Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Finite element seismic analysis of dams including interaction effects Computational model and validation

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ABSTRACT: In this paper a numerical-experimental study of the seismic analysis of double curvature arch dams under linear elastic conditions is presented. Both empty reservoir and full reservoir conditions taking into account fluid-structure interaction effects are considered. The numerical computations obtained via finite element methods are compared with experimental "in situ" tests on an existing dam, and also with results obtained in the experimental analysis of a 1:100 scale model.

1 NUMERICAL MODEL

1.1 Dam analysis

The dynamic analysis of double curvature arch dams is performed using the finite element method. After adequate discretization using 3D solid elements the dynamic equilibrium equations can be written in the standard matrix form (Zienkiewicz 1970)-(Bathe 1982):

$$\mathbf{M}\ddot{\mathbf{a}} + \mathbf{C}\dot{\mathbf{a}} + \mathbf{K}\mathbf{a} = -\mathbf{M}\mathbf{J}a_g \qquad (1)$$

where $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are the well known mass, damping and stiffness matrices, respectively, $\mathbf{a}$, $\dot{\mathbf{a}}$ and $\ddot{\mathbf{a}}$ stand for nodal displacement, velocities and accelerations vectors, respectively, $\mathbf{J}$ is a vector containing ones and zeros and a_g is the ground acceleration at the dam base (Barbat and Miquel 1989).

The dam body is discretized using 20 node isoparametric hexaedral elements. Also 15 node triangular prisms have been used to model more accurately the interaction zones with the foundation.

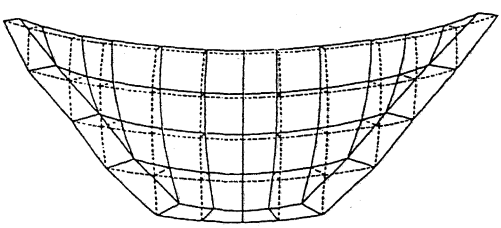

Figure 1 Double curvature arch dam discretized using 3D 20 node and 15 node solid elements.

The inclusion of the foundation in the discretization is not simple due to well know wave radiation effects at the mesh boundaries in the soil zone and special techniques are needed to avoid artificial reflection of waves inside the mesh at these boundaries (Fox and Chopra 1985)-(Miquel et al. 1990). In this work a soil region of a depth equal to the dam height has been modelled with massless 3D solid elements. This has led to good numerical results as it will be shown in a later section.

Eq (1) has been solved using a modal decomposition technique based on the *determinat search method* (Bathe 1982). The maximum response in the dam has been obtained using a response spectrum.

1.2 Analysis of the fluid and coupling effects

The effect of the reservoir water has been taken into account by solving the wave equation

$$\nabla^2 p = \frac{1}{c^2}\frac{\partial^2 p}{\partial t^2} \qquad (2)$$

where p is the water pressure and c is the wave speed.

All the analysis has been carried out in the frecuency domain.

The finite element discretization of the fluid equation leads to (Figure 2) (Fox and Chopra 1985), (Hall and Chopra 1980), (Miquel et al. 1990)

$$\begin{bmatrix} -\omega^2\mathbf{G}_{11} + \mathbf{H}_{11} & -\omega^2\mathbf{G}_{12} + \mathbf{H}_{12} \\ -\omega^2\mathbf{G}_{21} + \mathbf{H}_{21} & -\omega^2\mathbf{G}_{22} + \mathbf{H}_{22} \end{bmatrix} \begin{bmatrix} \mathbf{P}_1 \\ \mathbf{P}_2 \end{bmatrix} + \begin{bmatrix} 0 \\ \int_{s_c} \mathbf{N}\frac{\partial p}{\partial n} dS_c \end{bmatrix} = -\mathbf{S}\ddot{\bar{\mathbf{a}}}^t \qquad (3)$$

where w is the considered frequency, $\mathbf{G}$ and $\mathbf{H}$ the mass and stiffness matrices of the fluid respectively, and S the fluid structure coupling matrix. The variables $\mathbf{P}_1$ denote the pressures of the nodes in the contact fluid-structure region and $\mathbf{P}_2$ the rest of the fluid nodal pressures.

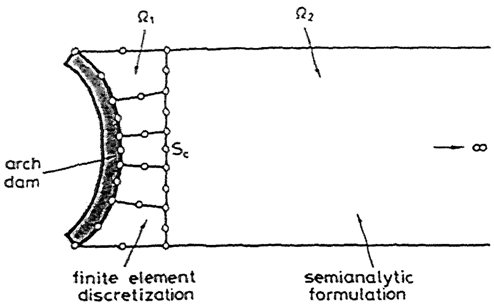

Figure 2 Fluid domain discretization.

Taking into account that for the non contact region Ω_2 *(Figure 2) the following condition must be satisfied*

$$p = P_2 \exp(-kx)$$

and coupling this equation with the discretized decoupled dam equation for each vibration mode, the following equation system is obtained (Fox and Chopra 1985), (Hall and Chopra 1980), (Miquel et al. 1990)

$$\begin{bmatrix} -\omega^2\mathbf{G}_{11} + \mathbf{H}_{11} & (-\omega^2\mathbf{G}_{12} + \mathbf{H}_{12})\boldsymbol{\alpha} & -\omega^2\mathbf{S}\boldsymbol{\Phi} \\ \boldsymbol{\alpha}^t(-\omega^2\mathbf{G}_{21}+ & \boldsymbol{\alpha}^t(-\omega^2\mathbf{G}_{22} + \mathbf{H}_{22})\boldsymbol{\alpha}+ & \mathbf{0} \\ +\mathbf{H}_{21}) & +\boldsymbol{\alpha}^t\boldsymbol{\Psi} & \\ \frac{1}{\rho}Phi^t\mathbf{S}^t & \mathbf{0} & \mathbf{T} \end{bmatrix} \times$$

$$\times \begin{bmatrix} \mathbf{P}_1 \\ \boldsymbol{\gamma} \\ \mathbf{y} \end{bmatrix} = \begin{bmatrix} -\mathbf{S}\mathbf{J}\ddot{a}_g \\ \mathbf{0} \\ -\boldsymbol{\Psi}\mathbf{M}\mathbf{J}\ddot{a}_g \end{bmatrix} \tag{4}$$

where the elements of $\mathbf{T}$ matrix are

$$T_{ij} = -\omega^2 + i\omega C_i + \omega_i^2 \quad \text{if } i = j$$
$$T_{ij} = 0 \quad \text{if } i \neq j$$

In above $\boldsymbol{\alpha}$ are the eigenvalues of S_c (Figure 2), $\boldsymbol{\Psi}$ the eigenvectors of the dam and $\mathbf{y}$ the generalized coordinates of the dam, $\boldsymbol{\gamma}$ the generalized coordinates of the cross section of the fluid and $\boldsymbol{\alpha}^T\boldsymbol{\Psi}$ is a term taking into account the effect of infinite fluid boundaries (Miquel et al. 1990).

By solving the above matrix equation for different values of ω, the peaks in the solution for $\mathbf{y}$ will indicate the resonant frequencies of the dam.

The maximum values of the coupled system using the response spectrum have been obtained with the assumption that the excess pressure measured on the upstream wall is proportional to the radial component of the acceleration at each point in the form

$$\mathbf{F}(\omega) = \mathbf{M}_{eq}\mathbf{A}_r(\omega) \tag{5}$$

Eq.(5) provides the values of the frequency dependent added mass $\mathbf{M}_{eq}$, where and $\mathbf{A}_r(\omega)$ contain the amplitudes of the radial acceleration. This hypotesis has led to very good numerical results (Miquel et al. 1990)-(Prove dinamiche, etc. 1980).

2 NUMERICAL-EXPERIMENTAL ANALYSIS

The previous formulation was used to study Llauset dam located in the nordeast of Spain over the Moralets hydroelectric power plant. This is a 84 height symmetric double curvature arch dam with three centers. The dam is owned by the Spanish hydrolectrical company ENHER. Further information on the geometrical and material properties of Llauset dam can be found in (Fernández-Bollo 1989). The seismic loading considered is that defined by the Spanish Seismic Standard (see Figure 3) with $A_0 = 0,075$ and $T_0 = 0,5s$ (Norma Simorresistente Esapñola).

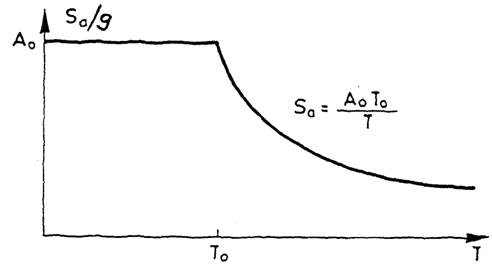

Figure 3 Seismic spectrum considered.

Experimental results obtained in the analysis of a 1:100 scale laboratory model of the same dam carried out at ISMES (Bergamo, Italy) were available for the maximum displacements and stresses and the first natural frequencies for both full and empty reservoir conditions. Also the first natural frequencies for the full reservoir case were experimentaly measured "in situ" by using dynamite explosion tests in the vicinity of the actual dam and by using excentric mass techniques in the dam body (Fernández-Bollo 1989).

2.1 Results for the empty reservoir case

Figures 4 and 5 show the two first vibration modes of the dam together with the discretization used. Table I shows the comparison of the first four natural frequencies values computed with values obtained experimentaly.

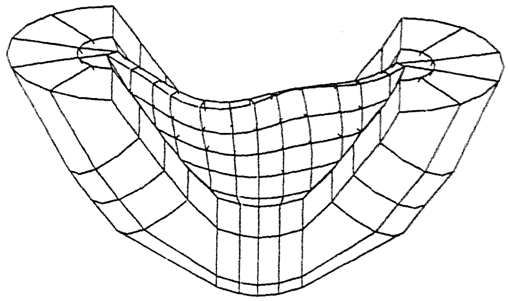

Figure 4 Llauset dam. First modal shape.

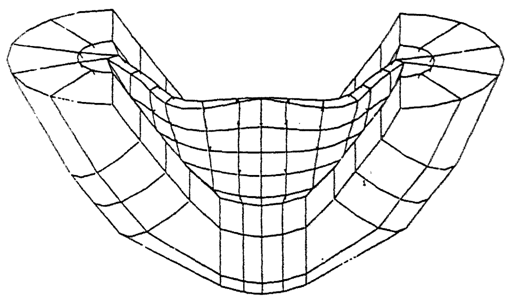

Figure 5 Llauset dam. Second modal shape.

Table I Comparison of experimental and computed first four frequencies for Llauset dam. Empty reservoir case.

Mode	Experimental values (rad/sg)	Computed values (rad/sg)
1	20,357	23,091
2	29,217	28,003
3	31,793	33,724
4	42,097	45,011

Figure 6 shows the deformed shape of the dam under the response spectrum of Figure 3 applied in the direction of the valley.

In Table II the values of the maximum displacements obtained experimentaly and numericaly are compared for different foundation conditions. The agreement of results is highly improved when the foundation effects are included by discretizing a soil zone equal to the dam height (Miquel et al. 1990).

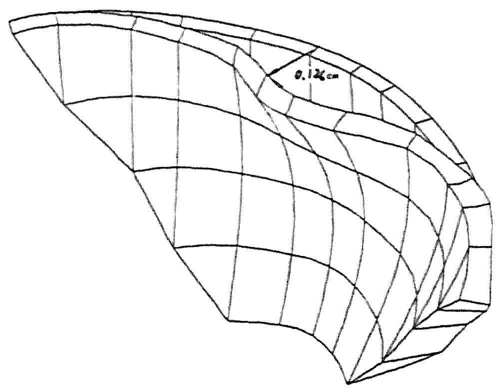

Figure 6 Llauset dam. Deformed shape under response spectrum of Figure 3.

Table II Comparison of experimental and computed displacements. Response spectrum acting in the direction of the valley.

	Experimental results	Computed results	
		Rigid Foundation	Discretised foundation equal to dam height
Displacement along the valley	0.126cm	0.175cm	0.115cm
Displacement perpendicular to the valley	0.089cm	0.035cm	0.077cm

Table III shows the same comparisons when the response spectrum acts in the direction perpendicular to the valley.

Table III Empty reservoir case. Comparison of experimental and numerical displacements. Response spectrum acting perpendicular to the valley.

	Experimental results	Computed results	
		Rigid Foundation	Discretised foundation equal to dam height
Displacement along the valley	0.216cm	0.118cm	0.180cm
Displacement perpendicular to the valley	0.132cm	0.072cm	0.105cm

Tables IV and V show the same type of comparison for the maximum horizontal and vertical stresses when the response spectrum acts along and perpendicular to the valley, respectively.

Table IV Empty reservoir case. Comparison of experimental and computed values of maximum vertical and horizontal stresses. Spectrum acting along the valley direction.

	Experimental results	Computed results	
		Rigid Foundation	Discretised foundation equal to dam height
Maximum vertical stress	1,51 Kg/cm²	1,81 Kg/cm²	1,64 Kg/cm²
Maximum horizontal stress	1,70 Kg/cm²	1,96 Kg/cm²	1,85 Kg/cm²

Table V Empty reservoir case. Comparison of experimental and computed values of maximum vertical and horizontal stresses. Spectrum acting perpendicular to the valley direction.

	Experimental results	Computed results	
		Rigid Foundation	Discretized foundation equal to dam height
Maximum vertical stress	1,53 Kg/cm²	1,91 Kg/cm²	1,74 Kg/cm²
Maximum horizontal stress	2,35 Kg/cm²	2,84 Kg/cm²	2,55 Kg/cm²

Table VI Full reservoir case. Natural frequencies for the first four modes. Comparison of computed and experimental values.

Mode	Computed Values	Experimental tests
1	17,1 rdn/s	16,78 rdn/s
2	21,5 rdn/s	23,66 rdn/s
3	26,5 rdn/s	25,7 rdn/s
4	36,5 rdn/s	37,13 rdn/s

Comparison of numerical and experimental results show an excellent agreement with maximum differences in the vicinity of 10%. Also the need for including the effect of a discretized foundation zone equal to the dam height in the computation is demostrated.

Finally, Table VI and VII show the same comparisons for the *full reservoir case.* Note that the "in situ" tests provided a value of the first natural frequency of 16.9 rad/s which again corresponds very well with the computed value.

3 CONCLUSIONS

In this work a finite element numerical model for the dynamic analysis of double curvature arch dams taking into account fluid interaction effects has been presented. The accuracy of the model has been tested on the analysis of a dam for which experimental results are available.

Table VII Full reservoir case. Comparison of computed values of displacements and stresses. Response spectrum acting along the valley direction.

	Experimental Values	Computed Values
Displacement along the valley	0.329 cm	0.2898 cm
Displacement perpendicular to the valley	0.067 cm	0.0651 cm
Maximum horizontal stress	5.8 Kg/cm²	4.494 Kg/cm²
Maximum vertical stress	3.02 Kg/cm²	3.199 Kg/cm²

ACKNOWLEDGEMENTS

The authors are grateful to P.I.E. for the finantial support provided for this research.

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