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A MODULAR APPROACH TO THE THREE-DIMENSIONAL SEISMIC COUPLED ANALYSIS OF ARCH DAMS/RESERVOIR/SOIL

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SUMMARY

A modular and general approach for the three-dimensional analysis of arch dams submitted to earthquake loadings including fluid reservoir and soil interaction is presented in this paper. The analysis is performed in the frequency domain. A particular emphasis is given to domain decomposition with immediate consequence the ability to use the best numerical technique for each domain : the structure, the reservoir and the soil. The arch dam is modelled by thick shell finite elements, the fluid reservoir by fluid boundary elements and the soil by solid boundary elements so that only the interfaces need to be meshed. A reduced basis approach is also undertaken by using the various kinematic modes on the interfaces between the three domains and the subsequent fields in the reservoir and the soil. A very modular computer code has been implemented following these specifications and one application to a real case is finally discussed.

INTRODUCTION

It is well known that the dynamic interaction of an arch dam with its reservoir and soil foundation may considerably modify the structural response in several ways. The influence of wave diffraction and propagation in the soil is also important either from the point of view of soil-structure interaction because the stiffnesses of the dam and the surrounding soil are comparable or soil-reservoir interaction which is also the source of further damping. Finally the so-called local site effect is not to be neglected in the classical geometry of an arch dam and its valley because propagating surface waves may be present. The boundary element technique is well suited to the simplicity of the Helmholtz equation in the reservoir body and the complexity of the reservoir bottom. Apart from the above mentionned special requirements the analysis is developped as far as possible with the exact continuous fields and the discretization process is isolated clearly.

MODELLING ASSUMPTIONS FOR THE SOIL/ RESERVOIR/ STRUCTURE SYSTEM

In this section the necessary equations to describe the dynamic interaction between a structure, a compressible fluid lying above a soil foundation are recalled. The typical geometry of the problem and the notations are presented in Figure 1.

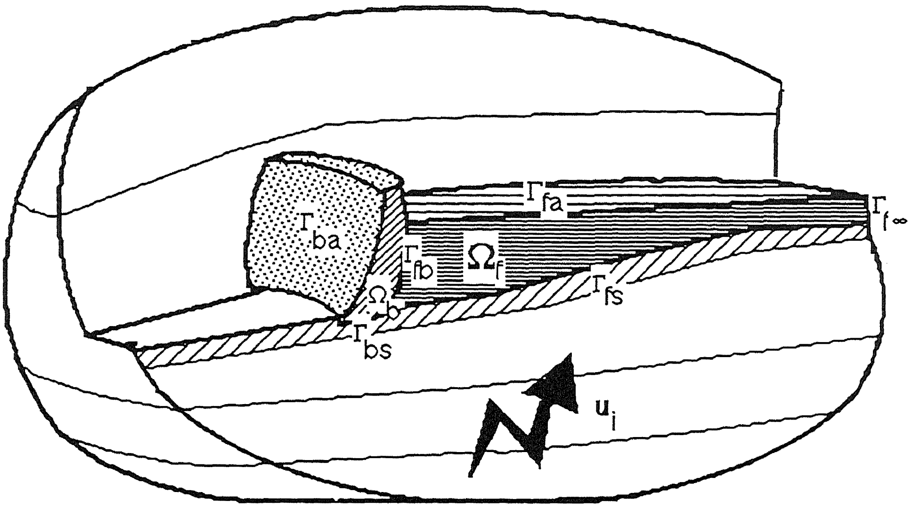

Figure 1: Definition of the geometry of domains and interfaces

The earthquake loading is introduced through the local seismic incident displacement field $\mathbf{u}_i(x,\omega)$ depending on the spatial variable $\mathbf{x}$ and on the circular frequency ω. The soil is assumed to be linear elastic. The following general notations will be used throughout the paper for fields p and q defined over the domain Ω_α, or the boundary Γ_α :

$$(p\,,q\,)_\alpha = \int_{\Omega\alpha} p(x)\, q(x)\ dV, \qquad <p\,,q>_\alpha = \int_{\Gamma\alpha} p(x)\, q(x)\ dS, \tag{1}$$

The generic double index $\alpha\beta$ will be used at the interface $\Gamma_{\alpha\beta}$ between the two domains Ω_α and Ω_β. ρ_α is the specific mass of the material inside the domain Ω_α. $\sigma(\mathbf{v})$ stands for the stress tensor field corresponding to the deformation tensor field $\varepsilon(\mathbf{v})$ and displacement field $\mathbf{v}$. On a boundary with exterior normal vector $\mathbf{n}$, the traction vector $\mathbf{t}$ is given by : $\mathbf{t} = \sigma.\mathbf{n}$.

<u>Elastodynamics of the soil domain Ω_s</u> The soil is modelled by the equations of the elastodynamics in the frequency domain. The total displacement field $\mathbf{u}_s$ in this domain, must satisfy boundary conditions on the free surface Γ_s, displacement and stress vector continuity at the interface between the eventual layers, it must also match the free seismic field of the site at the infinity.

$$\mathbf{u}_s = \mathbf{u}_i \qquad , \text{on } \Gamma_{s\infty} \tag{2}$$

and at the interface with the fluid on Γ_{sf}, the latter gives the pressure :

$$t_s(\mathbf{u}_s) = p\,\mathbf{n}_f \qquad \text{on } \Gamma_{sf}. \tag{3}$$

By definition, an <u>elastodynamic displacement field</u> of the soil will be defined as a field of displacements satisfying the elastodynamic equations in each layer together with the boundary conditions at the free surface, between the layers and the radiation conditions. The elastodynamic displacements in the soil are approximated by the boundary element method with the numerical layered half space Green's function .

Elastodynamics of the structure Let u_b , $\sigma_b(u_b)$, $t_b(u_b)$ respectively be the fields of real displacements, stresses in the structure and the field of the vector stresses on Γ_{bf} or Γ_{bs}. Let v_b be a field of virtual kinematically admissible displacements . The principle of virtual works applied to the structure Ω_b in the frequency domain gives :

$$- \omega^2 (\rho_b u_b , v_b)_b + (\sigma_b(u_b) , \varepsilon(v_b))_b = < t_b(u_b) , v_b>_{bf} + < t_b(u_b) , v_b>_{bs} \quad (4)$$

Up to now and for the sake of a simplified presentation the structure is considered as a three-dimensional solid. Actually thick shell kinematic assumptions are made to take into account the small thickness of the dam with regard to its height. In the present work elastodynamic displacements in the structure defined in a similar way as for the soil are approximated by the finite element method.

The imposed boundary conditions to the displacements of the structure are the following : at the interface with the soil, the displacements of the structure must match the displacements of the soil u_s , while at the interface between the structure and the fluid, the latter imposes the pressure :

$$u_b = u_s , \qquad t_b (u_b) + t_s (u_s) = 0 \text{ on } \Gamma_{bs}, \qquad t_b(u_b) = p\, n_f \quad \text{on } \Gamma_{bf} \quad (5)$$

Acoustics of the fluid The fluid is assumed inviscid and compressible and only small movements are studied. When c is the speed of propagation of the waves in the fluid medium, the pressure field must then satisfy the reduced wave equation in the frequency domain. The boundary conditions imposed to the pressure field are the following :

$$p = 0 \quad \text{on } \Gamma_{fa}, \qquad \partial_{nf}\, p = - \rho_f\, \omega^2 \, (u.n) \qquad \text{on } \Gamma_{bf} \cup \Gamma_{sf}. \quad (6)$$

By definition, an acoustic field of the reservoir, will be a pressure field which satisfies the preceeding equations. In any case in this paper the acoustic pressures are approximated by the boundary element method with the fluid half space Green's function.

REPRESENTATION OF THE SOIL-FLUID-STRUCTURE INTERFACE

Interface kinematic fields In order to solve the problem of the triple soil-fluid-structure interaction , a representation of the displacement field along the interfaces will be assumed, as it is illustrated on the figure underneath :

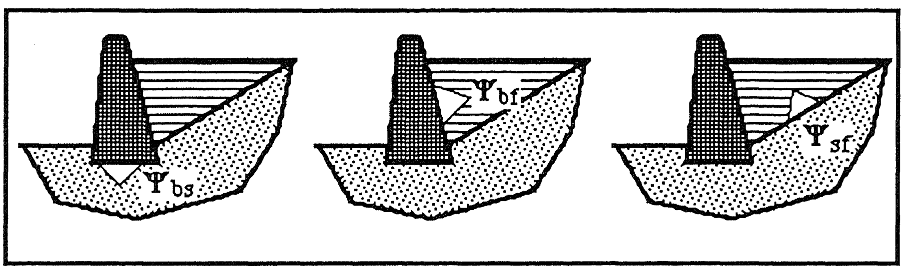

Figure 2 :Soil-structure, Fluid-structure, Soil-fluid interface fields

Consequently the interface fields $\Psi^{bs}{}_M$, $\Psi^{bf}{}_M$, $\Psi^{sf}{}_M$ are defined as interface modes. With the help of these modes we shall build in each domain the fields which match with the preceeding fields. Possible choices of these modes wil be discussed later on.

Decomposition of the displacement in the structure Let $u^{bs}{}_M$ and $u^{bf}{}_M$ be elastodynamic displacements in the structure which satisfy respectively the following boundary conditions :

$$u^{bs}{}_M = \Psi^{bs}{}_M \text{ on } \Gamma_{bs}, = 0 \text{ on } \Gamma_{bf}; \qquad u^{bf}{}_M = 0 \text{ on } \Gamma_{bs}, = \Psi^{bf}{}_M \qquad \text{on } \Gamma_{bf} \quad (7)$$

Then the total displacements u_b in the structure may be expanded by using the modal participation factors $\alpha^{bs}{}_M$ and $\alpha^{bf}{}_M$:

$$\mathbf{u}_b = \Sigma \ \alpha^{bs}{}_M \ \mathbf{u}^{bs}{}_M + \Sigma \ \alpha^{bf}{}_M \ \mathbf{u}^{bf}{}_M \quad (8)$$

Decomposition of the displacement in the soil Corresponding to the decomposition on the interface modes another decomposition of the movement of the soil taking into account the seismic incident field:

$$\mathbf{u}_s = \mathbf{u}_i + \mathbf{u}_{do} + \Sigma \ \alpha^{bs}{}_M \ \mathbf{u}^{sb}{}_M + \Sigma \ \alpha^{sf}{}_M \ \mathbf{u}^{sf}{}_M \quad (9)$$

All the preceeding fields except $\mathbf{u}_i$ are elastodynamic in the soil. They differ by the boundary conditions at the interface with the structure and the bottom of the reservoir :

$$\mathbf{u}_{do} = -\mathbf{u}_i \ \text{ on } \Gamma_{bs}, \ \mathbf{u}_{do} \cdot \mathbf{n}_s = -\mathbf{u}_i \cdot \mathbf{n}_s \text{ on } \Gamma_{sf}, \ \ t_{sT}(\mathbf{u}_{do}) = - \ t_{sT}(\mathbf{u}_i) \ \text{ on } \Gamma_{sf} \quad (10)$$

with similar expressions for $\mathbf{u}^{sb}{}_M$ and $\mathbf{u}^{sf}{}_M$.

Decomposition of the pressure field in the reservoir To the interface modes correspond in the same manner acoustic pressures in the reservoir satisfying particular boundary conditions in such a way that the dynamical pressure field in the reservoir may be expanded :

$$p = \Sigma \ \alpha^{bf}{}_M \ p^{fb}{}_M + \Sigma \ \alpha^{sf}{}_M \ p^{fs}{}_M \quad (11)$$

Equilibrium conditions across the interfaces The equations of continuity of the stress vector across the various interfaces shall provide the equations that must be imposed to the coefficients α. These equations will be expressed as always here in the sense of the principle of virtual works, for each domain. Let $\mathbf{v}_b$ be a virtual kinematically admissible displacement field of the structure, then the principle of virtual works gives :

$$< \mathbf{t}_b (\mathbf{u}_b) , \mathbf{v}_b >_{bsUbf} = < p \, \mathbf{n}_f , \mathbf{v}_b >_{bf} - < \mathbf{t}_s (\mathbf{u}_s) , \mathbf{v}_b >_{bs} \quad (12)$$

By choosing for $\mathbf{v}_b$ the interface fields defined above, and by using the decomposition of the elastodynamic displacements in the structure, the first equations that the participation factors α must satisfy are obtained:

$$\Sigma \ \alpha^{bs}{}_M < \mathbf{t}_b (\mathbf{u}^{bs}{}_M) + \mathbf{t}_s (\mathbf{u}^{sb}{}_M) , \Psi^{bs}{}_N >_{bs} + \Sigma \ \alpha^{bf}{}_M < \mathbf{t}_b (\mathbf{u}^{bf}{}_M) , \Psi^{bs}{}_N >_{bs}$$

$$+ \Sigma \ \alpha^{sf}{}_M < \mathbf{t}_s (\mathbf{u}^{sf}{}_M) , \Psi^{bs}{}_N >_{bs} = - < \mathbf{t}_s (\mathbf{u}_i + \mathbf{u}_{do}) , \Psi^{bs}{}_N >_{bs} \quad (13)$$

Similar equations are developped for the dynamical equilibrium of the fluid-structure interface and the dynamical equilibrium of the soil-fluid interface. The three corresponding equations provide the determination of the α with the seismic loading in the right hand side. To compute the coefficients of this system, the underneath terms must be established: (1). Design the interface fields Ψ, (2). In each domain : compute the elastodynamic corresponding fields, (3). Compute the above integrals along the interfaces. After this system is solved for the α, then it is possible to perform the modal synthesis in each domain and mainly in the structure in order to compute the total displacement $\mathbf{u}_b$, acceleration level and stresses.

CONSTRUCTION OF THE INTERFACE FIELDS

Particular case of a rigid soil foundation In those conditions, $\mathbf{u}_i$ is known and the stress vectors $\mathbf{t}_s$ in the soil vanish. It is then tempting to use for the interface modes : (1). At the soil-structure interface the six rigid body modes; (2). At the fluid-structure interface the six rigid body modes and the rigid basis eigenmodes of the dam. It should be noted that this basis is not elastodynamic but only its boundary value along the interfaces are needed.

Local interface modes defined by a full coupling between finite and boundary elements This is the most numerical approach where the interface modes are chosen equal to the successive finite element shape functions of the interface nodes. The computation of the elastodynamic solutions in the structure is then equivalent to dynamic condensation.

Global modes defined by the eigenmodes of the dam resting on elastic support The above method is simple but certainly very costly as it uses the whole local basis at the interfaces. The following basis is used in this paper. First of all the eigen modes of the dam resting on an elastic support are computed by the finite element method. These modes define the interface modes at the dam-fluid and dam-soil interfaces after which the elastodynamic displacements in the structure may be evaluated. Then the soil-fluid interface mode $\Psi^{sf}{}_M$ is defined as being the boundary value of an elastodynamic displacement in the soil which satisfies the following boundary conditions :

$$\mathbf{u}^{sf}{}_M = \Psi^{bs}{}_M \text{ on } \Gamma_{bs}, \ \sigma_n(\mathbf{u}^{sf}{}_M) = i\, c\, \rho_f\, \omega\, (\mathbf{u}^{sf}{}_M.\mathbf{n}_f)\,, \ \ \mathbf{t}_{sT}(\mathbf{u}^{sf}{}_M) = 0 \text{ on } \Gamma_{sf} \qquad (14)$$

Again these elastodynamic displacements are computed by boundary elements.

APPLICATION TO THE FULL SEISMIC ANALYSIS OF AN ARCH DAM

The above method of analysis has already been applied several times in the case of a rigid foundation for the dam and reservoir. It is generalized here to the possibility of propagating waves in the soil. The dam is 120 meters high with a length of 330 meters and a 5 meters thickness at the top. The arch is modelled by 108 thick shell finite elements with 1500 degrees of freedom. For the fluid analysis the bottom of the reservoir and the upward shell of the dam have been meshed with 380 fluid boundary elements of over 400 meters upwards. For the site effect analysis the soil surface has been meshed with 495 solid boundary elements both upwards and downwards. The first eleven modes of vibration of the arch resting on an elastic foundation have been computed with the empty reservoir The first frequencies respectives are of 2.5, 3., 3.6, 4.9, 5.6 Hz which are to be compared with the eigenfrequencies on a rigid basis : 2.8 3.9 4.2 5.3 6.7 Hz. On the figure 3 the mode shape number 4, 5 and 8 are shown. The influence of the flexible foundation is clearly seen. On the figure 4, the seismic site effect for an incident P wave inclined to 45 degrees and propagating along the axis of the valley is shown as the modulus of the y-componant of the diffracted field $\mathbf{u}_{d0}$. On the figure 5 the corresponding pressure mode at the reservoir bottom and dam shell is shown. It is seen that the effects of the propagation is not limited to the neighborhood of the dam. Space limitations do not allow to show further results and especially the componant mode synthesis.

CONCLUSION

A modular approach for the three-dimensional analysis of arch dams submitted to earthquake loadings including fluid reservoir and soil interaction has been discussed. A particular emphasis is given to domain decomposition in order to use the best numerical technique for each domain. The arch dam is modelled by thick shell finite elements, the fluid reservoir by fluid boundary elements and the soil by solid boundary elements so that only the interfaces need to be meshed. A reduced basis approach is developped by using special kinematic modes on the interfaces between the three domains. A computer code consisting of several modules one for each domain has been implemented and can now be applied to the full seismic analysis.

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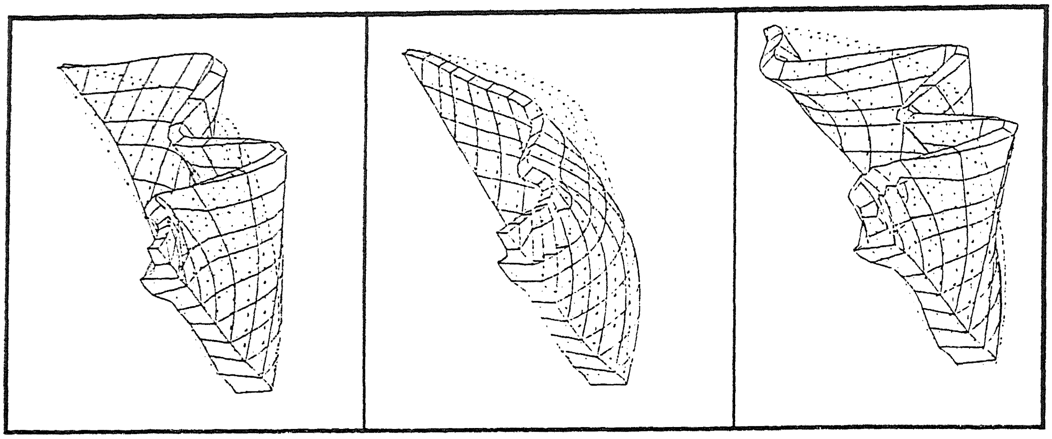

Figure 3: Mode shapes no 4, 5, 8 of the dam on elastic foundation

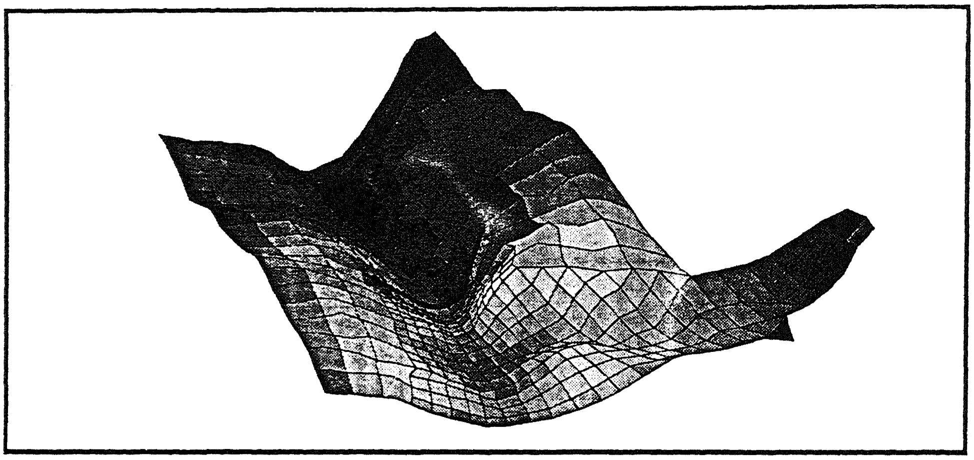

Figure 4: site effect, modulus of the y-componant of the diffracted field $\mathbf{u}_{d0}$

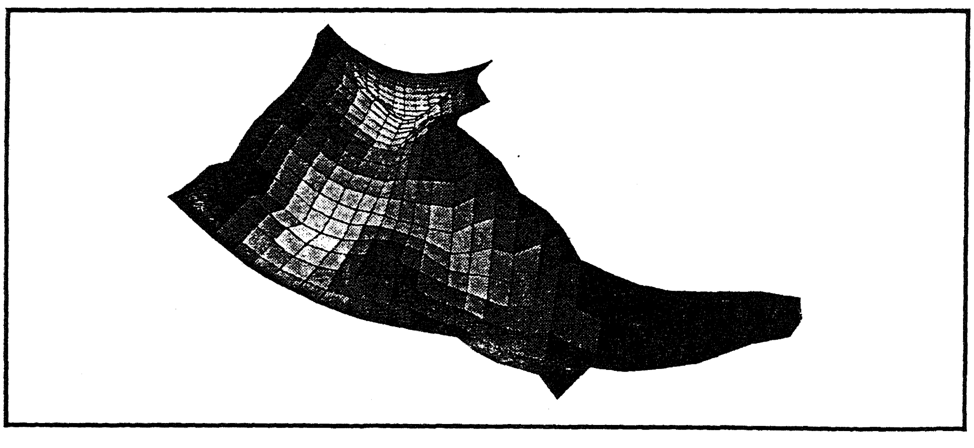

Figure 5: Dynamic pressure resulting from site effect