could be due to the increase in stiffness generated by partial saturation in the upper part of the core.

Table 3. Seismic parameters of the input motions.

ID	PGA (m/s^2)	PGV (m/s)	PGD (m)	I_a (m/s)	T_m
#1	1.70	0.083	0.015	1.92	0.25
#2	1.79	0.086	0.006	1.24	0.21
#3	2.39	0.133	0.026	2.62	0.25
#4	2.56	0.154	0.015	3.91	0.22

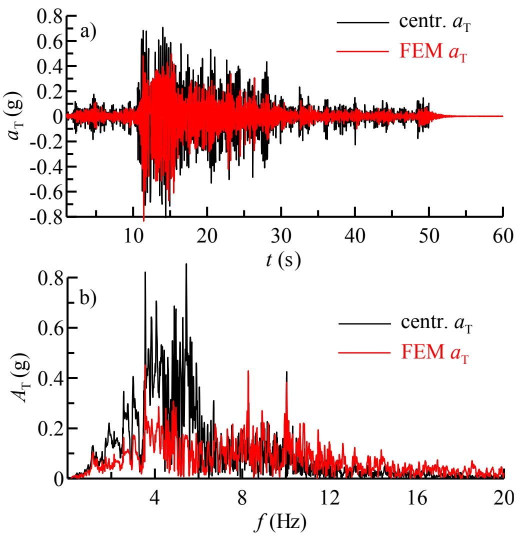

Fig. 7. Comparison between the calculated and measured (a) time histories of the horizontal acceleration and the corresponding (b) Fourier spectra at the dam crest.

Fig. 8 show the contour lines of the vertical (Fig. 8a) and horizontal (Fig. 8b) displacements (in meters) at the end of the last motion applied at the base of numerical model. The distribution of permanent displacements at the end of the earthquake is not symmetric and denotes the propensity of the dam to develop higher displacements in the upstream area where the reservoir is present. In fact, the sand of the upstream side is largely in conditions of total saturation, while the upper portion of the core benefits from an increase in effective stress state due to partial saturation. Larger displacement gradients develop within the shell dam, while the core exhibits a typical cantilever beam-like response. This behaviour can be attributed to the relatively high shear strength of the core and to the effects of partial saturation.

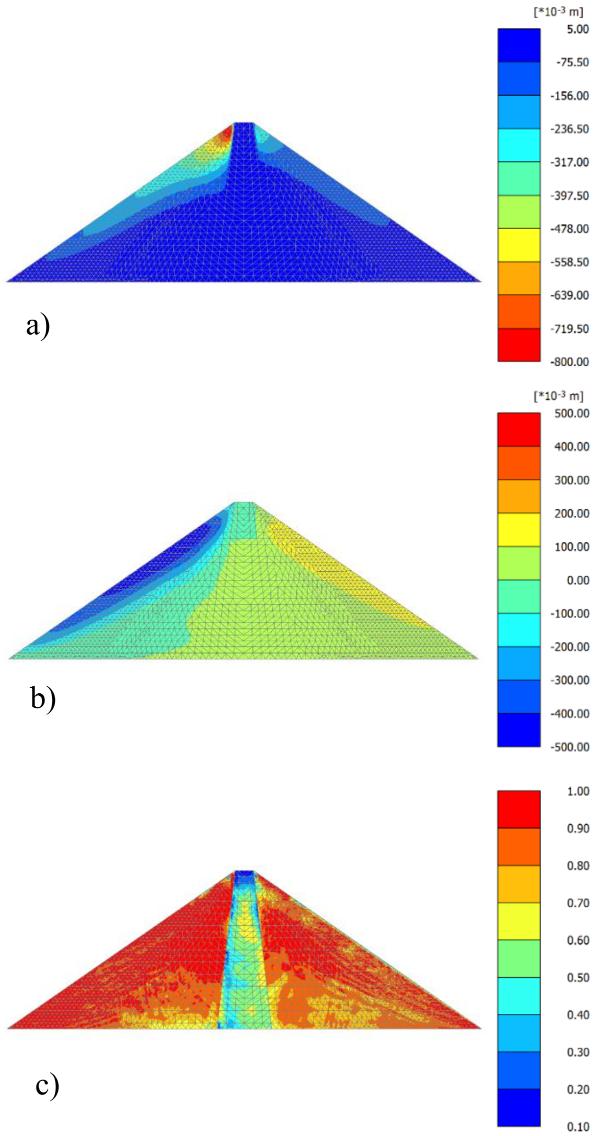

Fig. 8. Contour lines of the (a) vertical and (b) horizontal displacements at end of the shaking and of (c) relative shear stress at time instant of the maximum acceleration amplitude.

The stress state within the embankment was evaluated using the relative shear stress τ_{rel}, which is defined as the ratio between the radius of the current Mohr circle and that of a Mohr circle with the same centre but tangent to the failure envelope; it represents the degree of shear strength mobilisation and is $\tau_{rel} = 1$ when the available shear strength is reached. Figure 8c shows the contour lines of τ_{rel} calculated at the time of the peak acceleration of input #4. A large part of the upstream slope develops values of τ_{rel} close to unity, suggesting that the displacement patterns shown in Figs. 8a and 8b are mainly due to the transient activation of shear strength during the seismic shaking. The same deformation pattern was also observed in the centrifuge model, but with larger displacements partly ascribed to local movements of the soil underneath the base of some instruments. The response of the FE model under the three less intense seismic inputs was qualitatively similar.

10 CONCLUSIONS

The fully coupled hydro-mechanical approach to the numerical modelling of a zoned earth embankment allowed the correct reproduction of the initial pore water pressure distribution in the embankment, including the unsaturated behaviour of the soils. This ensured the correct initialisation of the model in terms of the effective stress state and hence the initial distribution of mobilised shear strength and stiffness when the seismic actions are applied. Although the FE model underestimated the displacement, requiring recalibration of some model parameters, it was still able to qualitatively reproduce the deformation pattern observed in the centrifuge test at the end of the seismic inputs, with larger displacements developing in the upstream flank.

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