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Dynamic analysis of Las Cuevas dam

P.S. Sêco e Pinto, J.P. Bilé Serra, R.J.C. Câmara & S.B. Martins de Oliveira
Laboratório Nacional de Engenharia Civil (LNEC), Lisbon, Portugal

ABSTRACT: The dynamic analysis of Las Cuevas dam with 100m of height, located in Venezuela in a zone of high seismicity is presented in this paper. Laboratory tests for the determination of static and dynamic mechanical properties of the material are described. The dynamic analysis has made possible the identification of hazard scenarios and particularly the evaluation of stability and residual deformation of the dam. As the dam is located in a narrow valley a comparison between two-dimensional and three-dimensional analysis was done.

1 INTRODUCTION

Las Cuevas dam is an earthfill embankment to be constructed in Venezuela, on the Doradas river, in a zone of very high seismicity. The dam is 100m high, 260m long at the crest, the total embankment volume will be 4.5×10^6 m^3 and will impound a reservoir of 1.2×10^9 m^3. The cross section is shown in Figure 1 and the upstream slope will be 1:1.8 and the downstream slope 1:1.8. The dam has a sloped narrow core of clay material and two shells of silty-sand material. A chimney-filter located downstream of the core is composed of sand.

The bedrock is composed from a sandstone with high permeability and for this reason an extensive foundation treatment was recommended.

2 LABORATORY TESTS

Laboratory tests to identify the characteristics of the materials were done. A summary of general soil properties is presented in Table 1.

The non-linear parameters for hyperbolic model were obtained by unconsolidated-undrained triaxial tests and

Table 1. Summary of identification tests.

Material type	Proctor test		Atterberg Limits			Unified	Borrow Pit
	Unit weight of dry soil (kN/m^3)	Optimum water content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Classification	
Core	16.3	22.1	71	37	34	OH	El Porvenir
Core	17.2	20.0	57	31	26	OL	El Porvenir
Core	18.5	16.0	41	22	19	CL	El Porvenir
Shell	17.7	10.8	---	---	---	SM	ICOS 1

Table 2 presents a summary of these values (Sêco e Pinto and Mateus da Silva, 1989a).

The maximum shear modulus $G_{máx}$ in kPa for the clay core materials was obtained by the equation (Hardin and Drnevich, 1972):

$$G_{max} = 3230 \frac{(2.97-e)^2}{1+e} (OCR)^k (\sigma'_m)^{0.5} \quad (1)$$

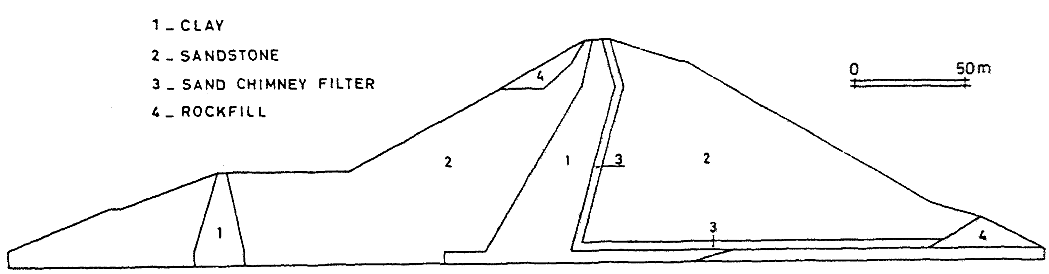

Figure 1. Las Cuevas dam cross-section.

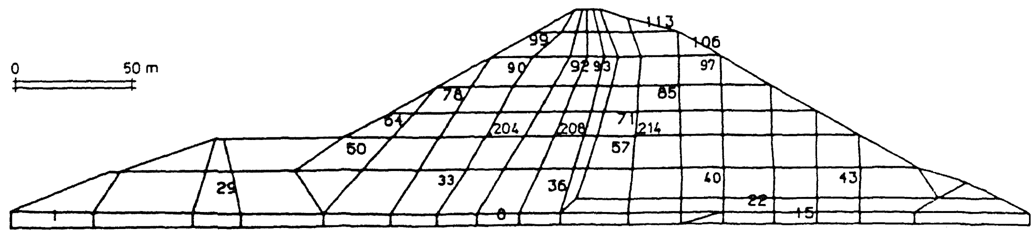

Figure 2. 2D finite element mesh.

Table 2. Summary of hyperbolic parameters for Las Cuevas dam materials.

Soil type Parameter	Symbol	Clay Core	Shell material Dry	Shell material Wet	Filter	Rockfill Dry	Rockfill Wet
Cohesion (kPa)	c	250	110	30	0.0	0	0
Friction angle at σ_3 = 1 atm (degrees)	ϕ	20,5	35	35	51.0	51.0	49.0
Reduction in ϕ per log cycle (degrees)	$\Delta\phi$	0	0	0	0.0	5.0	6.0
Modulus number	K	840	770	468	690	720	670
Unloading-Reloading Modulus No.	Kur	4200	3900	1404	2070	3600	3350
Failure Ratio	R_f	0.78	0.725	0.772	0.590	0.63	0.62
Modulus Exponent	n	0.17	0.441	0.668	0.450	0.55	0.49
Poisson´s Ratio Parameters	G	0.36	0.43	0.47	0.14	0.40	0.3
	F	0.16	0.15	0.23	0.10	0.14	0.12
	d	9.20	9.80	13.80	27.30	1.01	0.94
Threshold Confin. pressure (kPa)	σ_{3t}	---	350	---	---	---	---
Coef. of Isotropic collap. (kPa^{-1})	B	---	2.25×10^6	---	---	---	---

where e is the void ratio, σ_m' is the mean effective stress in kPa, OCR is the overconsolidation ratio and k is a constant depending on the plasticity index.

For the shell material a similar relationship developed by Hardin and Black (1968) was used:

$$G_{max}=3230\frac{(2.97-e)^2}{1+e}(\sigma_m')^{0,5} \qquad (2)$$

The variation of the shear modulus and damping ratio with the shear strain was obtained by cyclic simple shear tests (Sêco e Pinto et al., 1990a). The ratio of the shear modulus at strain γ to the maximum shear modulus G_{max} and the variation of damping ratio with strain γ were compared with the curves proposed by Seed and Idriss (1970). As the obtained data have shown good agreement for the strain interval 10^{-2} to 10% the laboratory results were extrapolated for the strain interval 10^{-4} to 10^{-2}%.

3 DYNAMIC ANALYSIS

The dynamic analysis for Las Cuevas dam was based on procedures proposed by Seed (1979) and has involved the following steps:

i. Determination of the preearthquakes stresses in the embankment dam.
ii. Estimation of the time-history of acceleration that might develop in the rock foundation of the dam for each earthquake under consideration.
iii. Determination of the embankment's response to the earthquakes, which includes the computation of induced shear stress time histories.
iv. Comparison of the cyclic shear stresses required to cause selected magnitudes of deformation (determined from dynamic strength tests) with the equivalent uniform shear stresses induced by the earthquake motions.
v. Evaluation of the overall strain potentials and deformations.

In the following these steps will be analyzed in detail.

3.1 Determination of preearthquakes stresses

Determination of the preearthquakes stresses in the embankment dam has assumed that a steady-state seepage condition had developed within the embankment (Sêco e Pinto and Mateus da Silva, 1989b). The construction stage was simulated in 9 layers and the reservoir filling in five steps. A mesh with 113 isoparametric 8 and 6 node elements and 368 nodal points is presented in Figure 2.

The shear stress distribution is shown in Figure 3.

3.2 Selection of design earthquakes

Las Cuevas Dam is located close to active faults of Caparo and Bocono where generation of very strong motions were recorded.

The characteristics of the main sources of earthquakes are synthesized in Table 3.

3.3 Determination of seismic response

The equation of motion taking damping forces into

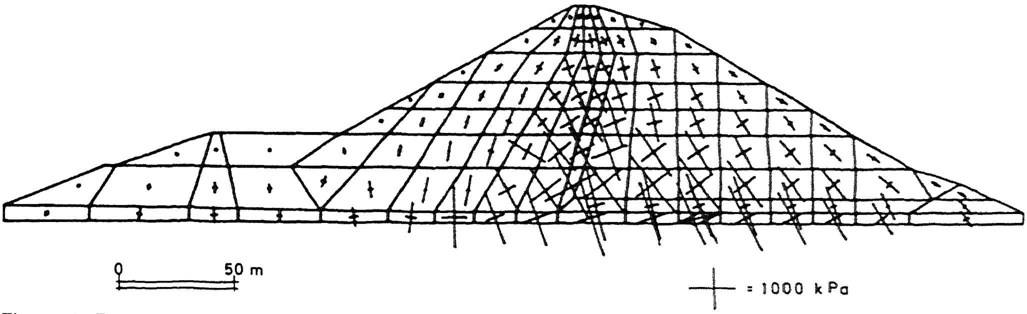

Figure 3. Preearthquake stresses.

Table 3. Selection of design earthquakes

	Near Source	Far Source
Period of life (years)	100	100
Probability of exceeding the ground acceleration (%)	10	10
Ground acceleration (gal)	406	274
Duration of earthquake (seg)	21	41

account and using the finite element method weak formulation can be written

$$M\ddot{q}_t(t)+iDq_t(t)+Kq_t(t)=0 \qquad (3)$$

where $q_t(t)$ represents the nodal displacement time histories and dots mean time second derivation, M is the mass matrix, K is the stiffness matrix, D is the linear hysteretic damping matrix and i is the imaginary unit.

The complex nature of these equations makes it convenient to solve them in the frequency domain.

A computer finite element program DINAPLANO was developed in Fortran 77 at LNEC (Bilé Serra, 1989) and isoparametric finite elements with six and eight nodes and Lagrangean elements with six nodes were implemented. The latter are intended to model mechanical discontinuities by thin interface elements.

Input motions can be incorporated in either one of the following ways:

i. Base horizontal and vertical acceleration time histories.

ii. Base horizontal and vertical acceleration power spectra.

An iterative procedure is considered to adjust soil properties to shear strain level achieved in each finite element associated with a convergence criterion. Linear elastic stress-strain relationships are assumed in each iteration. Eigenfrequencies and eigenshapes are obtained by the so-called subspace iteration method (Bathé and Wilson, 1976).

If seismic input is defined as in (i), acceleration time histories Fourier spectra are obtained by FFT procedures (Cooley et al.,1969) prior to the determination of transfer functions matrices. Inverse FFT is performed on obtained response Fourier spectra in order to obtain response time histories.

The results of the dynamic analysis include the time histories of acceleration at each finite element mesh node and the time histories of shear stress and shear strain in each finite element.

Only some results will be presented here and a detailed analysis can be found elsewhere (Sêco e Pinto et al., 1990b).

The first vibration mode is shown in Figure 4.

Figure 5 presents the maximum acceleration response in both horizontal and vertical directions for selected points located in the upstream shell, core and downstream shell for the near earthquake.

The shear stress time histories for six selected elements located at the upstream shell, core and downstream shell for the near earthquake are shown in Figure 6.

3.4 Deformation evaluation and seismic stability

The values of the equivalent cyclic stress ratio calculated from the results of the dynamic analyses were used to compute the corresponding values of the shear strain potential. Figure 7 shows a comparison between cyclic shear stresses required to cause potential strain of 2.5%, 5% and 10% obtained from cyclic simple shear tests and equivalent uniform shear stresses calculated using DINAPLANO for elements located in row A-A.

The shear strain potential distribution is presented in Figure 8. It is clear that relatively high values of shear strain potential are concentrated near the surface of the upstream shell.

It was proposed a gravel layer at the surface in the upper part of the upstream shell because it is assumed that the gravel zone is sufficiently pervious to dissipate pore water pressures rapidly enough to prevent any significant build-up. Based on past experience it is known that if excess pore water pressures are not allowed to develop because of high dissipation rate, dense soils in general exhibit negligible permanent strains and shear strength reduction.

The final decision will depend of the involved cost.

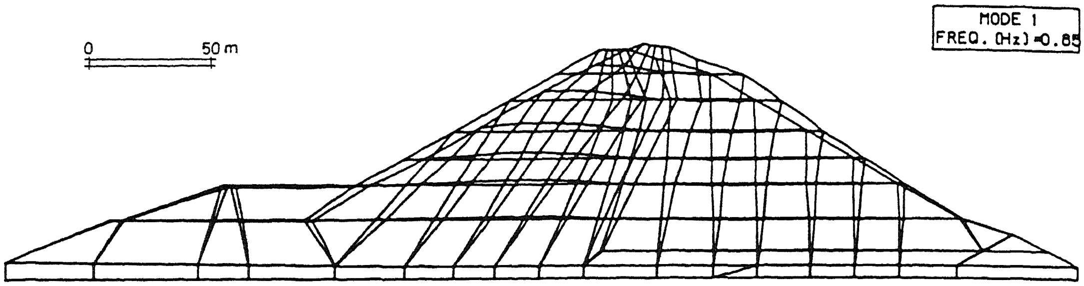

Figure 4. 2D first vibration mode

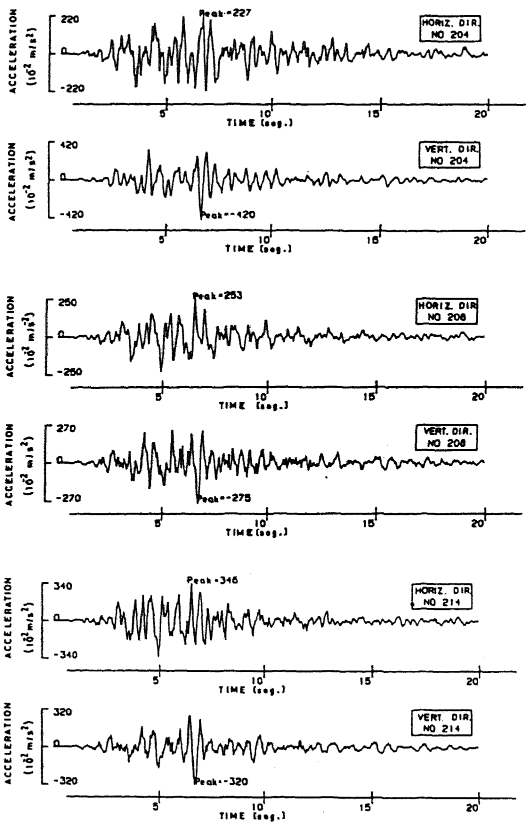

Figure 5. Maximum acceleration responses.

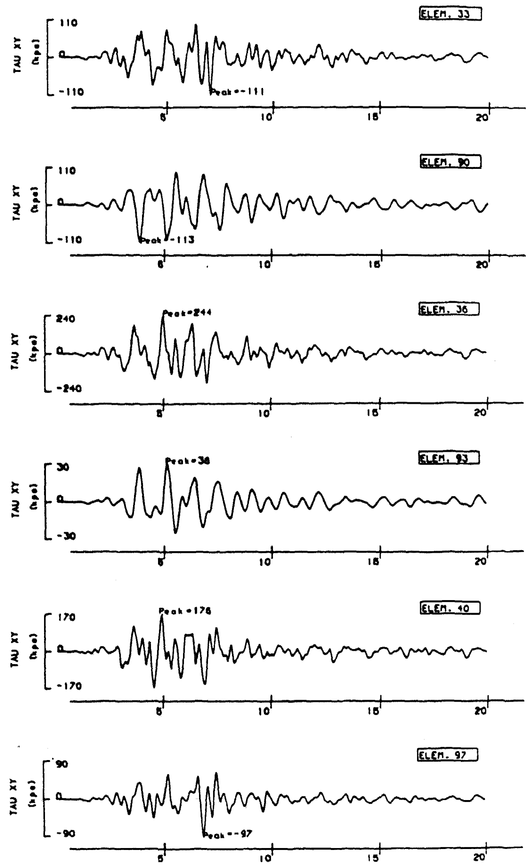

Figure 6. Shear stress time histories.

4 THREE DIMENSIONAL ANALYSIS

Since, for dams in narrow canyons, the response of the structure is of a three-dimensional nature and as the crest length to height ratio L/H=2.6 a three dimensional analysis was performed.

The computer program DYSE52 developed at LNEC (Câmara, 1989) was used and Figure 9 shows the finite element model. The structure was discretized in 555 isoparametric finite element (8 mode cubic element and 6 mode prismatic element) with a total of 648 modal points.

The materials that incorporate the dam were considered continuous and isotropic with the shear modulus and Poisson ratio obtained from the last iteration of the 2D dynamic analysis. A damping viscous coefficient of 10% was assumed.

The first hundred vibration modes were determined with fundamental frequencies between 1Hz and 2Hz.

The first vibration mode is shown in Figure 10.

A comparison between natural frequencies computed from 2D and 3D analyses has shown that for 3D

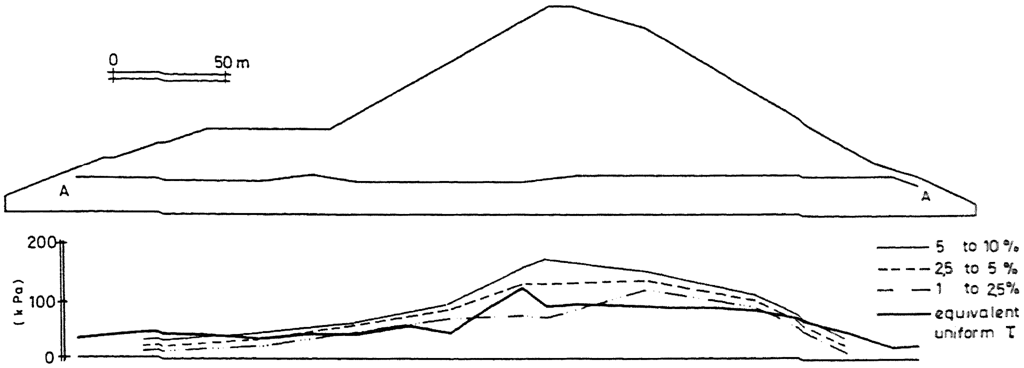

Figure 7. Comparison between laboratory shear stresses and equivalent uniform shear stresses.

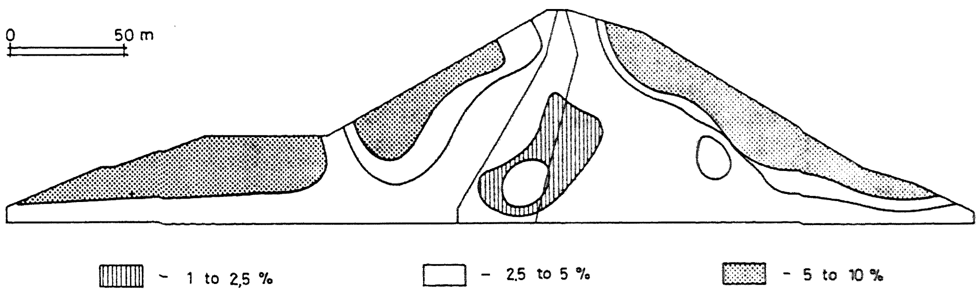

Figure 8. Strain potential distribution.

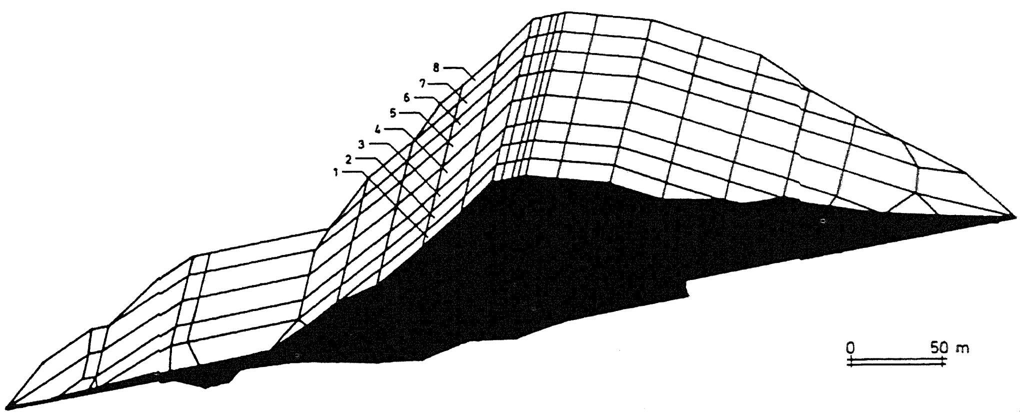

Figure 9. 3D finite element mesh.

analysis the fundamental natural frequencies were 25% higher than those computed from a plane strain analysis.

It was found that a plane strain analysis of the maximum section gave values for the shear stresses that were 50% higher than those computed from a 3D analysis of the dam.

5 CONCLUSIONS

From the present study the following conclusions can be pointed out:

- A shear strain potential distribution was obtained from 2D finite element analysis.
- The gravel zone recommended for the upper part of the upstream shell will reduce the crest deformation but a cost-benefit study is recommended.

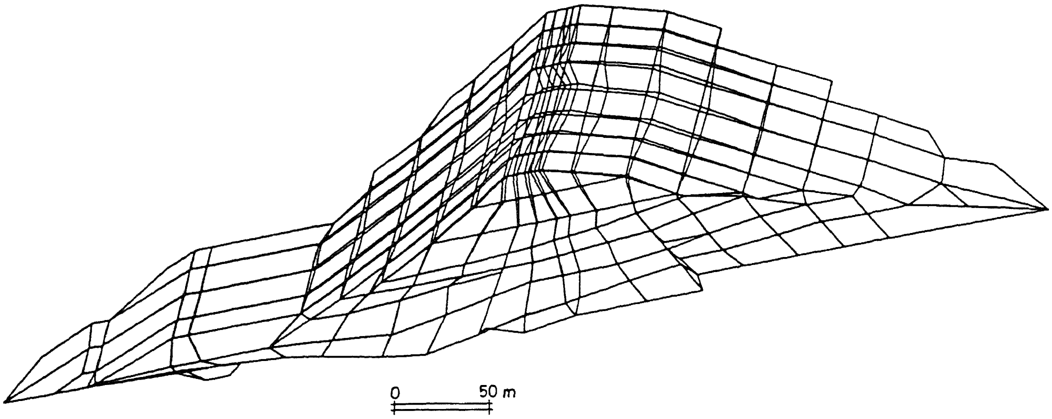

Figure 10. 3D first vibration mode.

- A plane strain analysis of the maximum section of Las Cuevas dam has given values for the shear stresses that were within 50% of those computed from a three-dimensional analysis of the dam.
- The fundamental natural frequencies of the three-dimensional model of the dam were 25% higher than the values obtained from a 2D analysis.
- The difficulties related with the definition of the seismic events and characterization of the materials recommend that the results of dynamic analysis should take into account the existent experience and the behaviour of these structures during earthquakes.

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