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STATIC AND DYNAMIC ANALYSIS OF LA VILLITA DAM IN MEXICO

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ABSTRACT

This paper presents two-dimensional plane strain static and dynamic finite element analyses of La Villita earth dam in Mexico, which has experienced a number of large earthquakes. Static analyses are employed to simulate the layered construction of the embankment, water impounding and consolidation, whereas dynamic analyses simulate the earthquake events. The static behaviour of the dam is well captured, as evidenced by the predicted and recorded crest settlement. The dynamic behaviour is satisfactorily captured by comparison of the accelerations recorded at both the crest and base of the dam.

Keywords: earth dams, dynamic analysis, earthquake, finite elements, reservoir water

INTRODUCTION

La Villita is a zoned earth dam located in the seismic region of Guerrero in Mexico. It has experienced 6 significant earthquakes between 1975 and 1985 and, although it did not fail, it sustained some permanent displacements, which were recorded along with accelerations at three locations. Because of the available data, a number of researchers have worked on this dam. Moreover, what attracted the researchers' attention was the asymmetry of the acceleration record of the crest of the dam which showed higher values of acceleration in the positive (downstream) direction. Elgamal et al. (1990) used a simple sliding block model in order to investigate the observed acceleration asymmetry, while Elgamal (1992) employed a three-dimensional shear beam method in order to numerically analyse its performance during two earthquakes. Succarieh et al. (1993) used a one-dimensional shear wedge method to analyse the dynamic response of the dam and further utilised the Newmark (1965) sliding block method to compute the permanent displacements.

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Gazetas & Uddin (1994) developed a new procedure for analysing earth dams in which, using pseudo-static slope stability analysis, a potential failure surface is identified with the smallest factor of safety, F_s. Finite element analysis (FE) was then performed with the pre-specified sliding surface behaving as perfectly plastic material (i.e. sliding occurs when the acceleration exceeds the strength) whereas the rest of the dam body behaves in a visco-elastic manner. The above-described method was applied on La Villita dam (Uddin, 1997) in order to investigate the observed response asymmetry. Two points on the two edges of the dam crest were monitored (one inside and one outside the sliding mass) and their response was compared, showing that the point inside the sliding mass presented an asymmetric acceleration response. Finally, Papalou & Bielak (2001) performed 3D numerical (combined FE and shear beam) elastic and elasto-plastic (Papalou & Bielak, 2004) analyses of the dam with the surrounding canyon and explored dam-canyon interaction effects.

In this investigation, new numerical analyses are carried out, first giving emphasis on the rigorous representation of the construction of the dam. Hence static coupled consolidation finite element analyses are performed in order to simulate layered construction, water impounding and operation (consolidation). Subsequently, dynamic analyses are undertaken to investigate the response of the dam during the seismic events.

LA VILLITA DAM

La Villita is a 60m high zoned earth dam in Mexico with a slightly curved crest about 420m long which is founded on an alluvium layer of varying thickness. The dam cross-section is composed of a central clay core of very low permeability, with filters, transitions and outer rockfill shells. Alluvial deposits beneath the clay core were grouted to a depth of 26m below the dam, while there is also a 0.6m thick concrete cut-off wall to control seepage through the alluvium below the dam. Figure 1 shows a cross-section view of the dam. Table 1 lists the known material properties of the dam which were obtained from Elgamal (1992).

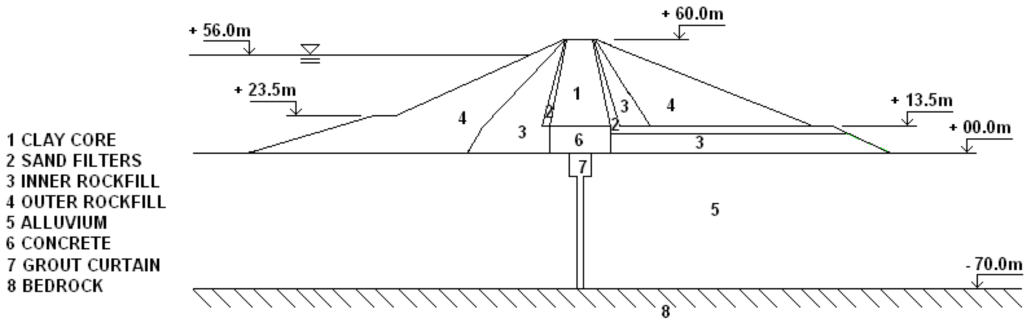

Figure 1: Cross-sectional view of La Villita dam

Table 1: Material properties of La Villita dam

No	Material	Density	Shear Wave Velocity	Cohesion	Angle of shearing
		ρ	V_s	c’	φ’
		Kg/m^3	m/s	kPa	deg
1	Clay core	2000	343	68	0
2	Sand filters	2180	307	0	35
3	Inner Rockfill	2080	314	0	45
4	Outer Rockfill	2080	280	0	45
5	Alluvium	2080	301	0	35

The dam experienced six major seismic events during the period between 1975 and 1985. Although it did not fail, it sustained some minor deformations. The earthquake motions were recorded by three accelerographs which were installed on the dam soon after the end of the construction. There is one instrument on rock on the right bank and two on the dam body, crest and downstream berm (referred herein as “base”). Acceleration records were obtained in electronic form from the Sociedad Mexicana de Ingenieria Sismica which owns the monitoring instruments. The earthquake events are summarized in Table 2.

Table 2: Summary of the earthquake events

No	Date	Ms	Epic. dist	a_{max} of rock	a_{max} of crest
EQ1	11/10/1975	4.5	52km	0.07g	0.36g
EQ2	15/11/1975	5.9	10km	0.04g	0.21g
EQ3	14/3/1979	7.6	121km	0.02g	0.40g
EQ4	25/10/1981	7.3	31km	0.09g	0.43g
EQ5	19/11/1985	8.1	58km	0.12g	0.76g
EQ6	21/11/1985	7.5	61km	0.04g	0.21g

Not all three acceleration records in all three global directions are available. Some of the records are incomplete, containing only a small part of the actual record. Elgamal (1992) states that due to instrument malfunction, only the bedrock records of 15 November 1975 (EQ2) and 19 November 1985 (EQ5) are useful for numerical analysis. Besides, the orientation of the instruments is convenient, as the two horizontal components are in the upstream-downstream (UD) and longitudinal (L) direction of the embankment. Therefore, there is no need for re-orientation of the recorded motion (Elgamal, 1992).

NUMERICAL MODEL

Finite element (FE) analyses, employing the Imperial College Finite Element Program (ICFEP) (Potts & Zdravkovic, 1999), were performed to analyse the response of the dam. The FE mesh shown in Figure 2 was used in all the analyses. It consists of 6254 eight-noded isoparametric quadrilateral elements and 19143 nodes. Elements belonging to consolidating materials (clay core and riverbed alluvium) have also pore water pressure degrees of freedom in corner nodes. The maximum element size (4m) was chosen to be smaller than 1/5 of the smaller wavelength (lowest shear wave velocity, $V_{s\ min}$ over highest frequency, f_{max} of the input wave). The bottom boundary of the mesh was placed at the interface between the foundation alluvium and the bedrock, while the lateral boundaries were placed sufficiently far, so that interaction between them and the dam is avoided.

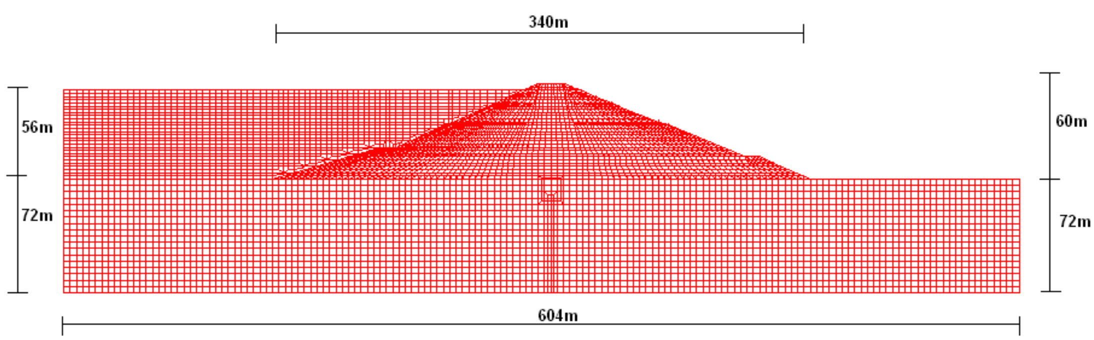

Figure 2: FE mesh of La Villita dam

The constitutive model used in all the analyses is a cyclic nonlinear model, which adopts a logarithmic function to describe the backbone curve (Puzrin & Shiran, 2000; Taborda et al, 2010; Taborda, 2011) coupled with a Mohr-Coulomb failure criterion. The logarithmic relation (Equation 1) dictates the degradation of shear stiffness, G_{max}, and the increase of damping, ξ, with cyclic shear strain, γ.

$$J^* = E_d^* G_{max} \left\{1 - \alpha \left[\ln\left(1 + \frac{E_d^* G_{max}}{J_L}\right)\right]^R\right\} \quad \textbf{Eq. 1}$$

where J* and E_d* are the three-dimensional stress and strain invariants respectively, G_{max} is the maximum shear stiffness, whereas α, J_L and R are model parameters (see also Taborda (2011)).

Due to lack of experimental data, the cyclic nonlinear model (CNL) was calibrated on empirical relations. The Vucetic & Dobry (1991) curves were used for the clay core, whereas the curves of Seed et al (1986) were used for the rest of the materials. Table 3 lists the results of the calibration which are shown in Figures 3 and 4. The reduction of damping in the ξ-γ plot is due to the adoption of a G_{min}.

Table 3: Results of the calibration of the CNL model

Material	E_{dL}	J_L	**c**
Clay core	0.007	290	1
Coarse materials	0.0015	65	1

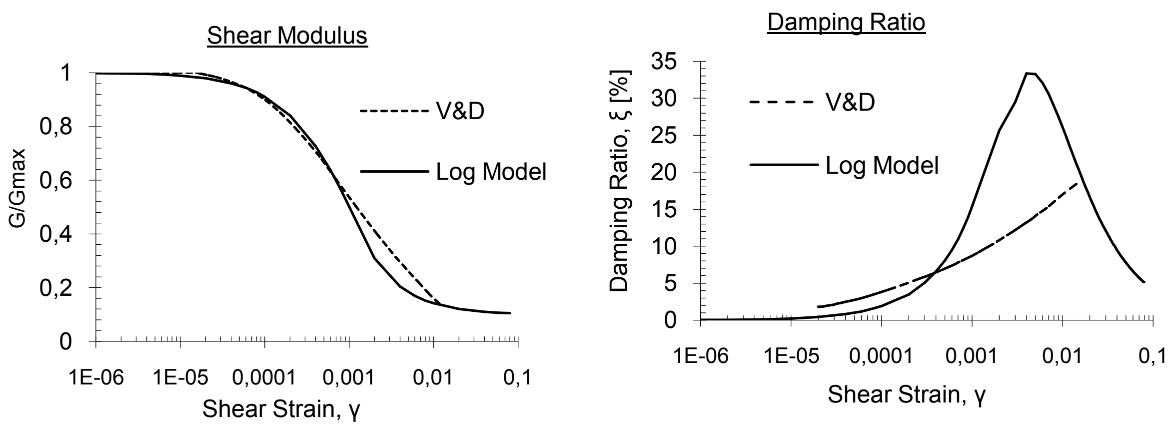

Figure 3: Calibration of the CNL model for the clay core

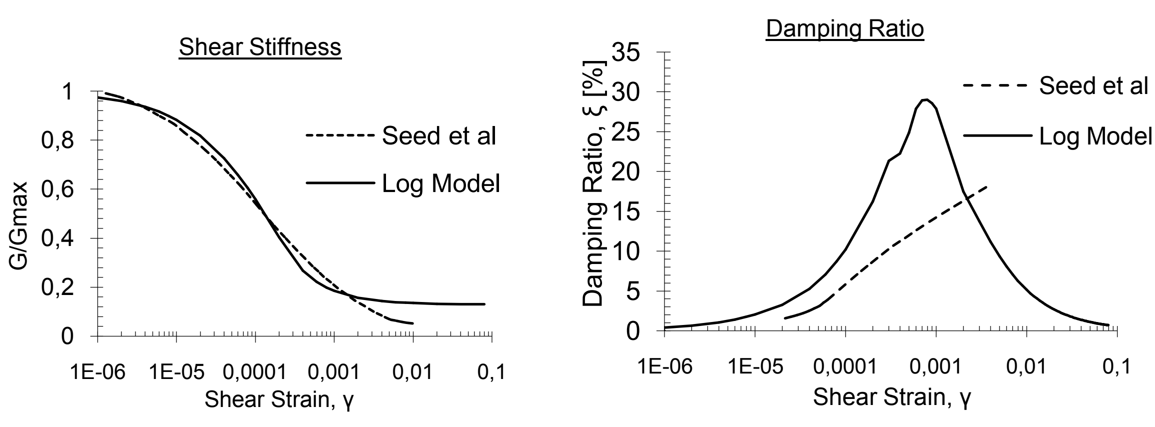

Figure 4: Calibration of the CNL for the coarse materials

STATIC ANALYSIS

Two-dimensional static coupled-consolidation analyses were performed in order to model the history of the dam prior to the earthquake events. First, layered construction is modelled over one year (1967), followed by one year of consolidation (1968). Then, water impoundment is simulated over six months (first six months of 1969 followed by another long period of consolidation (6.5 years: mid 1969 - late 1975) before the first seismic event. For the boundary conditions (BCs), in all the static analyses, zero horizontal and vertical displacements were prescribed along the bottom boundary, whereas zero horizontal displacement and vertical force were prescribed on the lateral boundaries. As far as the hydraulic BCs are concerned, zero change in pore water pressure (PWP) was specified along the lateral boundaries, apart from the water impounding stage, during which PWP increments were prescribed on the upstream boundary, according to the new hydrostatic distribution. The precipitation BC was prescribed on both sides of the clay core. This allowed water flow from the core outwards with zero PWP specified on the boundary whenever the PWP was higher in the core and also zero flow in the core whenever the PWP was higher out of the core (i.e. at places were suction existed in the core).

Water impoundment was modelled in two ways: (a) by applying a hydrostatic boundary stress (BS) on the upstream face and alluvium (i.e. the water finite elements as shown in Figure 2 are not activated and not used) and (b) by modelling the reservoir water with finite elements (WFE). In the latter case, the water elements were assigned the bulk modulus of water, K_w=2.2 10^6 kPa and a very small shear modulus, G_w, for numerical stability. Rayleigh damping was specified in the water corresponding to 20%.

Figure 5 shows the vertical displacement of the crest using both approaches for modelling the reservoir water, BS and WFE. Figure 6 shows the pore water pressure distribution in the dam after water impoundment following the WFE approach. There is a tension-positive convention in ICFEP, and therefore the negative values correspond to compressive pore water pressure, whereas the positive values correspond to tensile pressure (i.e. suction). There is a hydrostatic distribution in the upstream part of the dam following the reservoir impounding and the pore pressure drops quickly within the core. It is clearly shown that the very small permeability of the clay core significantly minimizes the seepage. This confirms that the static part of the analysis (construction and impounding) is satisfactorily captured with the appropriate stress conditions necessary for the subsequent dynamic analysis. The pore water pressure distribution for the analyses with the BS approach is not presented because it is very similar. However, by comparing the predicted and recorded crest settlements from Figure 5 it may be observed that the WFE approach can better predict the crest settlements during water impounding and subsequent consolidation than the BS approach.

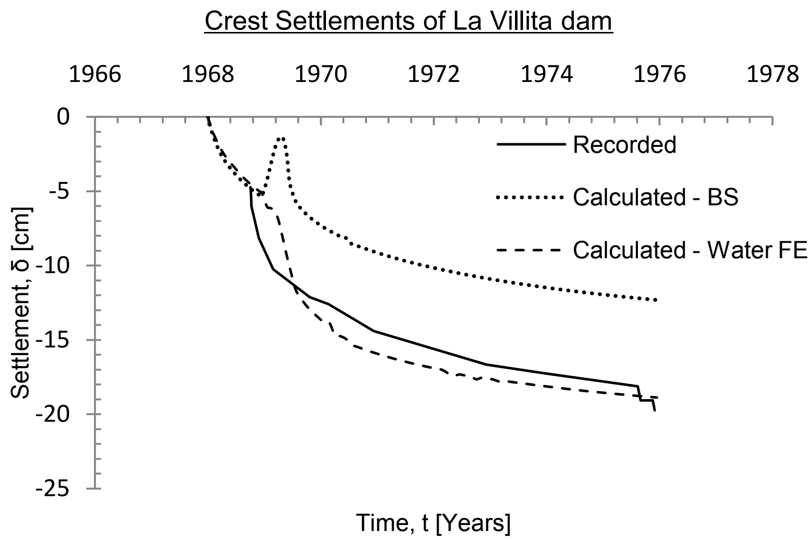

Figure 5: Observed and predicted displacement of the dam crest

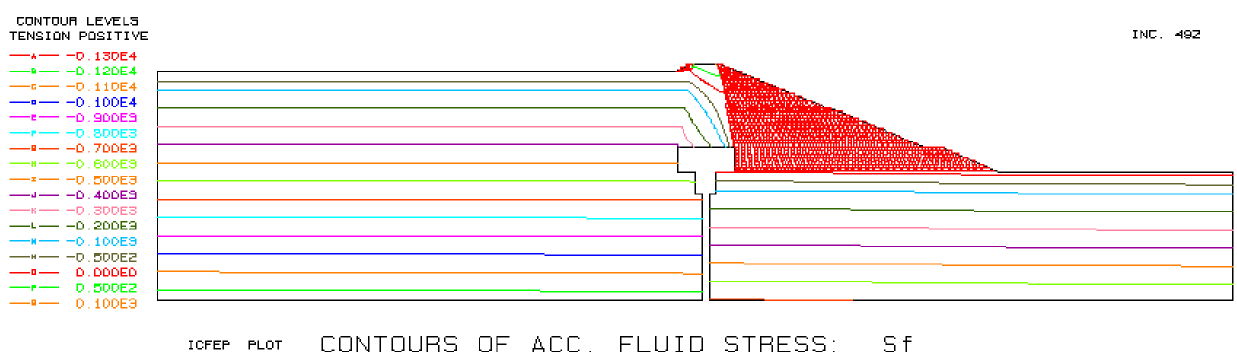

Figure 6: Pore water pressure distribution after water impoundment for the analysis with the WFE approach. All values are in kPa. Negative values correspond to compressive pore pressure, whereas positive values correspond to tensile pressure (i.e. suction).

DYNAMIC ANALYSIS

After the static analyses finished and the appropriate stress conditions in the dam and foundation soil were obtained, dynamic analyses were carried out to investigate the seismic performance of the dam. The rock records are used as input to the analyses. As it was mentioned earlier, Elgamal (1992) states that only EQ2 and EQ5 records are useful for numerical analysis, although only a part of EQ2 is available. After some attempts to use other records, it was similarly concluded that only these two events could be investigated. Therefore, dynamic analyses were performed only for EQ2 and EQ5. In both events the reservoir water pressures were modelled in the two ways mentioned earlier in the static analysis, i.e. the BS and WFE approaches.

The BCs along the bottom boundary were fixity in the vertical direction and prescribed values of acceleration in the horizontal. The lateral BCs on the alluvium were prescribed tied-degrees-of-freedom (TDOF) in both directions and zero vertical displacement on the reservoir. The time integration scheme adopted is the generalised-α algorithm of Chung & Hulbert (1993) which is able to use numerical damping and selectively filter the high frequency components (Kontoe, 2006). The algorithmic parameters used correspond to spectral radius at infinity, ρ_∞=0.3 (Kontoe et al., 2008).

Figures 7-9 show the recorded and calculated accelerations for both earthquakes (base and crest for EQ2 and crest for EQ5) using the WFE approach. A better match is perhaps observed for the base record of EQ2. It is worth mentioning that both approaches to model the reservoir, BS and WFE, yield similar results for both earthquakes. The calculated accelerations from the dynamic analysis generally show a good agreement with the recorded data, although the calculated values seem to be generally smaller. However, the high peaks of the response at the crest which have been previously attributed to a localised failure (Elgamal et al., 1990, Gazetas & Uddin, 1997) were still not obtained. No failure was predicted in the dam during the seismic events in this study, and this is believed to be the reason that high peaks have not been obtained.

Furthermore, Figures 10-12 show the response spectra of the predicted and recorded accelerations at the crest and base for EQ2 and EQ5. It is clear that the recorded spectral accelerations are generally larger than the predicted ones. The higher ordinates of the spectral acceleration for the recorded motion especially for EQ5 are believed to be originated from the high peaks found in the recorded acceleration motion and which are not included in the calculated accelerations. This is not surprising as the response spectra include only the peak values of the acceleration response for different fundamental periods. Therefore, the difference between the recorded and predicted spectra is due to the inability of the model to predict this localised failure and hence the asymmetric response. That is the reason why the difference is larger in the spectra of the crest records.

Moreover, it may be observed from these spectra that there is no great difference between the results of the two approaches as they both give similar results. Again, the spectra of the record of EQ2 at the base of the dam seem to have the best match with the spectrum of the recorded accelerations.

Figure 13 shows the deformed mesh after the end of EQ5. It may be observed from Figure 13 that no major failure seems to have taken place. Therefore, as failure of the dam is not predicted, asymmetry in the response is not expected to be obtained. Again, this is in agreement to the conclusion of the previous researchers (Elgamal et al., 1990, Gazetas & Uddin, 1997) that only a localised failure occurred and not a global failure of the dam.

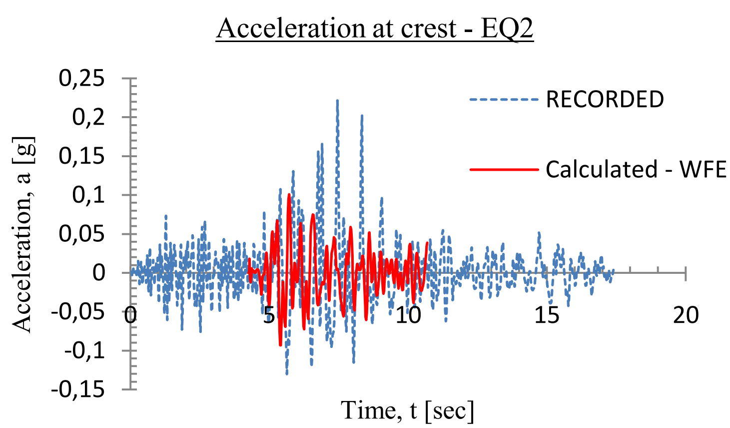

Figure 7: Recorded and calculated (WFE) accelerations at crest for EQ2

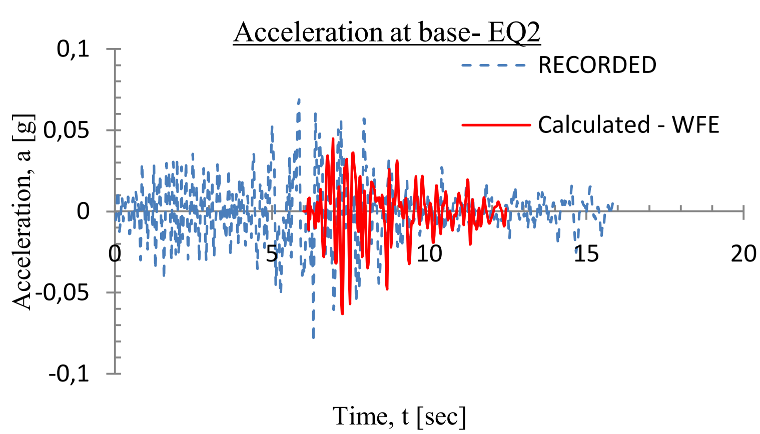

Figure 8: Recorded and calculated (WFE) accelerations at base for EQ2

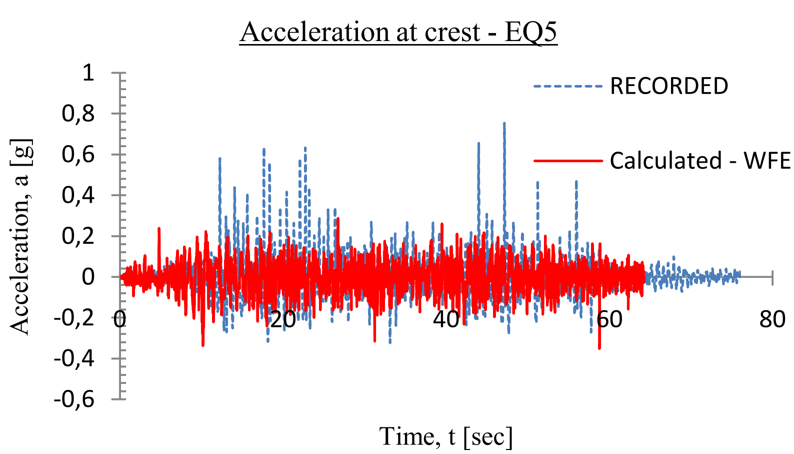

Figure 9: Recorded and calculated (WFE) accelerations at crest for EQ5

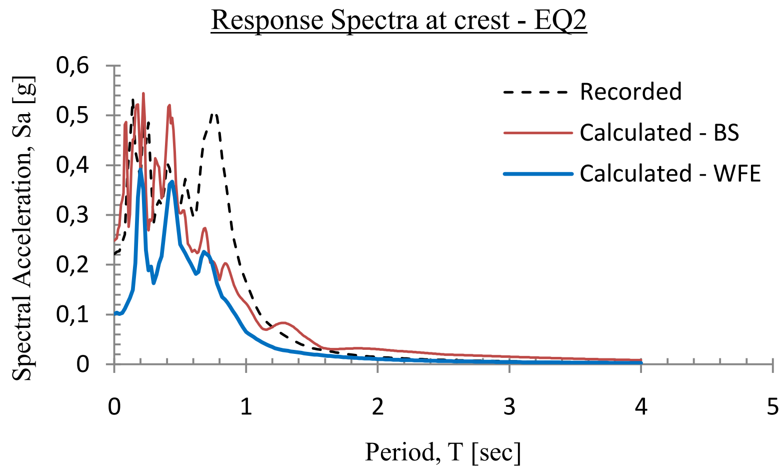

Figure 10: Response spectra of recorded and calculated (both BS & WFE) accelerations at crest for EQ2

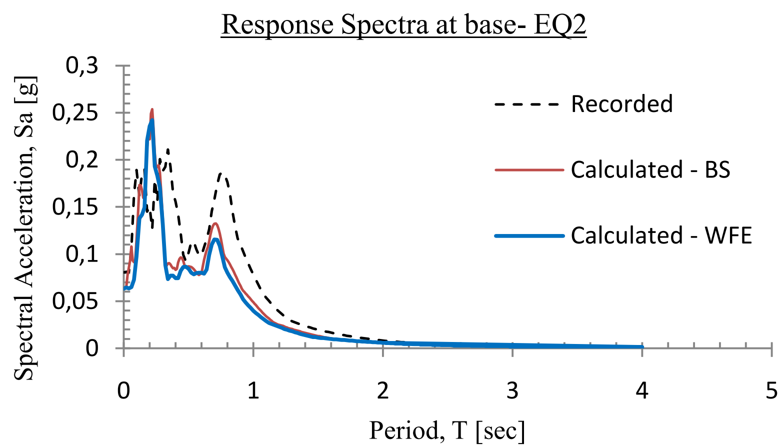

Figure 11: Response spectra of recorded and calculated (both BS & WFE) accelerations at base for EQ2

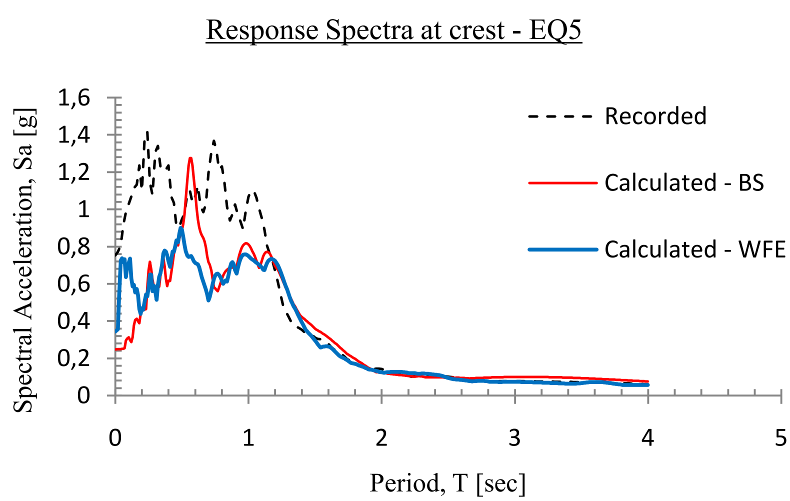

Figure 12: Response spectra of recorded and calculated (both BS & WFE) accelerations at crest for EQ5

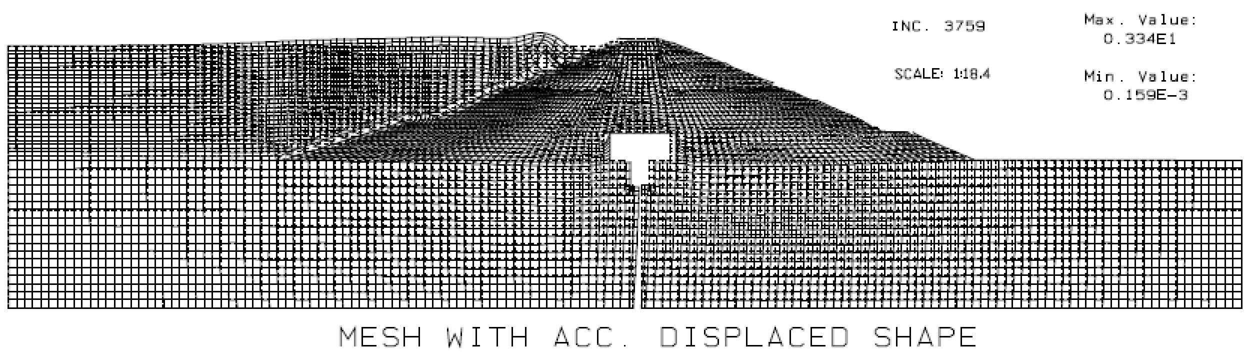

Figure 13: Deformed mesh after the end of EQ5 (Legend values are in meters)

CONCLUSION

La Villita is an earth dam in Mexico which has experienced a number of significant earthquakes during the period 1975-1985. Displacements and accelerations recorded during earthquakes are available, making the dam a well-documented case. In this study, static coupled consolidation and dynamic nonlinear finite element analyses have been performed in order to investigate the performance of the dam during earthquakes. The analyses show that during the static stage a good prediction was obtained for the crest settlements while during the dynamic stage, a good prediction was obtained for the crest and base accelerations. No major failure in the dam during the seismic events has been predicted, as evidenced by the obtained crest accelerations and deformed mesh. Moreover, when the reservoir water is discretised with finite elements rather than with an applied boundary stress, a better prediction of the measured crest displacements is obtained for the static analyses. However, the predicted crest and base accelerations do not differ significantly for both approaches in the dynamic part of the analysis.

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