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Reservoir-dam-soil interaction: a comparative study

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Abstract

Direct time domain numerical procedures to analyze the transient dynamic response of reservoir-dam-soil systems are compared. Emphasis has been given to the modelling of the dam-soil interaction. The dam is modelled by the finite element method, water is modelled using a displacement-based Lagrangian element. The foundation has been considered in five ways: (a) by assuming a rigid foundation; (b) by including a significant massless portion of the foundation rock; (c) foundation rock as a viscoelastic half-space represented by a stiffness and a damping matrix, computed by neglecting the frequency dependence of the dynamic stiffness of the soil, assuming the first eigenfrequency of the dam as the only one relevant to system; (d) the same as (c) but neglecting the damping; (e) using the analytical solution in frequency domain and Fourier synthesis. Illustrative numerical results are presented.

1 Introduction

The behavior of dams may be greatly influenced by the foundation conditions. To take this effect into account, it is usual practice to include a significant portion of the foundation rock together with the dam in the finite element analysis. This approach is adequate under static loads where the only function of the foundation is to represent the flexibility of the support medium. However, under vibrations, such a model causes spurious wave reflections from the finite boundary of the foundation considered and thus not represent the actual behavior.

However, Fok and Chopra [1] have included a sufficient portion of the foundation rock to represent only the static foundation flexibility effects; the foundation rock was assumed to be massless for the dynamic analysis, and the earthquake input was specified as spatially-uniform motion of the basement rock. Since there is no wave propagation mechanism in the massless foundation rock, the specified basement rock motion is transmitted without modification to the dam-foundation rock interface. In the context of the substructure method of analysis, the above-mentioned approximation is equivalent to specifying the same free-field motion through the dam-foundation rock interface, with the foundation rock assumed to be massless in computing the foundation impedance matrix.

Frequency domain solutions to consider the half-space have become available using infinite elements [4], as well as boundary elements [5,6] and semi-analytical solutions [7]. Outstanding work on reservoir-dam-foundation in the frequency domain or the indirect time-domain has been carried out by Chopra and his co-workers [2] using the Fourier synthesis technique. The reservoir-dam-foundation interaction problem has been studied in the direct time-domain by Antes and Estorff [8] using the full-space transient Green's function for wave propagation in both the foundation and the reservoir. Wolf and Obernhuber [9], have introduced the application of the flexibility of soil, using the Green's functions in the time-domain, for non-linear soil-structure interaction analysis. Lately Guan et al [10], have represented an elastic, isotropic and homogeneous soil by a boundary condition in the form of generalized impedance.

Numerical procedures to investigate the dynamic response of a dam-soil system directly in the time-domain are compared in this paper. The dynamic soil-structure interaction is included in five different ways mentioned later. The response of a reservoir-dam-soil system subjected to vertical and horizontal earthquake ground motion is examined.

2 Concrete gravity dam

An independent monolith under plane-strain condition is considered. In the absence of cracks, the plain concrete throughout the monolith is assumed linear, elastic, isotropic and homogeneous. Structural damping is defined as Rayleigh's damping

$$[C_s]^e = \alpha\,[K_s]^e + \beta\,[M_s]^e \qquad (1)$$

where α and β are determined by specifying a desired damping at two given frequencies. However, when cracks are included in the analysis the mass-proportional term should be omitted because it would provide some artificial stability to the portion of the dam above the crack [3]. The present analysis uses either a 6-node triangle linear strain element (LST) or a 4-noded element, with 5% damping for the first two eigenfrequencies.

The equation of motion for the discretized dam can be written as

$$[M_s]\{\ddot{U}_s\} + [C_s]\{\dot{U}_s\} + [K_s]\{U_s\} = -[M_s]\{\ddot{U}_s^g\} \qquad (2)$$

where $[M_s]$, $[C_s]$, and $[K_s]$ are mass, damping, and stiffness matrices, and $\{\ddot{U}_s^g\}$ is the free-field ground motion at the basement.

3 Reservoir

The displacement-based fluid model adopted in this investigation is based on the one described in [11] where the fluid is assumed linear, compressible, inviscid and irrotational. In this paragraph, the water element matrices and vectors are defined, and the boundary conditions to the water are incorporated into the finite element equations.

The element mass matrix $[M_w]^e$, and the earthquake load vector $\{F_{dyn}\}$ are identical to their counterparts for the dam, except for the use of the water mass density ρ_w. Since the fluid is assumed inviscid, its strain energy is only due to deformational modes with volumetric strains. This leads to the following tangent element stiffness matrix:

$$[K_w]^e = \int_{V^e} [B_w]^T k_T [B_w]\, dV \qquad (3)$$

where k_T is the tangent water compressibility, and $[B_w]$ is the nodal displacement -volume strain transformation matrix. Reduced integration must be used to obtain stiffness-free element bending modes. This formulation enables cavitation to be considered by relaxing k_T in the element whereas the water cavitation strain, $V_c = p_c/k_w$, is reached, where p_c the cavitation pressure and k_w elastic compressibility of water.

The element stiffness matrix as defined in Eqn. (3) has mostly zero-energy deformational modes, and hence the assembled water stiffness matrix is expected to be singular. This is remedied by enforcing the irrotational condition and including the linearized small amplitude wave boundary condition at the free surface [11]. Irrotationality is included through a penalty formulation that leads to an additional term in the element stiffness matrix given by

$$[K_i]^e = \int_{V^e} [B_i]^T \alpha_i [B_i]\, dV \qquad (4)$$

where α_i is a penalty number, and $[B_i]$ is the nodal displacement-vorticity transformation matrix (i for irrotational). Reduced integration is again used in integrating Eqn. (4), as required by penalty methods. Including the small amplitude free-surface waves,

$$p = -\rho_w g u_n + p_{atm} \tag{5}$$

on the free surface, where p absolute pressure (static + dynamic), ρ_w density of the water, u_n is the normal component of the free-surface displacement; and g is the gravitational acceleration. The low frequency behavior of a fluid system involves incompressible modes of displacements. This sloshing mode of motion is shown in Figure 1. The weight df, of a column of fluid of area dS, and height $D + u_n$ is given by

$$df = \rho_w g (D + u_n) dS \tag{6}$$

The average vertical displacement of the fluid column is $u_n/2$; therefore, the increase in the potential energy of the system is

$$\Pi_s = \frac{1}{2}\int_{S_f} u_n \rho_w g u_n dS + \frac{1}{2}\int_{S_f} \rho_w g u_n dS \tag{7}$$

The first integral will produce surface stiffness terms. The second integral represents the weight of the fluid which is normally evaluated as an element volume integral rather than as surface integral. For a fluid element with a face at the open side an additional surface stiffness arises as

$$[K_{surface}]^e = \int_{S_f} \rho_w g [\bar{N}]^T [\bar{N}] dS \tag{8}$$

where $[\bar{N}]$ is the shape function matrix whose term are connected to the nodes at the surface. Thus, the total stiffness matrix becomes

$$[K_T]^e = [K_w]^e + [K_i]^e + [K_{surface}]^e \tag{9}$$

Energy dissipation associated with the water is due to radiation in the infinite upstream direction. This dissipation can be introduced by enforcing the so called Sommerfeld condition or using a analytical solution [11]. It is doubtless advantageous to introduce radiation because a huge number of degrees-of-freedom can be saved, nevertheless, for the sake of simplicity, in this work a large portion of the reservoir was included in the analysis; this is feasible because theoretical solutions for rigid tanks show that for a length greater than three times the height of the wall, the answer does not depend any more on the length of the reservoir.

4 Contact reservoir-dam

The contact dam-fluid is enforced by an interface element derived from the penalty method. The compatibility condition means that the normal displacements at the interface should be the same. The previous condition holds for every couple of nodes located at the interface region Ω, thus the compatible restrains can be written in matrix form as $[C]\{U\} = \{0\}$. Introducing the constraint condition into the problem formulation by the penalty function method, the stiffness matrix in the global system, $[K]_{interface}$, will be given as $[K]_{interface} = \kappa[C]^T[C]$ where κ is a large number, about two or three order of magnitude larger than the largest stiffness coefficient of the structure.

5 Dam-Soil Interaction

The modelling of the dam-soil interaction may play an important role in the seismic analysis of reservoir-dam-soil systems, therefore it is worth to investigate the performance of the different ways to model the dam-soil interaction phenomena. The five models that have been used in this work are:

(a) Ideal rigid foundation
(b) Inclusion of a significant large portion of soil
(c) Representative stiffness and damping matrix as superelement
(d) Representative stiffness matrix as superelement
(e) Analytical solution

Finally, the equation system may be coupled, leading to the following general equation:

$$\begin{bmatrix} M_{ss} & M_{sb} \\ M_{bs} & M_{bb} \end{bmatrix}\begin{bmatrix} \ddot{u}_s \\ \ddot{u}_b \end{bmatrix} + \begin{bmatrix} C_{ss} & C_{sb} \\ C_{bs} & C_{bb}+C_f \end{bmatrix}\begin{bmatrix} \dot{u}_s \\ \dot{u}_b \end{bmatrix} + \begin{bmatrix} K_{ss} & K_{sb} \\ K_{bs} & K_{bb}+K_f \end{bmatrix}\begin{bmatrix} u_s \\ u_b \end{bmatrix} = -\begin{bmatrix} M_{ss} & M_{sb} \\ M_{bs} & M_{bb} \end{bmatrix}\begin{bmatrix} \ddot{u}_s^g \\ \ddot{u}_b^g \end{bmatrix} \quad (10)$$

where the subscripts b and s denote the nodes at the dam-soil boundary and the remaining nodes of the system respectively, the superscript g denotes the free-field ground motion. The interface water-dam has not been explicitly denoted.

6 Comparative study

Numerical computations of the dynamic response of a realistic case were carried out in order to assess the performance of different ways to model the reservoir-soil-structure interaction directly in the time domain. For comparative aims the same system was analyzed using the computer code EAGD'84 that uses an indirect time-domain approach by applying Fourier synthesis; further details can be found elsewhere [2, 7].

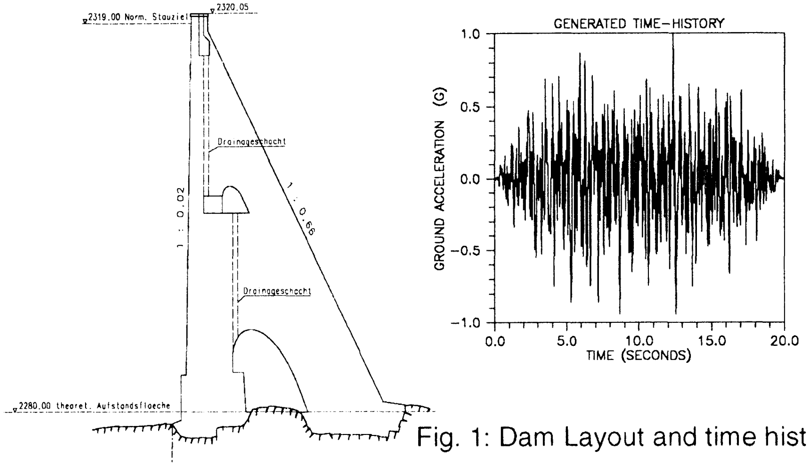

Fig. 1: Dam Layout and time history

The structure under consideration (Fig.1) is a 42 m high gravity dam located in Carinthia, Austria. The excitation are two synthetic generated earthquakes that fulfill standard response design spectra. The normalized time history is shown; the applied horizontal acceleration has a maximum value 0.1 g, and the applied vertical acceleration has a maximum value of 0.067g.

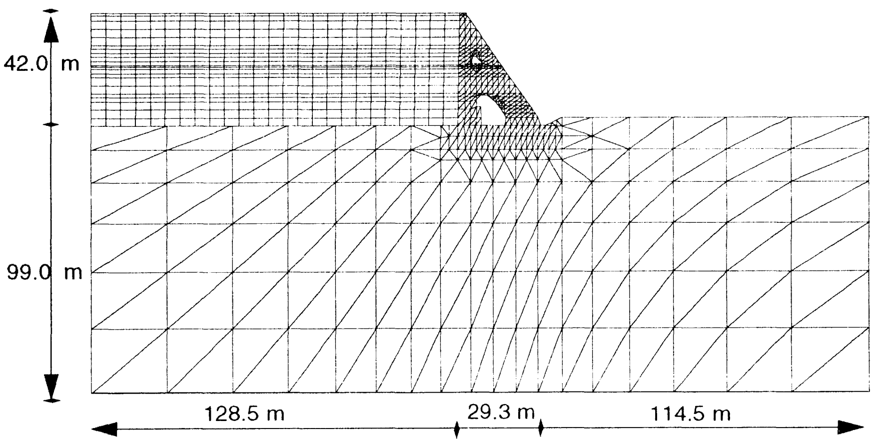

Fig. 2: Considered finite element mesh

The calculations for the models (a) to (d) were carried out using the finite element system FINAL [12]; the model (e) with the program EAGD84. As meaningful parameters we select the extreme and 'root mean square', *rms*, values of the horizontal acceleration at the crest and the vertical stress at the heel of the dam; the results are presented in Table 1.

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Table 1: Upstream dam crest accelerations and heel stresses

model	ACCELERATION (m/s^2)			STRESS (MPa)		
	Horizontal			Vertical		
	max	min	*rms*	max	min	*rms*
a	3.88	-3.79	1.22	1.14	-0.88	0.29
b	6.84	-5.26	1.66	1.39	-1.26	0.42
c	3.33	-2.83	0.97	0.88	-0.81	0.25
d	4.36	-3.97	1.19	0.99	-1.04	0.32
e	5.93	-5.42	1.92	0.91	-0.98	0.28

7 Conclusions

As a result of this comparative study, several conclusions have been reached. These are summarized as follows:

1) In spite of that considering a rigid foundation (model a) does not take into account the conditions in site, it produces reasonable results that could be useful for preliminary non-linear analysis, specially concerning the vertical stress at the heel.

2) Modelling the foundation by mean of a large portion (model b) of ground produces larger vertical stresses near the heel. This is not surprising because the resulting stiffness matrix is independent of the frequency, and it is known that for higher frequencies the compliances of the foundation decrease.

3) Modelling the foundation using the stiffness and damping matrix for the first eigenfrequency of the dam (model c) result in disminished values of crest acceleration and vertical stresses. This can be attributed to excessive damping for frequencies other than the first eigenfrequency, resulting in an overdamped solution.

4) Introducing only the stiffness matrix for the first eigenfrequency of the dam (model d) shows results in good agreement with those that consider a general analytical solution (model e). It is also much more economic in computing time compared with model (b) because of the condensation; therefore it appears as a promissory tool applicable in non-linear seismic analysis of dams.

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