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NONLINEAR DAM - RESERVOIR INTERACTION ANALYSIS

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ABSTRACT

The earthquake analysis of a single-degree-of-freedom model of a gravity dam on rigid foundation responding in the nonlinear range is performed. The dam impounds a reservoir of compressible water considered to extend to infinity for calculation purposes. The dam's nonlinear behavior and the frequency dependence of the hydrodynamic mass interaction coefficients are accounted for by using the hybrid frequency-time domain procedure as a solution scheme. It appears that water compressibility substantially affects the dam's nonlinear response. The study performed is exploratory and this conclusion should not be unduly generalized.

KEYWORDS

Earthquake response; fluid-structure interaction; numerical procedure; nonlinear analysis; frequency analysis; compressible water; gravity dam.

INTRODUCTION

Concrete dams may perform in the nonlinear range during strong earthquakes due to overstressing of the dam body, sliding and overturning at the foundation-dam interface, and nonlinear behavior of the foundation (Chopra and Zhang, 1991). The analysis of nonlinear dam behavior requires the use of a time-stepping algorithm with which the development and the progression of the nonlinearities are followed with time. Concurrently, the influence of the compressible reservoir fluid on the earthquake response is represented by complex-valued interaction coefficients which vary with frequency. Their direct account requires an analysis in the frequency domain. This dual requirement of a time-domain analysis and of a frequency-domain analysis has been the incentive for developing the hybrid frequency-time domain (hftd) procedure. The hftd procedure is used here to analyze the single-degree-of-freedom model of a gravity dam on rigid rock responding in the nonlinear range. The dam impounds a reservoir of compressible water considered to extend to infinity for calculation purposes.

Fundamental equations

Referring to Fig. 1, the equation of motion for a dam-reservoir interacting system is

$$[M]\{\ddot{u}(t)\}+\{F_{\text{int}}(t)\}=-[M][A]\{\ddot{u}_g(t)\}+\{P_d(t)\} \tag{1}$$

where [M] and [A] are the mass and kinematic compatibility matrices, {u(t)} and {u_g(t)} are the vectors of relative and ground motions, {F_{int}(t)} is the vector of internal forces that depend on the history of incursions in the nonlinear range and {P_d(t)} is the hydrodynamic load vector. The latter is related to the dam relative and ground motions through the complex-valued frequency-dependent added-mass matrices [$L_g(\omega)$] and [$L(\omega)$] as

$$\{P_d(\omega)\}=-[L_g(\omega)][A]\{\ddot{u}_g(\omega)\}-[L(\omega)]\{\ddot{u}(\omega)\} \tag{2}$$

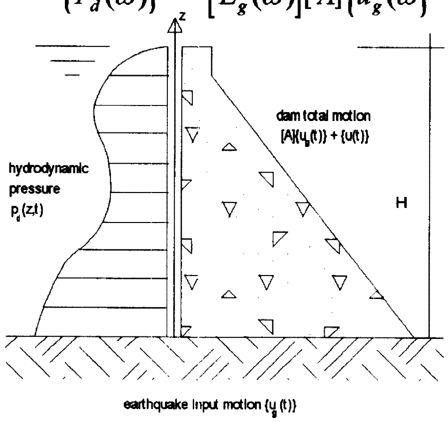


Fig. 1 Dam-reservoir interacting system

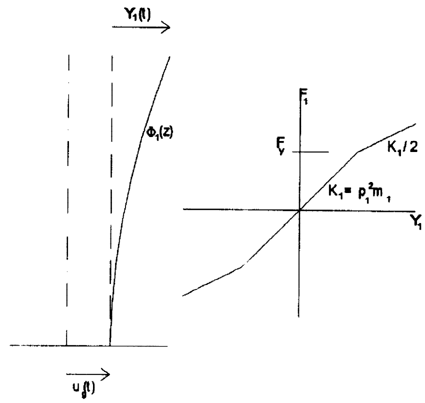


Fig. 2 SDOF nonlinear model

Hybrid frequency-time-domain formulation

In the hftd formulation, {F_{int}(t)} is decomposed into a linear component (usually selected to correspond to the initial tangent properties of the system) and into a nonlinear remaining component {-Q(t)} as

$$\{F_{\text{int}}(t)\}=[C]\{\dot{u}(t)\}+[K]\{u(t)\}-\{Q(t)\} \tag{3}$$

where [C] and [K] are damping and stiffness matrices, respectively. Treating the nonlinear component {Q(t)} as a pseudo-load (and thus bringing it to the right-hand side of equation 1) leads to the following differential equation that governs the behavior of the pseudo-linear system

$$[M]\{\ddot{u}(t)\}+[C]\{\dot{u}(t)\}+[K]\{u(t)\}=-[M][A]\{\ddot{u}_g(t)\}+\{P_d(t)\}+\{Q(t)\} \tag{4}$$

This equation can be solved in the frequency domain when combined with equation 2

$$\left(-\omega^2([M]+[L(\omega)])+i\omega[C]+[K]\right)\{u(\omega)\}=\omega^2\left([M][A]+[L_g(\omega)][A]\right)\{u_g(\omega)\}+\{Q(\omega)\} \tag{5}$$

The pseudo-load vector {Q} depends on the structural response through nonlinear constitutive relations. The solution of equation 5 is thus obtained in an iterative fashion. Details on the hftd procedure and the way it is implemented here are found in (Darbre and Wolf, 1988, and Darbre, 1990).

Modal decomposition

The relative motion {u(t)} is expressed in terms of the generalized modal coordinates {Y(t)} and matrix [Φ] of modal vectors. The governing equation 5 then becomes

$$\left(-\omega^2\left([\Phi]^T[M][\Phi]+[\Phi]^T[L(\omega)][\Phi]\right)+i\omega[\Phi]^T[C][\Phi]+[\Phi]^T[K][\Phi]\right)\{Y(\omega)\}= \omega^2\left([\Phi]^T[M][A]+[\Phi]^T\left[L_g(\omega)\right][A]\right)\left\{u_g(\omega)\right\}+[\Phi]^T\{Q(\omega)\} \qquad (6a)$$

or, in a conciser form,

$$\left(-\omega^2([m]+[B(\omega)])+i\omega[c]+[k]\right)\{Y(\omega)\}=\omega^2\left(\left[D_g(\omega)\right]+\left[B_g(\omega)\right]\right)\left\{u_g(\omega)\right\}+\{q(\omega)\} \qquad (6b)$$

All matrices on the left-hand side are diagonal (assuming that damping can be expressed accordingly), with the exception of the hydro-dynamic mass matrix [B(ω)].

DAM-RESERVOIR SYSTEM ANALYZED

Single-degree-of-freedom dam model

The model of Fig. 2 is considered. The dam rests on rigid rock and is presumed to deform in its fundamental natural mode given by (Chopra, 1970)

$$\Phi_1(z)=0.18(z/H)+0.82(z/H)^2 \qquad (7)$$

The rigidity of the dam associated with this mode shape is assumed to be bilinear-elastic as illustrated in Fig. 2. The geometry and material properties used are:

Dam's height H	200 m
Concrete's mass density ρ_c	2'400 kg/m^3
Fixed-based dam's linear fundamental frequency p_1	13.5 rad/sec
Dam's linear structural damping ζ_1	5% of critical
Water's mass density ρ_w	1'000 kg/m^3
Water pressure waves velocity c_w	1'440 m/sec

Interaction coefficients

The interaction coefficients used are those valid for a prismatic reservoir of infinite length containing compressible water, assuming that the waves impinging on the bottom are fully reflected. Assuming the reservoir to extend to infinity in the upstream direction reduces the effort associated with the numerical discretisation of the reservoir as it allows the use of readily available analytical expressions or relatively straightforward development thereof. In a practical situation, one way of deciding if the fluid region is of infinite extent for calculation purposes is to compare the time needed by a water pressure wave created at the dam-fluid interface to reach the back of the reservoir of length L and come back to the dam-reservoir interface (duration $2L/c_w$) to the duration of earthquake strong ground motion T_s (although the duration of dam strong response might be slightly different). The assumption of infinite length is appropriate if $2L/c_w > T_s$. This argumentation may be questioned in the case of nonlinear behavior as an increase in damage associated with the response of the dam to waves returning after the severer portion of excitation / response is over can occur. In this case, the duration $2L/c_w$ can be compared to the total duration of the calculation.

The interaction coefficient B_{11} is shown in Fig. 3 as a function of the dimensionless frequency $\omega H/c_w$ (see e.g. Fenves and Chopra, 1984). It is reported as the dimensionless mass and damping coefficients $Real(B_{11}(\omega))/\rho_w H^2$ and $-\omega H/c_w \, Imag(B_{11}(\omega))/\rho_w H^2$, respectively. B_{11} is the fundamental modal hydrodynamic mass associated with the fundamental dam's deformational mode. The coefficients obtained by assuming the water to be incompressible (Westergaard, 1933) and the ones obtained from the two-parameters model of Fig. 4 (Darbre, 1993) are also shown. In the latter phenomenological model, an incompressible body of water is attached to the dam through distributed dampers. In doing so, it is recognized that (1) the hydrodynamic pressure is due to the incompressible body of water entrained by the motion of the dam at low frequencies of harmonic excitation, and that (2) waves propagating upstream are generated at high frequencies of harmonic excitation. While this simple two-parameters model captures the essential features of dynamic-reservoir interaction, discrepancies remain. The greater value of the model thus lies in the insight that it provides in the way fluid contained in a reservoir influences the earthquake response of dams rather than - at least at this stage - in its use as a substitute to other models.

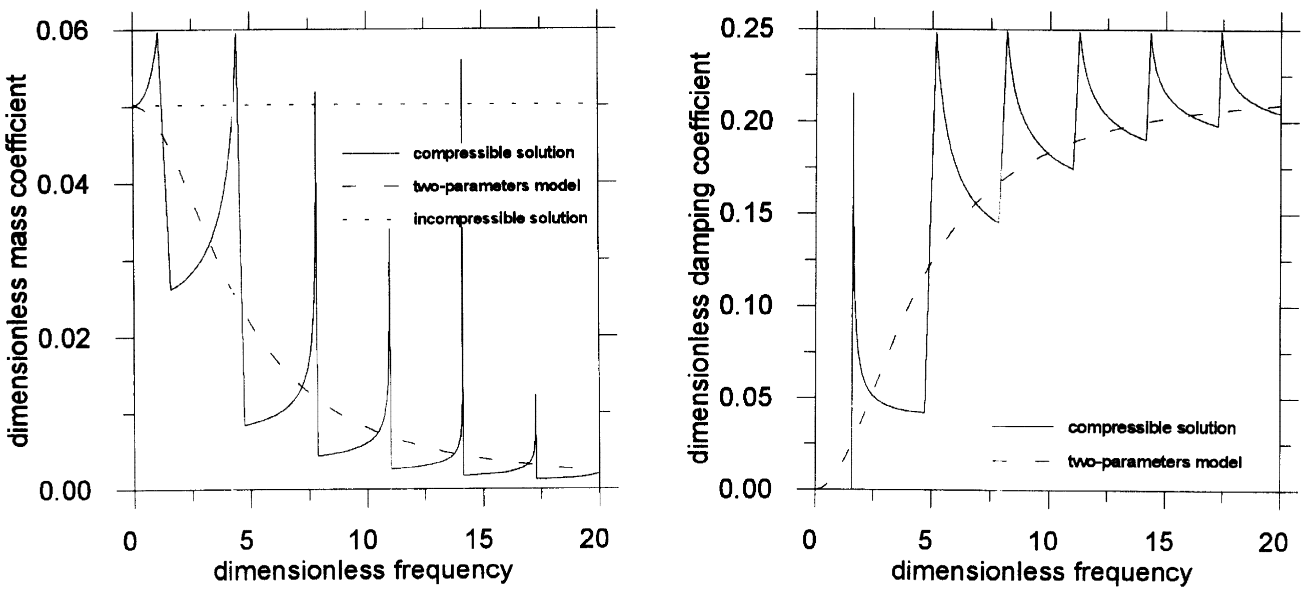


Fig. 3 Hydrodynamic interaction coefficient B_{11}

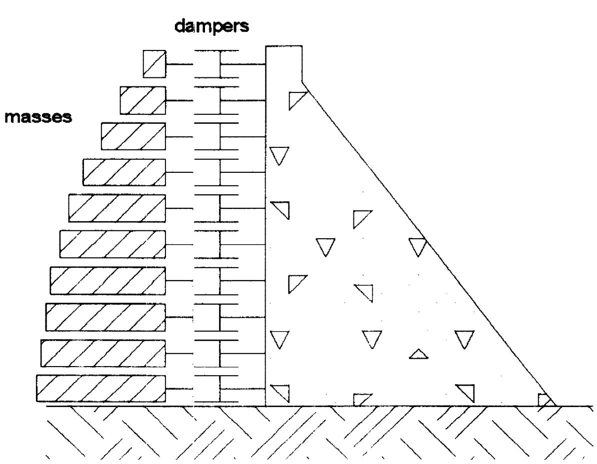


Fig. 4 Two-parameters model

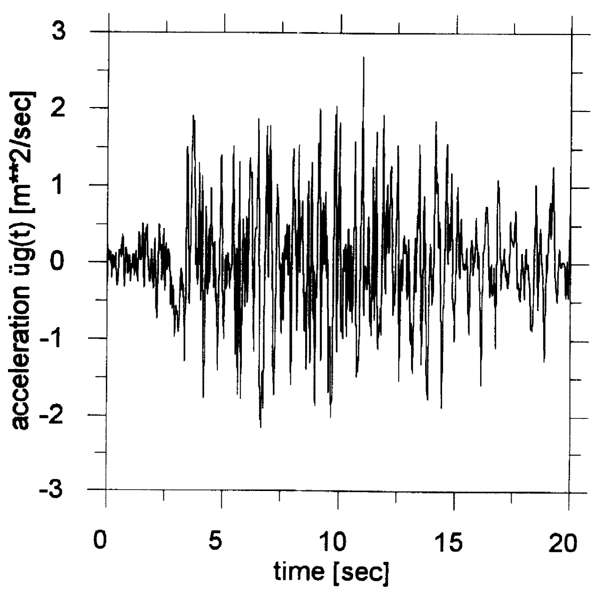


Fig. 5 Ground acceleration

RESULTS

Cases considered

The ground acceleration time history of Fig. 5 is introduced as an excitation (peak value of 30% g). The cases considered are those of empty reservoir, of reservoir filled with incompressible water and of reservoir filled with compressible water. The case of the two-parameters model for the reservoir is further considered. In all cases, the corner modal force F_y (Fig. 2) is varied to cover a wide range of nonlinear response conditions. A reference linear calculation is also performed for each case.

The peak values of modal displacement response $(Y_1)_{max}$, of base shear $(V)_{max}$ and of base overturning moment $(M)_{max}$ are compared. The latter base forces are obtained from equilibrium of the inertia and hydrodynamic loads.

Explicit scheme versus hftd procedure

All calculations for empty reservoir and for reservoir with incompressible water are performed using both an explicit integration scheme and the hftd procedure. This serves as a verification of the accuracy of the solutions obtained in the compressible case, what is necessary due to the exploratory nature of the study. The peak deformation, base shear and base overturning moment are reported in Figs. 6 to 8. While the discrepancies between both solutions are greater than expected based on previous calculations (Darbre, 1990 and 1991), the agreement is still considered acceptable. It is in particular recognized that two totally different integration methods are used (time domain in the explicit scheme and frequency domain in the hftd procedure) and that largely different integration time steps are used (Δt=0.001 [sec] for the explicit scheme and Δt=0.02 [sec] for the hftd scheme). Inspecting the response time histories, hardly any difference is seen. This is illustrated in Fig. 9 for the case of empty reservoir and $F_y = 10^7$ [N], the case showing the least agreement of all.

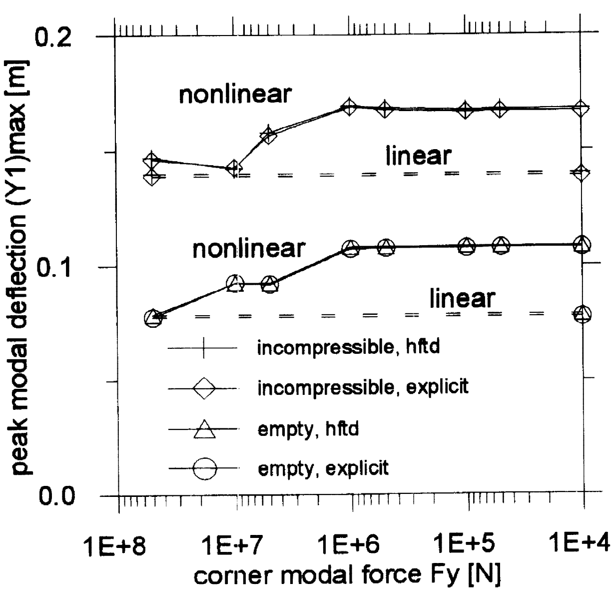


Fig. 6 Peak modal deflection: comparison of hftd and explicit schemes

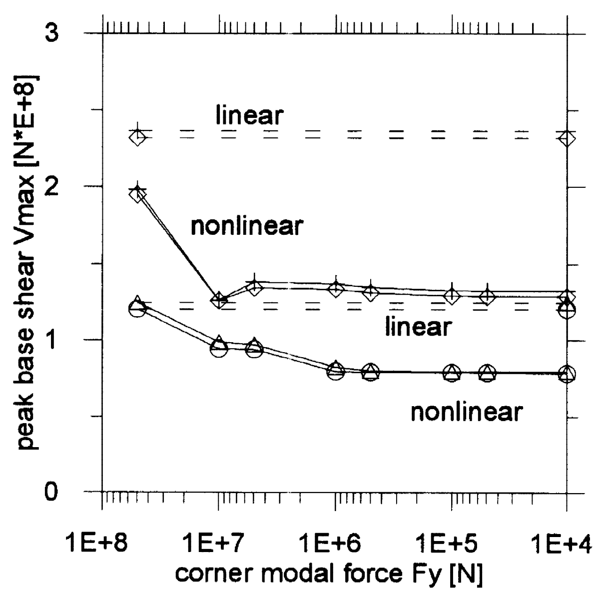


Fig. 7 Peak base shear: comparison of hftd and explicit schemes

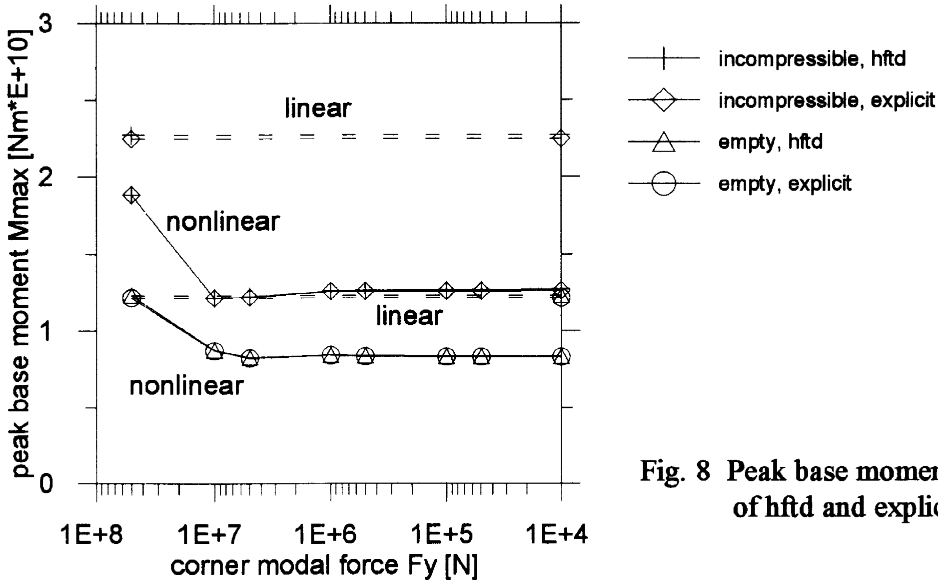
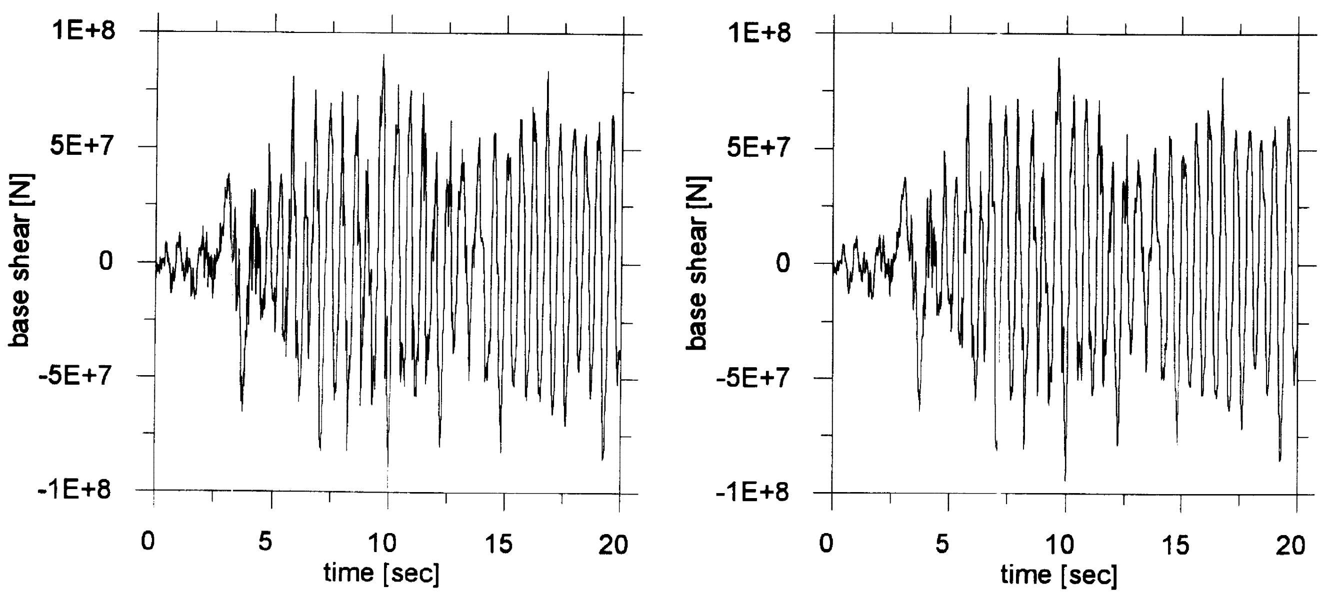


Fig. 8 Peak base moment: comparison of hftd and explicit schemes



a) hftd procedure b) explicit scheme

Fig. 9 Base shear time history for empty reservoir and $F_y = 10^7$ [N]

Nonlinear response for compressible water

The dam's nonlinear response for compressible water is obtained by application of the hftd procedure. The corresponding results are indicated in Figs. 10 to 12 and compared to the results of the other cases (all obtained by application of the hftd procedure). The following trends are observed:

- The responses for full reservoir are larger than for empty reservoir (applies to modal deflection and to base forces);
- The nonlinear behavior tends to increase the peak modal deflection (empty reservoir, incompressible water and compressible water) and to decrease the peak base forces (empty reservoir and incompressible water, no much changes for compressible water);
- The nonlinear peak base forces obtained for compressible water are generally larger than those obtained for incompressible water (substantial differences for peak base overturning moment);
- The frequency dependence of the interaction coefficients can have less influence on the nonlinear response quantities than on the linear ones (modal deflection and base shear), or more (base overturning moment);

- While it may be argued that the assumption of incompressible water leads to an acceptable estimate of the nonlinear peak modal deflection and base shear for compressible water, the estimate of the peak base overturning moment is inadequate;
- The two-parameters model is no more acceptable than the assumption of incompressible water.

From these limited results, it appears that water compressibility can affect substantially the dam's nonlinear response. A similar conclusion was reached by (Fenves and Chavez, 1990, and Chavez and Fenves, 1993) in situations involving nonlinearities in the dam body and sliding at the dam's base. This conclusion is different from the one reached for nonlinear soil-structure interaction. In the latter situation, nonlinear behavior tends to increase the overall flexibility and energy dissipation capability of the system. The soil's contribution to these parameters then becomes less important than in a linear situation and their approximate treatment does not affect the nonlinear response too much. In the case of reservoir-dam interaction, the effect of water is that of inertia and damping, the former losing none of its importance in a nonlinear situation. Furthermore, the variation of the hydrodynamic mass coefficients with frequency over the range of interest (see e.g Hall, 1988) is more substantial than the one of soil's spring coefficients.

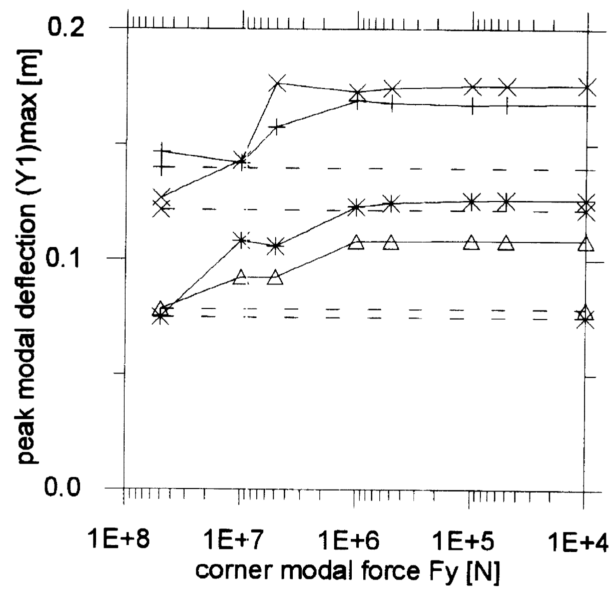


Fig. 10 Peak modal deflection

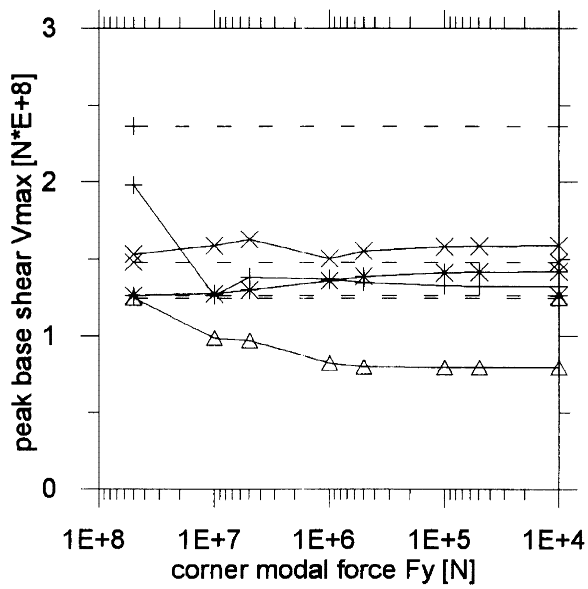
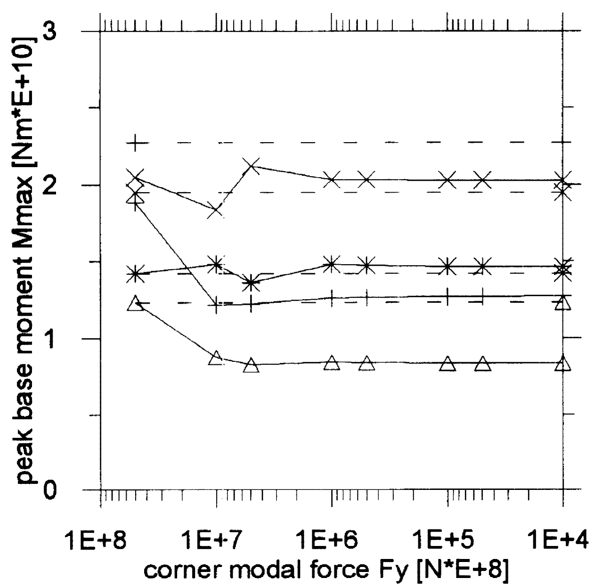


Fig. 11 Peak base shear



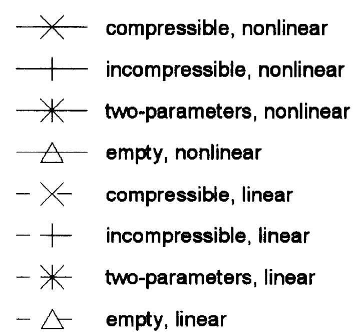


Fig. 12 Peak base moment

CONCLUSIONS

The study performed is exploratory and it is thus not appropriate to draw general conclusions. However, it appears that water compressibility is no less important in a nonlinear situation than it is in a linear situation. It is observed here that this is more particularly true for the base overturning moment. Fenves and Chavez, 1990, have isolated this aspect and looked at the response obtained from application of the incompressible water model to compute the modal deflection while using the compressible solution to obtain the base forces. This aspect should further be looked into, pragmatically by investigating a multi-degree of freedom dam's model permitting to obtain the base forces from the dam's deformation rather than from overall equilibrium.

From this and previous studies, the hftd procedure is seen to be a powerful numerical method to analyze nonlinear dam-reservoir interacting systems, in spite of the challenging numerical implementation. It is now part of a computer program for earthquake analysis of sliding gravity dams (Chavez and Fenves, 1994).

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