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Nonlinear Seismic Response Analysis of Realistic Gravity Dam-Reservoir Systems

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Abstract

A methodology for the earthquake response analysis of concrete gravity dam-reservoir systems is presented, giving emphasis at the development of an appropriate nonlinear model capable of reproducing the effects on response of all the forms of nonlinearities present in a realistic system. The numerical simulation of the displacement response history of a real-life system to a known seismic excitation has been performed using the finite element method and specially developed interface elements have been employed to model the discontinuities of the structure. The results obtained demonstrate that the earthquake response of the system is significantly affected by the behaviour at the interfaces between contacting materials.

Keywords: Gravity dam, joint, interface element, seismic response.

1 Introduction

Finite element methods for the linear elastic analysis of the responses of concrete gravity dams to earthquake excitations are well established in the literature and they are commonly used in practice. Extensive parametric studies have been carried out in the past by Chopra and his co-workers [1-3] using two-dimensional linearly elastic models. They have shown that the dam response is influenced to a significant degree by the interaction with its foundation, the interaction with the water in the reservoir, the water compressibility, and the absorption of hydrodynamic pressure waves onto the reservoir bottom materials. The seismic response analysis of a realistic system, which considers interaction between all three constituent aspects of the system, is complex and computational expensive. The complexity of the analysis is considerably simplified when the solid elements (i.e. the dam and/or the foundation soil) are assumed to be infinitely stiff. A number of investigators [4-6] have considered this problem in the past, the solution of which leads to unrealistic results, since the conventional simplifications introduced in the model diminish the integrity of the system model to an unacceptable level.

In the event of a major earthquake, it is recognised that the structure will be stressed beyond its elastic limit and thus significant structural damage may be caused. In such cases, the assumptions made in the linear-elastic range are no longer valid. Clearly, non-linear analysis procedures that account for tensile cracking in concrete and include the effects of opening, closing and sliding of joints or cracks, are necessary for the representative analysis of dams because the structural stiffness of the system will undergo changes as a result of significant damage [7].

Several sources of non-linear behaviour can be identified in a realistic dam-reservoir system, such as material non-linearity of the solid elements (i.e. dam concrete and foundation rock), cracking of concrete in the dam, cavitation in the reservoir water, relative motion at construction joints between monolithic blocks, imperfect bonding at dam-foundation interface, layering and the presence of a fault zone in the rock foundation mass. In this study, particular attention is drawn on the effects the construction joints have on the behaviour of a concrete gravity dam during earthquake motion. Concrete gravity dams are traditionally built as a system of independent concrete monoliths separated by contraction joints, which are either grouted or ungrouted, required to prevent haphazard cracking. In addition to the transverse and longitudinal (to the axis of the dam) joints, there are horizontal planes of weakness too. This is because a dam is constructed in a series of lifts, each lift being several feet high. The bond between the concrete in successive lifts is imperfect, possibly giving rise to planes of weakness. Imperfect bonding may also be present at the dam-foundation interface.

The division of the large body of a gravity dam by expansion joints leads to important advantages from the point of dynamic behaviour. The seismic load does not induce the motion of the entire dam, but of several interacting monolithic blocks of different vibration characteristics. The behaviour of a dam during large amplitude motion depends on the extent to which the inertia forces can be transmitted across the joints. Irrespective of the degree of grouting, the joints have a limited capacity to sustain tensile and shear stresses. Therefore, under seismic loads of large intensity (when the state of stresses in the joints exceeds their tensile or shear strength), sliding or opening at the joints may occur and, consequently, the dam losses its monolithic character and the monoliths vibrate independently. Such action is evidenced by the spalled concrete and increased water leakage at the joints of Koyna dam during the Koyna earthquake of 11 December 1967. The effect of these joints, which divide the dam into blocks, on the seismic response of gravity dams does not appear to have been studied so far, apart from the work reported recently in Refs [8-10]. Clearly, an analysis procedure to include the effects of these non-linear features on the seismic response of gravity dam-reservoir systems is needed.

2. The Problem and its Finite Element Modelling

The work reported in this paper is part of a wider research programme aiming at the investigation of the responses of realistic arch and gravity dams to earthquakes including dam-foundation interaction, reservoir-dam interaction, and modelling the dam as an assemblage of blocks with grouted or keyed construction joints. A general-purpose finite element computer program has been developed and used to study the behaviour of a real-life system to a known earthquake excitation. A similar study using an analytical procedure, based on the substructure method, has been performed in detail by Fenves and Chopra [11, 12] and it was considered as the test example in order to verify the accuracy of this work. In particular, the earthquake response of Pine Flat concrete gravity dam is computed (Figure 1), using both horizontal and vertical components of the recorded Taft (California) ground motion, with a maximum ground acceleration of 0.18g in the horizontal component (Figure 2). Since this is the only record used in the analysis, the response results obtained are not meant to be general, and they mainly depend on the characteristics of this particular excitation.

The Taft ground motion, however, is a typical moderate earthquake, particularly in the short-period range of its spectrum, which is the main range of interest in the analysis of concrete gravity dams [12].

Both dam-reservoir and dam-foundation rock interactions were considered in the analysis, and, in order to investigate the effect on response of the various discontinuities present in a realistic dam, the dam is also considered as a structure which is built-in-blocks with construction joints. Figure (1) shows the system geometry considered, together with a typical two-dimensional finite element discertization used in the analysis (the heavier lines represent joints). Conventional four-noded linear isoparametric elements have been employed to model the dam and its foundation, while behaviour at the joints between adjacent blocks of the dam and between the dam and its foundation, was modelled with specially developed interface elements designed to mimic the mechanics at these joints.

The finite element idealisation consists of 259 quadrilateral elements, with 295 nodal points and 544 degrees of freedom, when the foundation is assumed to be rigid. When foundation flexibility is included in the analysis, the finite element idealisation consists of 425 quadrilateral elements, with 467 nodal points and 896 degrees of freedom. The discretised area of the foundation is extended up to 300ft (0.75H) in depth, and 900ft (2.25H) along the axis of flow. The foundation base is assumed to be fixed, and side boundaries are allowed only horizontal displacements (Fig. 2). Both the dam and its foundation rock are assumed to be in a state of generalised plane stress. This assumption, although not appropriate for the foundation rock, is dictated by the relatively small longitudinal volume of foundation rock expected to participate in the earthquake response of a single dam monolith due to the expected behaviour of the non-keyed transverse construction joints.

Apart from the ground motion history, the input data required for the dynamic analysis of a dam-foundation-reservoir system, consist of the geometry and material properties of the dam concrete and foundation block, boundary conditions, damping coefficients and earthquake-induced water pressures (hydrodynamic forces). In this work, the influence of the reservoir water on the dynamic behaviour of the dam is considered by taking into account the mass of water attached to the upstream face of the dam. The extent to which the compressibility of the water in the impounded reservoir might be important to the seismic response of a realistic gravity dam-reservoir system has been investigated in a previous work by one of the authors [13], where it was concluded that in the case of realistic gravity dams, built on flexible foundations (and especially those with construction joints which further reduce the overall stiffness of the system), the compressibility effect of reservoir water on the dam's seismic response is generally not significant; except when the dam is very stiff compared with the reservoir. Consequently, in most cases, their seismic response can be accurately predicted via the 'added mass' method of analysis. It is for this reason that this simple representation was implemented in this study.

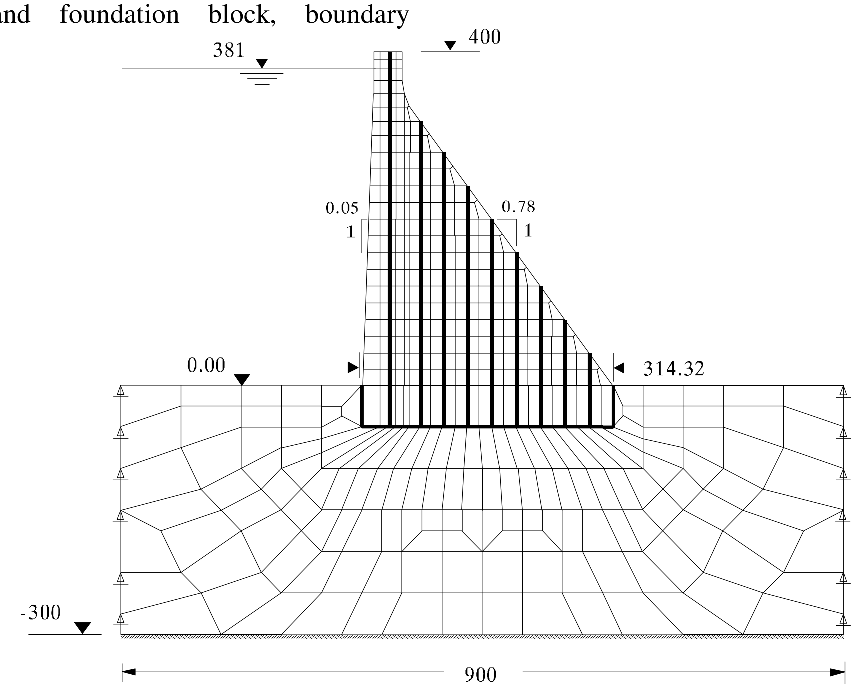

Fig. 1. System geometry and finite element discertization (in feet).

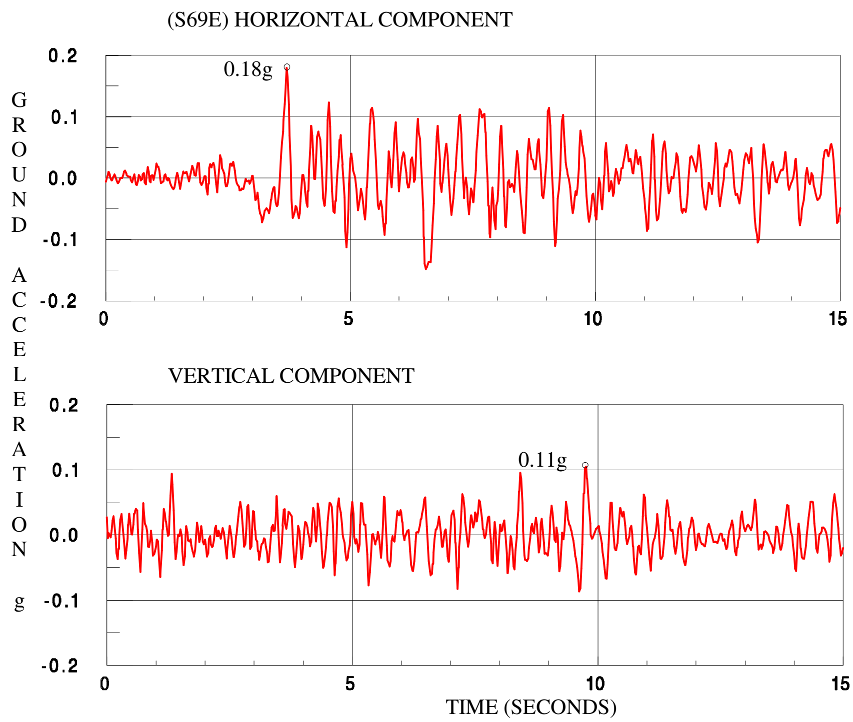

Fig. 2. Ground motion recorded at Taft Lincoln School Tunnel, California (taken from Ref. [7]).

3. The Interface Element

The geometry of the interface element used to model the discontinuities in the system is shown in Figure (3). The function of the element is to allow large relative displacements to occur between adjacent blocks and to permit the transfer of shear stresses across the interfaces. Detailed mathematical features of the element, together with simplified constitutive relations to define its behaviour are given below, while additional information on the entire three-dimensional version of the element can be found in Ref. [8]. Although the thickness (t) of the interface element is often taken as zero [14, 15], in this work it has been taken greater than zero (but small), in order to prevent adjacent nodes coalescing into each other.

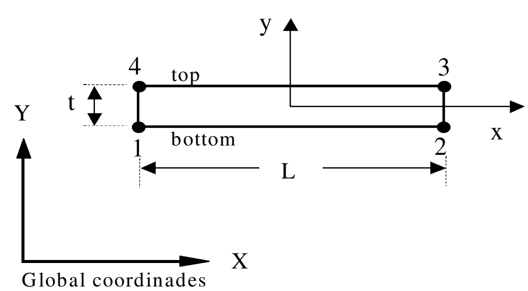

Fig. 3. Geometry of the interface element.

With reference to Figure (3), the displacements at any point within the element can be expressed as

$$\{u_i\} = [B_i]\{q_i\} \qquad (1)$$

where $\{u_i\}$ is the vector of displacement components, and $\{q_i\}$ the vector of nodal displacements. Matrix $[B_i]$ contains the interpolation functions of the element given by

$$B_i = \frac{1}{4}(1 \pm x_i)(1 \pm y_i) \qquad (2)$$

The relative displacements between the top and the bottom of the element can then be computed as

$$\{u_i\} = \left[[B_i]\{q_i\}_{top} - [B_i]\{q_i\}_{bottom}\right] = \\ = [N]\{q\} \tag{3}$$

Thus,

$$[N] = \begin{bmatrix} -B_1 & 0 & -B_2 & 0 & B_3 & 0 & B_4 & 0 \\ 0 & -B_1 & 0 & -B_2 & 0 & B_3 & 0 & B_4 \end{bmatrix} \tag{4}$$

and

$$\{q\}^T = \{u_1 \quad v_1 \quad u_2 \quad v_2 \quad u_3 \quad v_3 \quad u_4 \quad v_4\} \tag{5}$$

On minimising the potential energy of the element, we can show that its stiffness matrix, $[K']$, in the local coordinates is given by

$$[K'] = \iint [N]^T [k][N] \, dx \, dy \tag{6}$$

where [k] is the material property matrix containing joint stiffnesses per unit length along the normal and shear directions.

$[K']$ can now be transformed into its global counterpart [K] through

$$[K] = [H]^T [K'][H] \tag{7}$$

where [H] is the transformation matrix between the local and global coordinate axes.

3.1. Interface Behaviour

The interface problem is complex and inherently non-linear. The development of improved models of material behaviour is made possible by the increased sophistication of numerical methods of stress analysis. Adequate determination of constitutive laws to define the behaviour at the interfaces is important, and a number of publications have provided details of non-linear elastic and elasto-plastic constitutive laws that can be used. In the simple procedure adopted here, only the shear and normal stiffness of the interface element are defined, and therefore the material property matrix in Eq. (6) takes the form

$$[k] = \begin{bmatrix} k_s & 0 \\ 0 & k_n \end{bmatrix} \tag{8}$$

in which k_s and k_n denote unit joint stiffnesses along the shear and normal directions respectively. This definition of [k] assumes that the shear and normal modes are uncoupled, and ignores the effects of shear dilatancy which was considered to be negligible in this case because of the type of joint [16].

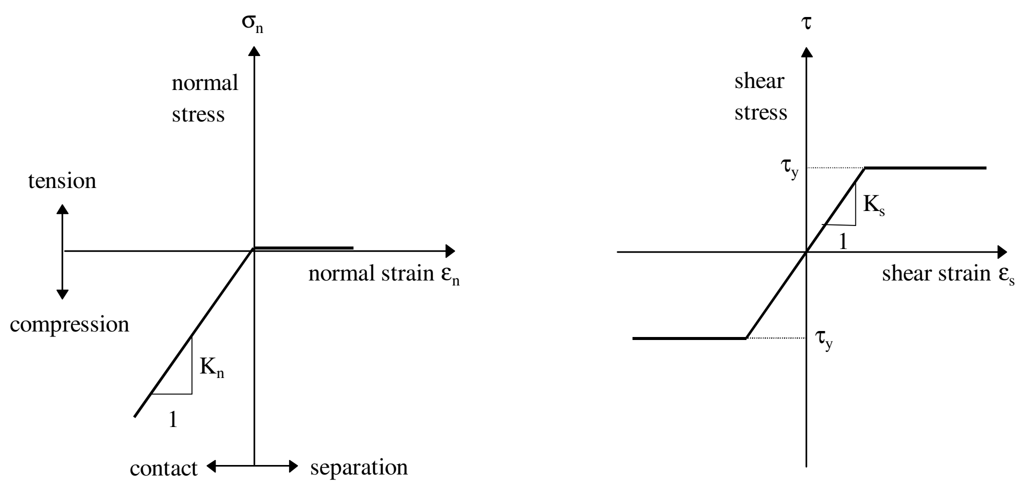

Fig. 4. Constitutive relations of the interface element.

The behaviour of the joint material is both complex and non-linear. Figure (4) shows the idealised constitutive relations of the joint element in the normal and tangential direction to the joint that can be adopted for the sake of simplicity. The joint behaviour can be characterised as elastic-perfectly plastic, and incapable of withstanding any tensile stress in the direction normal to the bed joint. In relating stress to deformation in the direction normal to the joint, two distinct stages are defined (Fig. 4):

1. eparation, which occurs when the normal strain is less than or equal to zero; the joint cannot now sustain any tensile stress in the normal direction. During separation, both normal and shear stiffnesses of the interface element are set equal to zero; consequently shear or direct stress cannot be transmitted across the joint.
2. Contact, which is restored when the normal strain returns to the value at which separation occurred.

The tangential stress-strain relationship is assumed to be elastic-perfectly plastic, based on the Mohr-Coulomb yield criterion:

$$\tau_y = c + \sigma_n \tan\phi \tag{9}$$

in which c and ϕ denote cohesion and the friction angle respectively. Within the range of $\tau<\tau_y$, sliding does not occur and the behaviour is elastic. If shear stress reaches its yield value, τ_y, slippage is assumed to occur. During slippage, joint stiffness in the normal direction is assumed to remain constant, but a reduced (or residual) shear stiffness is allocated to the joint. The non-linear behaviour of the joints can be treated by assigning the joint properties corresponding to the level of stress obtained from the last load step in a step-by-step loading procedure.

Many factors may influence the joint parameters, and these should be taken into account before assigning the relevant properties to the joints. In particular, the value of k_n will depend on the contact area ratio between the two joint surfaces and the relevant properties of the joint filling material, if present. The tensile strength of joints is usually considered to be negligible. The value of k_s will depend on the roughness of the joint surfaces, which can be determined by the distribution, amplitude, and inclination of the asperities; as well as the relevant properties of the joint filling material, if present. The value of shear strength will depend on the friction along the joint, the cohesion due to interlocking, and the strength of the filling material, if present. Thus, the filling material present in the joint may have a decisive effect on all the three parameters mentioned above. Moisture in a joint may also influence all three parameters, indirectly, through the influence on the filling material properties, or directly, by altering the frictional strength of an unfilled joint.

In general, the factors influencing the values of the joint parameters are difficult to quantify for a given joint. Even if relevant data for each factor could be measured, a decision on the relative influence of each factor has to be made in order to obtain the parameters required. Therefore, the direct measurement of the individual joint parameters is necessary in order to obtain realistic values. The joint stiffness concept is relatively new, and no values are to be found in reports and publications about joint parameters, apart from some data in the results of direct shear tests performed on individual specimens with joints. These values, however, can only be used for the analysis of the particular structure under consideration, since the wide range of possible joint conditions indicates the likelihood of extremely different response to the applied load for joints of different classifications and characteristics. In the present study, the following properties were assumed for the joint material: normal stiffness k_n=468 ksf/ft, shear stiffness k_s=195 ksf/ft, residual stiffness k_r=117 ksf/ft, cohesion c=1.5 ksf, and angle of friction ϕ=30°. The type of joints used in this investigation, and the material properties assigned to these, may not be representative of the real system. Additional research is needed to obtain realistic values of material properties that should be assigned to the joints for a more representative analysis.

4. Method of Analysis and Cases Considered

The following four cases were considered for the analysis of the dam-foundation-reservoir system responses: Case (1), monolithic dam on rigid foundation; Case (2), monolithic dam on flexible foundation; Case (3), dam, built in blocks with construction joints, on rigid foundation; and Case (4), dam, built in blocks with construction joints, on flexible foundation. Dam-foundation interaction is generally affected by the state of the interface stresses at the dam-foundation interface, which represent a measure of the interaction between the two solid aspects. In order to investigate the effect of imperfect bonding between the dam and the foundation rock on the overall response of the structure, two additional sub-cases were considered: (4a), dam-foundation interaction is considered using joint elements with restricted properties to model the dam-foundation interface (i.e. sliding and/or separation possible - imperfect bonding); and (4b), without using joint elements to model the dam-foundation interaction (i.e. perfect bonding is assumed at the interface).

A general-purpose transient dynamic finite element computer program named TRANDYN, capable of non-linear response analysis of discontinuous structures subjected to earthquake excitations, has been developed and used in the present study. In order to illustrate the versatility of this computer program, the following four types of analysis were performed for the Pine Flat dam-reservoir system: (i) linear elastic (small deformation) analysis; (ii) elastic (large deformation) analysis; (iii) elasto-viscoplastic (small deformation) analysis using the central difference method, and (iv) elasto-plastic (small deformation) analysis using a combined explicit-implicit algorithm. In cases (i) and (ii), both explicit and implicit time integration schemes were used.

The following mechanical properties were assigned to the constituent materials of the dam-foundation system: the mass concrete of the dam is assumed to be a homogeneous, isotropic, linear elastic solid with a modulus of elasticity E_d=468000 ksf, unit weight=155 lb/ft^3, and a Poisson's ratio ν=0.2. The dam is also assumed to rest on a linearly elastic foundation block with a Poisson ratio ν=1/3, and unit weight =165 lb/ft^3. The foundation modulus of elasticity, E_f, is varied such that E_f/E_d=1.0, 0.8 and 0.5.

Material non-linearity of the solid aspects of the system (i.e. dam concrete and foundation rock) has also been considered, and both material and geometrically nonlinear effects have been included in the analysis in order to study their significance to the overall response of the system. Although the program developed has several capabilities and a choice of four different yield criteria that can be employed, the general Mohr-Coulomb criterion applicable to concrete, rock and soil problems, has been used in the present analysis. Two of the most widely known time integration methods for solving structural dynamics problems by the finite element method were used to predict the dam's seismic response. In particular, the Newmark's average acceleration method (i.e., α=1/4 and δ=1/2), and the explicit central difference scheme, have been employed in order to investigate their performance to the analysis of this complex problem.

Critical time step

The time step length that has to be adopted for the non-linear analysis of the system, depends on the following criteria: for the explicit central difference integration scheme for the dynamic equilibrium equations, the critical time step length Δt_c is defined by the highest circular mesh frequency, ω_{max}, and it is limited by the expression $\Delta t_c \leq 2/\omega_{max}$. This severe time step limit is required for stability and it ensures accuracy in practically all the modes of vibration. To avoid the exact evaluation of the highest finite element mesh frequency approximate expressions are usually employed. The most common form is [17]:

$$\Delta t_c \leq \mu L \left(\frac{\rho(1+\nu)(1-2\nu)}{E(1-\nu)} \right)^{\frac{1}{2}} \qquad (10)$$

where μ is a coefficient dependent on the type of element employed and L is the smallest length between any two nodes in the finite element mesh. When an elasto-viscoplastic model is adopted, care must be taken not to exceed the critical time step for the Euler scheme in evaluating the viscoplastic strains.

For an accurate solution, based on implicit time integration, the time step length Δt should be limited to 1/100 of the fundamental (largest) period. In the present study, for a stable and accurate solution based on explicit time integration, the time step length Δt was found to be 2×10^{-4} sec (using Eq. 10). For an accurate solution based on implicit time integration the time step length Δt is taken as 2×10^{-3} seconds. This is because the fundamental period of the dam-reservoir system was found to be equal to 0.317 sec.

Damping matrix

Energy-loss mechanisms that are operative in the dynamic analysis of dam-foundation-reservoir systems are difficult to represent precisely. In this analysis, Rayleigh damping was assumed and thus the global damping matrix was computed from using the expression

$$[C] = a[M] + b[K] \tag{11}$$

where [M] and [K] are the combined system mass and stiffness matrices and a, b are proportionality constants selected to control the damping ratios of the lowest and highest modes expected to contribute significantly to the response. These can be calculated from the relation

$$a + b\omega_i^2 = 2\omega_i\xi_i \tag{12}$$

where ξ_i is the damping ratio and ω_i the i^{th} natural frequency of the system. The main disadvantage of the Rayleigh damping method is that the higher modes are considerably more damped than the lower modes, and that the damping can be controlled at only two modes of vibration. In the present study damping is controlled only at the fundamental mode of vibration.

For cases (1) and (2) of the study, where the dam is assumed to be monolithic, the values of the damping ratios used in the analysis are the same as those in Ref. [11, 12] for direct comparison of the results. In particular, for case (1) damping ratios of 4% for the full, and 5% for the empty reservoir state were considered, while for case (2), the values of 12.2% and 12.6% were used respectively. For cases (3) and (4), however, where the dam is assumed to be built-in-blocks with construction joints, the value of damping ratio used had to be assumed. This is because the permanent system of joints represents an energy-loss system with damping characteristics that are difficult to assess from analysis and experimental procedures are required. In the absence of experimental results, a damping ratio of 10% and 15% has been assumed for cases (3) and (4) respectively.

5. Results and Discussion

Response results were obtained for a wide range of system parameters, but only a small set of the results is presented here, in order to highlight the important effects. For a monolithic dam on rigid or flexible foundation, with the reservoir empty or full, results obtained under the assumption of linear elastic behaviour of both concrete and foundation rock are in good agreement with those of earlier investigations [11, 12]. Results from the non-linear analysis of the system show, that, even for the fairly mild earthquake excitation used (with a maximum ground acceleration of 0.18g in the horizontal component), several non-linear phenomena occur in the system. The most important of these is the sliding and opening of the vertical contraction joints and those at the dam-foundation interface.

By comparing the linear elastic with the elasto-plastic solutions given in Figure (5), we can conclude that non-linear material behaviour occurs in the monolithic system of Case (2), with a full reservoir condition. The displacement response history from the non-linear analysis of the system, slightly diverges from that predicted from the linear calculation, with the indication of a permanent plastic deformation of the order of 0.3 inches after 10 seconds of Taft ground motion. When the reservoir is empty, however, the stresses in the system remain within their elastic limits.

In case the dam is assumed to be constructed in blocks, with vertical contraction joints, energy dissipation due to sliding and/or opening of the joints between adjacent concrete monoliths reduces the degree of non-linearity and gives response results that are identical to those of the linear elastic case.

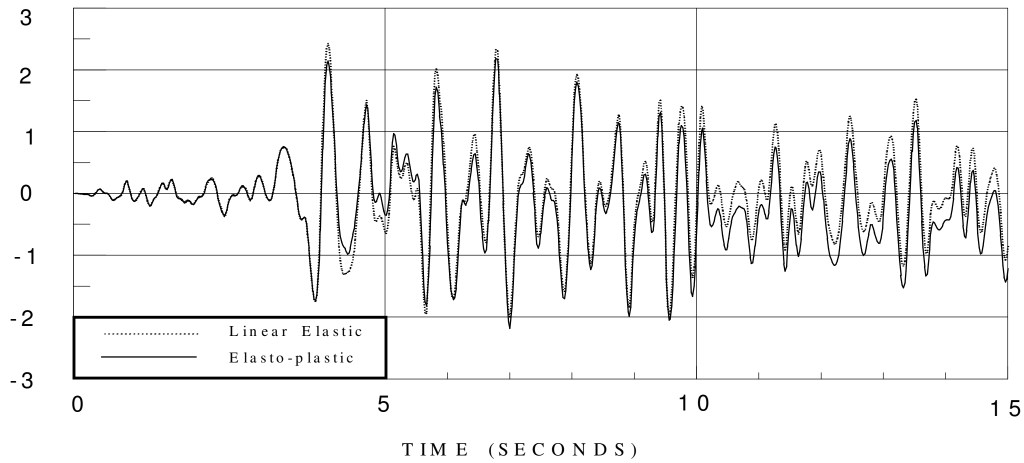

Fig. 5. Horizontal displacement response (inches) of dam crest to Taft ground motion. Comparison of linear elastic and elasto-plastic material responses using the combined explicit-implicit algorithm. Monolithic dam on flexible foundation with full reservoir -Case (2).

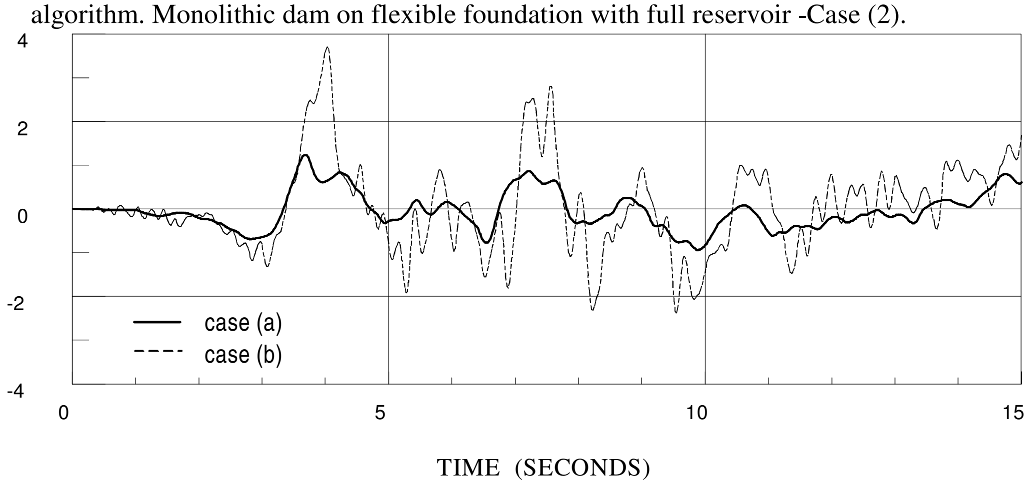

Fig. 6. Horizontal displacement response (inches) of dam crest to Taft ground motion. Dam built-in-blocks, with construction joints, on flexible foundation, with full reservoir – Case (4).

By comparing the results obtained for the system analysed under Cases (2) and (4) (see Figures 5 and 6 respectively), we can conclude that the response of the system is significantly affected by the behaviour at the interfaces between the contacting materials. Insertion of interface elements to model the dam-foundation interface (Case 4a), results in a reduced response (compared to Case 4b) since the joints act as 'energy absorptive' devices (i.e. when the interface joints fail, either in tension or shear, seismic loads cannot induce the entire dam into motion).

6. Conclusions

Results presented in this paper demonstrate that the response of concrete gravity dam-reservoir systems, is significantly affected by the behaviour at the interfaces of the various discontinuities present in real systems. By means of the inclusion of discontinuities with particular measurable properties, the proposed method of analysis conforms better to actual conditions than do other methods in which a continuum is assumed. The computer program developed for the analysis of the system can be eventually used as an effective tool for the design of new dams and for the seismic safety evaluation of existing dams. However, constitutive and strength properties of multiaxially loaded mass concrete under dynamic conditions need to be properly defined before any non-linear response analysis procedure can reliably predict the extent of damage that may be caused during an intense ground motion and evaluate its implications to the safety of concrete dams. In addition, reliable data on the properties of joint materials have to be evaluated from experiments under dynamic loading conditions; such data are essential for an accurate prediction of system response.

The results of this study call attention to current dam design criteria based on linear models of dams. Developing efficient techniques to handle all forms of non-linearities present in a real system is the most pressing need before the earthquake behaviour of concrete gravity dams can be fully understood.

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