COUPLED FINITE ELEMENT ANALYSIS OF A DAM-RESERVOIR SYSTEM UNDER DYNAMIC LOA*DING*

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ABSTRACT

This investigation presents a coupled analysis of a dam-reservoir problem which includes all aspects of fluid-structure interaction (*class I coupling*) and soil-pore fluid–structure interaction (*class II coupling*) under earthquake excitations using the finite element method. The analysis involves the compressibility of water, the flexibility of the dam, the earthquake excitation, the structural damping and the material nonlinearity on the response.

An efficient computer program in FORTRAN is developed for this analysis from the original computer code named MIXDYN. The new software for predicting and analyzing the coupled behaviour is established using *the pressure formulation* for modelling of fluid and the *u-p formulation* for modelling of soil-pore fluid behaviours. Also, the program prepared is provided with post-processing routines to plot the original and deformed meshes of the problem.

A staggered partitioned solution technique for coupled field problems is implemented and used in the computer code. This scheme is incorporated in terms of sequential execution of single-field analyzers. The Drucker Prager model is used to simulate the behaviour of the soil and concrete. Implicit-Implicit Newmark's scheme with a predictor-corrector algorithm is employed for time integration of the equations of motion. The capability and efficiency of the model are found to be very useful when applied on a dam-reservoir system.

الخلاصة :

يقدم هذا البحث تحليلاً مزدوجاً لمسألة سد مع خزان و الذي يتضمن كافة اوجه التفاعل المتبادل مابين المائع و المادة الصلبة (ازدواج من الصنف الاول) فضلاً عن تفاعل متبادل ما بين التربة و مائع المسام و المادة الصلبة (ازدواج من الصنف الثاني) تحت تأثيرات هزة ارضية باستخدام طريقة العناصر المحددة. يتضمن التحليل انضغاطية الماء و قابلية تكيف السد و الهزة الارضية و الاخماد الانشائي و لا خطية المادة من خلال تأثيرها على الاستجابة.

لقد تم تطوير برنامج حاسوبي باستخدام لغة فورتران خاصة بهذا التحليل من برنامج MIXDYN الحاسوبي. لقد تم التحقق من البرامجيات المستحدثة من خلال توقع و تحليل التصرف المزدوج باستخدام معادلة الضغط لتمثيل المائع و معادلة u-p لتمثيل التربة مع مائع المسام. لقد تم تحضير خوارزميات ما بعد المعالجة لرسم التقسيمات الاصلية و ما بعد التشوه للمسألة.
لقد تم تضمين و استخدام طريقة حل التقسيم المتعرج لحل مسائل الوسط المزدوج في البرنامج الحاسوبي المطور. لقد تم اضافة هذه الطريقة بدلالة التنفيذ المتناوب لتحليلات الاوساط المنفردة. لقد تم استخدام طريقة دروكر بريكر لتمثيل تصرف التربة و الخرسانة.
كما تم استخدام طريقة نيومارك الضمنية – الضمنية مع خوارزمية التوقع– التصحيح للتكامل الزمني لمعادلات الحركة. لقد وجد ان امكانية و كفاءة النموذج عالية جداً عند تطبيقها على نظام سد مع خزان.

KEYWORDS
coupled analysis, dam-reservoir, finite element method, u-p formulation.

INTRODUCTION

The dynamic analysis of soil-fluid-structure interaction includes all aspects of both fluid and solid mechanics (i.e., fluid-structure interaction (*class I coupling*) and soil-pore-fluid interaction (*class II coupling*)). In a fluid-phase, the viscosity of the fluid, the magnitude of the gradient of the velocity field throughout the flow and whether the fluid is (*compressible or incompressible*), depending on whether density variations are large or small, play a key role in choosing the kind of formulation to be used. However, in the solid-phase, the time scale and the solver algorithm to be used depend on the loading rate and the permeability of the porous medium. *Traditionally, fluid problems can be classified into two categories: (i) non-flow problems, such as impounded water in a reservoir, tank, etc. and (ii) flow problems, such as free surface flow, flow around an airfoil etc...*

In this study, the former type of problems is considered. The second class of problems to be considered here lies between the undrained and drained extremes where dynamic loading is applied and transient pore fluid motion is significant (Simon et al. [36]). The undrained analysis is possible when relatively rapid loads are applied and permeability is low, i.e., where the load rate is greater than the pore fluid diffusion rate. Otherwise, drained analysis is possible for situations with a relatively slow loading and high permeability, i.e., where the load rate is less than the pore fluid diffusion rate. *Consequently, the problem to be solved in this research is a triple interaction: fluid-structure-soil pore fluid.*

Fluid-Structure Interaction (Class I Coupling):

The dynamic interaction between an elastic structure and a fluid has been the subject of intensive investigations in recent years, e.g. ([10], [11], [21], [22], [32] and [35]). Since analytical solutions procedures are available only for very simple problems, numerical approaches, which can be formulated in the time or frequency domains, had to be employed, e.g. ([15], [17], [19], [27], [33] and [34]).

Many researchers have attempted to derive variational functionals for different classes of fluid-structure interaction problems. The size of the coupled fluid-structure interaction problem is generally large. That is why attempts were made to reduce the problem size in different ways

Out of all the works done in the area of developing a finite element method for fluid-structure interaction problems, two approaches predominate. The *first approach* is the displacement-based method where the displacements are the nodal variables in both the fluid and the structure. Bathe and Hahn [3], Belytschko [4], Belytschko and Kennedy ([5] and [6]), Chopra et al [9] and Nitikitpaiboon and Bathe [24] described the method in detail. This approach is not well suited for problems with large fluid displacements. Another difficulty with this method is that special care must be taken to prevent zero-energy rotational modes from arising. In the *second approach*, the potential-based method, displacements remain the nodal variables in the structure, while velocity potentials or pressures are the unknowns in the fluid. Everstine [12], Everstine and others ([13] and [14]), Hamdi et al. [16], Morand and Ohayon [23], Ohayon and Valid [25], Olson and Bathe ([26] and [28]) and Zienkiewicz and others ([38], [39] and [42]) demonstrated techniques for formulating finite elements using potential-based methods. In all these works, only a linearized version of the problem has been considered.

Several finite element studies have considered the gravity and free surface effects along with the fluid structure interaction.

SOIL-PORE FLUID-STRUCTURE INTERACTION (CLASS II COUPLING):

Soils are multiphase materials exhibiting a strong mechanical coupling between the solid skeleton and the fluid phase. This coupling can be particularly strong in the case of saturated soils of low permeability and under fast transient or dynamic loading, where the pore pressure plays a significant role. The first successful attempt to develop a model for solid skeleton-pore fluid interaction was due to Biot [7 and 8] for linear elastic materials. This work was followed by further development at Swansea University, where Zienkiewicz and others ([41], [43], [44], [45], [46] and [47]) extended the theory to non-linear materials and large deformation problems.

Zienkiewicz [37] described extensively several kinds of coupled problems and their numerical solutions with some applications. The analysis of coupled soil-pore fluid interaction during an earthquake shock applied to a dam shows that the non-linear soil response causes a pore pressure build up and failure of the actual structure.

Park and Felippa [30] reviewed several developments of computational procedures for solving coupled field problems with emphasis on stabilization of partitioned analysis. It was found that the resulting matrices after semi-discretization are not symmetric. The non-symmetry in the coefficient matrices often induces *conditional stability* of partitioned solutions and, therefore, stabilization at the differential equation level before attempting to implement a partitioned solution procedure is necessary.

The behaviour of multiphase flow in deforming porous media is of interest in engineering problems such as the simultaneous flow of three immiscible fluids; e.g. gas, oil and water through a tar sand formation during the bitumen recovery process, with environmental studies, etc.. For most cases of fluid transport in soil, two or more fluid phases are present simultaneously in the pores and are separated from one another by interfaces. Li and Zienkiewicz [20] developed a numerical procedure for modelling the behaviour of porous media interacting with the flow of multiphase immiscible fluids based on Biot's theory and the principle of effective stress. The displacement of the solid, pressure and saturation of the wetting fluid were taken as primary unknowns of the model. Unconditionally stable direct and staggered solution procedures were used for the time domain while the numerical solution of the coupled finite element equations were set with u-p_w-S_w form and discreteized by Galerkin's method.

Fluid Formulations:

Various formulations are generally used for inviscid fluid fields. The most common formulations for non-flow problems are the *displacement, displacement potential, velocity potential and pressure formulations.* In finite element analysis, the displacement formulation gives rise to two or more variables compared with one in the other formulations (Paul, [31]). Therefore, in this work, the *pressure formulation* is used because it results in fewer unknowns.

PRESSURE FORMULATION:

Governing Equation of Motion:

The equation governing fluid motion is the well-known wave Equation (Joseph [18]):

$$\nabla^2 P + \xi' \nabla^2 \dot{P} = \ddot{P}/c^2 \quad \text{(Linearized-Navier-Stokes Equation)} \qquad (1)$$

where: $\xi' = 4\mu'/3\rho_f c^2$, μ' = the dynamic viscosity of fluid and $c^2 = K/\rho$.

For an inviscid fluid, Equation (1) reduces to:

$$\nabla^2 P = \ddot{P}/c^2 \qquad (2)$$

Boundary Conditions:

(i) At moving boundaries (at interface with solid) where the fluid has a normal acceleration, $\ddot{u}_n$, **n** being the direction of the unit normal to the boundary, the pressure gradient can be expressed as:

$$\partial P/\partial \mathbf{n} = -\rho_f \ddot{u}_n \qquad (3)$$

At fixed boundaries; $\partial P/\partial \mathbf{n} = 0$.

(ii) At a free surface with surface waves (considering only primary waves):

$$P = \rho_f g u_y \quad \text{or} \quad \partial P/\partial y = \ddot{p}/g \qquad (4)$$

At a free surface without surface waves: P = 0.

(iii) At radiating boundaries, the condition for no reflection of pressure waves can be expressed as:

$$\partial P/\partial \mathbf{n} = -\dot{P}/c \qquad (5)$$

where: **n** = the direction of the unit normal at the radiating boundary.

Fluid Isoparametric Element:

The fluid domain is usually represented by finite elements in Cartesian coordinates. The number of nodes may be variable (4-9) in two dimensions, with one degree of freedom per node inside the fluid domain. This degree of freedom is the value of the pressure P at the nodes. At the free surface, the element has an extra translational degree of freedom to accommodate the free surface motion. This element enforces the continuity (equilibrium in solids) equation along the mesh domain. The applied forces represent the water pressure (unit volume per second) at this node. The positive pressure is in-pressure and the negative one is out-pressure. For global equilibrium, the in-pressure must equal to the out-pressure. At the boundaries, only the normal velocity may be specified because the tangential velocity does not affect the pressure. The nodal equilibrium is satisfied if the sum of the water pressure increments at the node is equal to the total applied pressure.

Fluid-Structure Interaction (Pressure Formulation):

The structure and fluid are together idealized as a two dimensional system subjected to support excitations both in the horizontal and vertical directions and the equations of motion can be expressed, after spatial discretization, by two sets of second order coupled differential equations. The fluid can be modeled using any of the various formulations mentioned before. However, in this study, only the *pressure formulation* is used in which the coupled fluid-structure equations can be expressed as:

$$\underline{M}_s \underline{\ddot{u}} + \underline{C}_s \underline{\dot{u}} + \underline{K}_s \underline{u} = \underline{f}_s - \underline{M}_s \underline{\ddot{d}} + \underline{L}\,\underline{P} \quad \ldots(6)$$

$$\underline{M}_f \underline{\ddot{P}} + \underline{C}_f \underline{\dot{P}} + \underline{K}_f \underline{P} = \underline{f}_f - \rho_f \underline{L}^T(\underline{\ddot{u}} + \underline{\ddot{d}}) \quad \ldots(7)$$

where:

$$\underline{M}_s = \int_\Omega \underline{N}_u^T \rho \underline{N}_u d\Omega \quad \ldots(8a)$$

$$\underline{C}_s = \alpha\, \underline{M}_s + \beta\, \underline{K}_s \ \ldots \text{(Rayleigh Damping)} \ldots(8b)$$

$$\underline{K}_s = \int_\Omega \underline{B}^T \underline{D}_T \underline{B}.d\Omega \quad \ldots(8c)$$

$$\underline{f}_s = \int_{\Gamma u} \underline{N}_u^T \underline{t}\, d\Gamma + \int_\Omega \underline{N}_u^T \rho\, \underline{b}\, d\Omega + \int_\Omega \underline{B}^T \underline{D}^T d\underline{\varepsilon}^o\, d\Omega \quad \ldots(8d)$$

$$\underline{L} = \int_\Omega \alpha_c\, \underline{B}^T \underline{\delta}\, \underline{N}_p\, d\Omega \ldots(8e)$$

$$(\underline{M}_f)_{ij} = \int_{\Gamma F} \underline{N}_{pi}\, 1/g\ \underline{N}_{pj}\, d\Gamma + \int_{\Omega F} \underline{N}_{pi}^T\, 1/c^2\, \underline{N}_{pj}\, d\Omega \quad \ldots(8f)$$

$$(\underline{C}_f)_{ij} = \int_{\Gamma R} \underline{N}_{pi}^T\, 1/c^2\ \underline{N}_{pj}\, d\Gamma \quad \ldots(8g)$$

$$(\underline{K}_f)_{ij} = \int_{\Omega F} (\nabla \underline{N}_{pi})^T (\nabla \underline{N}_{pj})\, d\Omega \quad \ldots(8h)$$

$$(\underline{L}^T)_{ij} = \int_{\Gamma I} \underline{N}_{ui}^T\, n\, \underline{N}_{pj}\, d\Gamma \ldots(8i)$$

and $\underline{N}_p$ and $\underline{N}_u$ are the shape functions used for pore pressure and solid skeleton, respectively. α and β are Rayleigh damping constants, Ω is the domain, Γ is the boundary surface, $\underline{B}$ is the strain-displacement matrix and t is the surface traction.

Pore Fluid–Solid Interaction (u-p Formulation):

When the seepage velocity relative to the solid skeleton is small compared with the motion of the solid skeleton or if the permeability is low, the relative acceleration of the fluid with respect to the solid can be neglected. With this approximation (i.e., neglecting the $\ddot{w}$ term) and replacing the unknown w with the pressure P, the equilibrium equation of the fluid can be rewritten as (Paul [31]):

$$\underline{\dot{w}} = -k \underline{\nabla} P + k\rho\underline{b} - k\rho\, \underline{\ddot{u}} \quad \ldots(9)$$

which can be used to eliminate w from the continuity equation. Upon discretization, it is possible to write:

$$\underline{u} = \underline{N}_u\, \underline{u} \quad \ldots(10)$$

$$\underline{P} = \underline{N}_p\, P \quad \ldots(11)$$

and using the standard Gelerkin method, the resulting equations can be expressed as:

$$\underline{M}_s \underline{\ddot{u}} + \underline{C}_s \underline{\dot{u}} + \underline{K}_s \underline{u} = \underline{f}_s - \underline{M}_s \underline{\ddot{d}} + \underline{L}\, \underline{P} \quad \ldots(12)$$

$$\underline{C}_P \underline{\dot{P}} + \underline{K}_P \underline{P} = \underline{f}_P - \underline{L}^T \underline{\dot{u}} + \underline{\hat{M}}\underline{\ddot{u}} \quad \ldots(13)$$

where:

$$\underline{M}_s = \int_\Omega \underline{N}_u^T \rho \underline{N}_u d\Omega \ldots(14a)$$

$$\underline{C}_s = \alpha\, \underline{M}_s + \beta\, \underline{K}_s \ldots \text{(Rayleigh Damping)} \ldots(14b)$$

$$\underline{K}_s = \int_\Omega \underline{B}^T (\underline{D}_T + \alpha_c^2 \delta.Q.\delta^T)\underline{B}.d\Omega \qquad (14c)$$

$$\underline{f}_s = \int_{\Gamma u} \underline{N}_u^T \underline{t}\, d\Gamma + \int_\Omega \underline{N}_u^T \rho\, \underline{b}\, d\Omega + \int_\Omega \underline{B}^T\, \underline{D}^T\, d\underline{\varepsilon}^o\, d\Omega \qquad (14d)$$

$$\underline{L} = \int_\Omega \alpha_c\, \underline{B}^T\, \underline{\delta}\, \underline{N}_p\, d\Omega \qquad (14e)$$

$$\underline{C}_p = \int_\Omega \underline{N}_P^T\, 1/Q\, \underline{N}_p\, d\Omega \qquad (14f)$$

$$\underline{K}_p = \int_\Omega (\nabla \underline{N}_P)^T\, k\, (\nabla\, \underline{N}_p)\, d\Omega \qquad (14g)$$

$$\underline{f}_p = \int_{\Gamma p} \underline{N}_P^T\, P\, d\Gamma + \int_\Omega (\nabla \underline{N}_P)^T\, k\, \rho_f\, \underline{b}\, d\Omega \qquad (14h)$$

$$\underline{L}^T = \int_\Omega \alpha_c\, \underline{N}_p{}^T\, \underline{\delta}\, \underline{B}\, d\Omega \qquad (14i)$$

$$\underline{\hat{M}} = \int_\Omega (\nabla \underline{N}_P)^T\, k\, \rho_f\, \underline{N}_u\, d\Omega \qquad (14j)$$

In this study, this formulation is implemented and used in the computer program.

Staggered Solution for Coupled–Field Problems:

Many engineering problems involve two or three fields, such as soil-structure interaction, fluid-structure interaction, soil-fluid-structure interaction, etc…Such problems are generally partitioned into well defined fields which are distinct in behaviour, material model or solution technique. These fields are linked continuously together through two-way interaction with other fields. Each field may be coupled (totally or partially) with all the other participating fields or with only few of them (at interfaces via the contact boundaries only).

The concept of staggered solution can be organized in terms of sequential execution of single-field analyzers. This leads in the nodal based implicit-explicit partitioning of time stepping, to a complete solution of the explicit scheme independently of the implicit one and then using the results to progress with the implicit partition. This approach offers several advantages over the field elimination and simultaneous solution approaches as follows:
(1) Completely different methodologies could be used in each part of the coupled system. (2) Independently developed codes dealing effectively with single systems could be combined. (3) Parallel computation with its inherent advantages could be used. (4) The time step size restrictions can be excluded from consideration. (5) In systems of the same physics, efficient iterative solvers could easily be developed. (6) This approach permits decoupling of high frequency and low frequency components of a single system, so that an alternative time marching algorithm can be used in each part (Zienkiewicz and Taylor [40]). Finally, (7) it turns out to be unconditionally stable with a predicted (approximate) value of u at $t+\Delta \mathbf{t}$ and with suitable integration formulae for each set of equations of motion (Li and Zienkiewicz [20]. *Therefore, in the present study, the staggered partitioned solution scheme for a coupled field problem as shown in Figure (1) is implemented and used in the computer code.*

NUMERICAL ALGORITHM:

Based on the procedures and equations described, a coupled dynamic finite element algorithm is developed from the original uncoupled code MIXDYN (Owen and Hinton [29]) by the name DCAPII (Al-Nu'aimy [1]). It is also an extension of the computer code DCAPI developed by Al-Shereffi [2] under the supervision of al-Damluji. DCAPII includes classes I and II couplings presented above.

Numerical Example: Dam-Reservoir System:

The Koyna concrete gravity dam-reservoir system (India) is analyzed with all the aspects of fluid-structure interaction (*class I coupling*) and soil-pore fluid interaction (*class II coupling*). The shape and dimensions of this dam-reservoir system are shown in Figure (1). The material properties of the system are taken from Paul [31] and listed in Table (1). The analysis involves the compressibility of water, the flexibility of the dam, the structural damping, the earthquake excitations and structural nonlinearity on the response. *This problem*

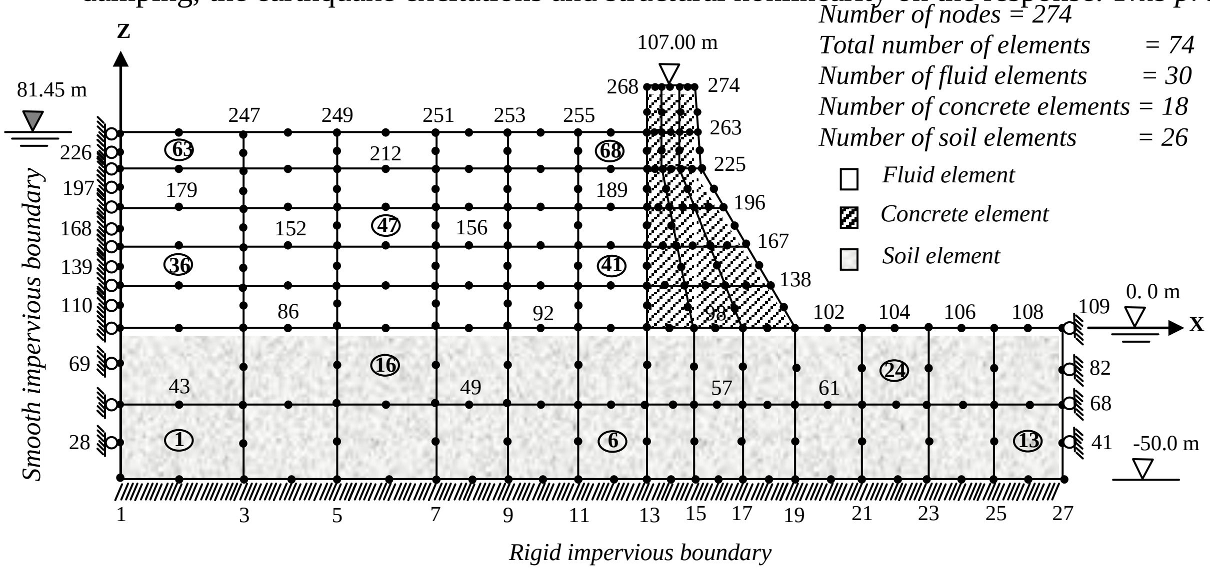


Figure (1): Koyna dam-reservoir system (India).

Table (1): Dimensions and properties of Koyna dam-reservoir system (from Paul, [31]).

Material and Property	*Value*
1. Dam (concrete)	
Height of dam above foundation (m).	107.00
Depth of reservoir (m).	81.45
Young's modulus of concrete, E_c (T/m^2)	3164000.0
Poisson's ratio of concrete, υ_c	0.20
Density of concrete, ρ_c (T/m^3)	2.690
2. Soil (rock)	
Young's modulus of soil, E (T/m^2)	1800000.0
Poisson's ratio of soil, υ_s	0.20
Density of soil, ρ_s (T/m^3)	1.830
3. Fluid (water)	
Compressibility of water, c (m/sec)	1439.0
Density of water, ρ_f (T/m^3)	1.000

The ratio of fundamental periods of reservoir to the dam: $\gamma_T = (T_f)$ reservoir/ (T_f) dam	0.566

is solved by Paul [31] as a fluid-elastic structure interaction (i.e., with class I coupling) only. The eight node isoparametric element is adopted for both the solid and fluid phases as shown in Figure (1). The boundary conditions are as depicted in the figure.

RESULTS AND DISCUSSION:

<u>Effect of Water Compressibility:</u>

For this study, the rigid Koyna dam is subjected to a horizontal Heaviside unit base excitation. The velocity of water is taken as a measure of water compressibility ($K = c^2 \rho_f$). Figure (2) shows the pressure distribution for cases with *incompressible* and *compressible* water. It is observed that as the velocity of water is increased from 1c to 4c, the peak hydrodynamic force does not change significantly. But, there is a shift in the occurrence of the peak force. The ratio of the peak hydrodynamic pressure to the hydrostatic force is 0.15 for the compressible water (at 1c) when compared with the incompressible one. This implies that the compressibility of water has a significant effect on the distribution of pressure on the rigid dam.

EFFECT OF DAM FLEXIBILITY:

Again, the rigid Koyna dam is subjected to a horizontal Heaviside unit base excitation. The pressure distribution for several cases of dam-foundation flexibilities are shown in Figure (3). For the case of dam on a flexible foundation, as the flexibility of the dam system increases (by decreasing its modulus of elasticity), the hydrodynamic force or the pressure distribution on the face of the dam also increases. The maximum effect is obtained when both the dam and the foundation are most flexible. Conversely, when the dam is rigid, foundation flexibility is not so important.

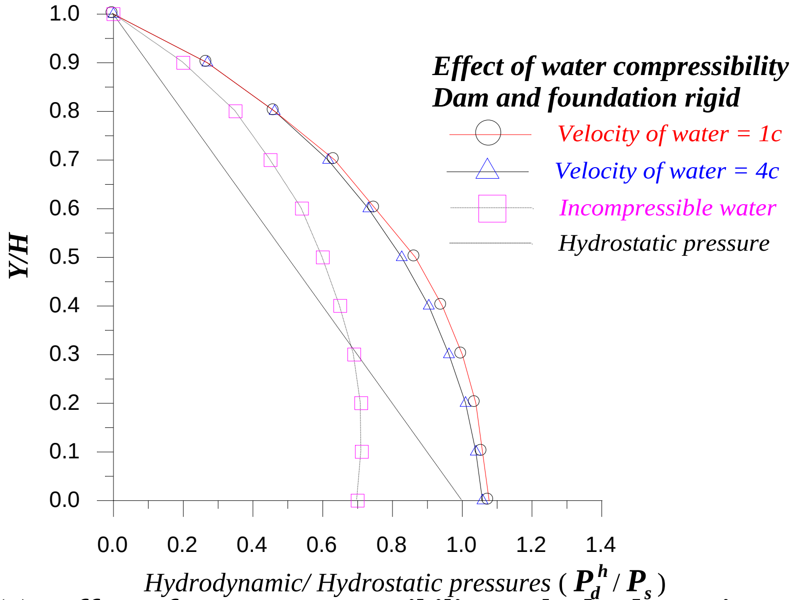


Fig (2): Effect of water compressibility on hydrodynamic pressure distribution due to a Heaviside unit base excitation.

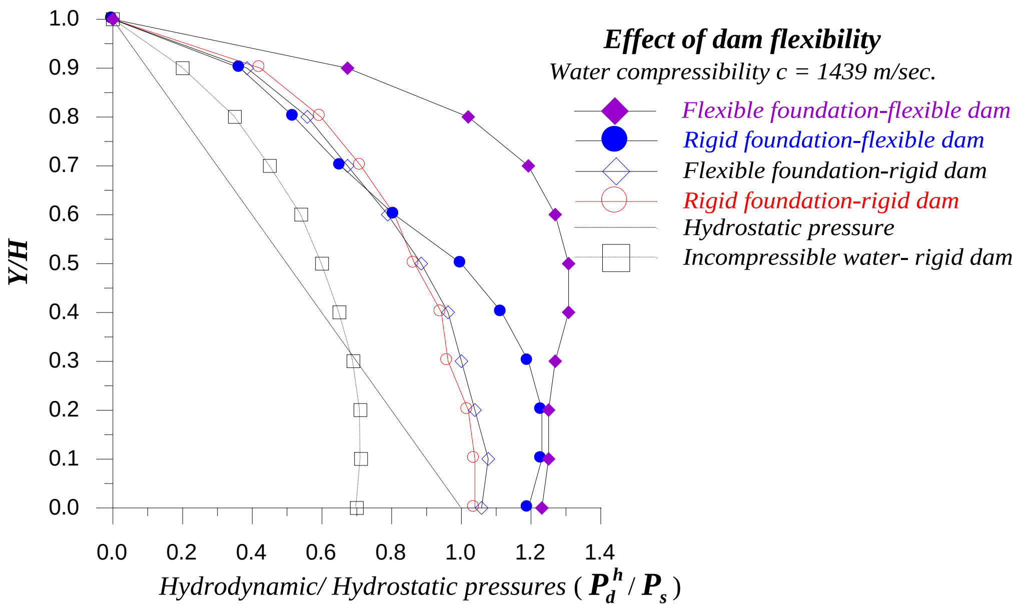


Fig (3): Effect of dam flexibility on hydrodynamic pressure distribution due to a Heaviside unit base excitation.

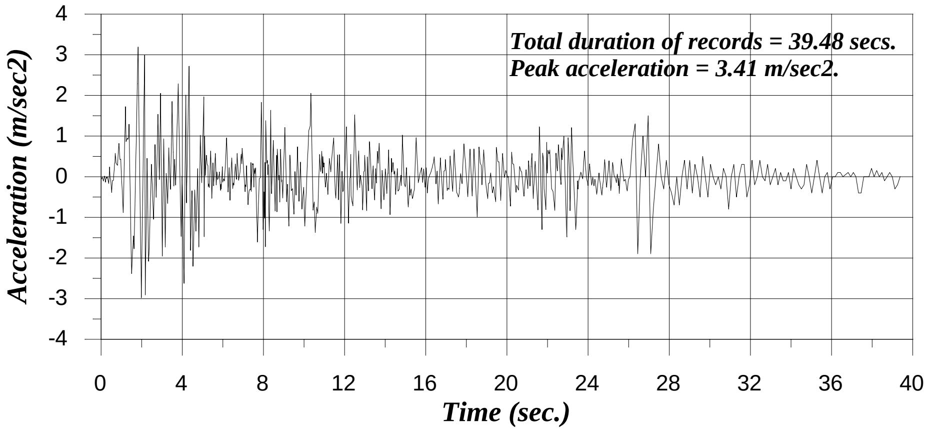
The peak hydrodynamic force is given in Table (2). This table shows that as the flexibility increases, the response also increases.

Table (2): Effect of dam flexibility on hydrodynamic pressure distribution due to a Heaviside unit base excitation.

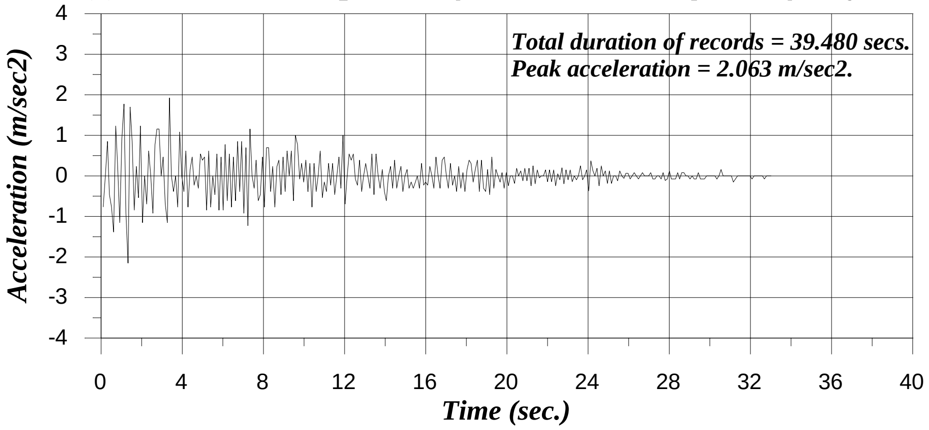
Response Description	***Fexible Foundation***	***Rigid Foundation***			
	Ē = 1E	Ē = 1E	Ē = 2E	Ē = 4E	Ē = ∞
P^h_d/P_s	1.360	--	0.505	0.500	0.395

EFFECT OF EARTHQUAKE EXCITATION:

For this study, three different earthquakes each with different ground motion characteristics are considered as shown Figure (4a-e).

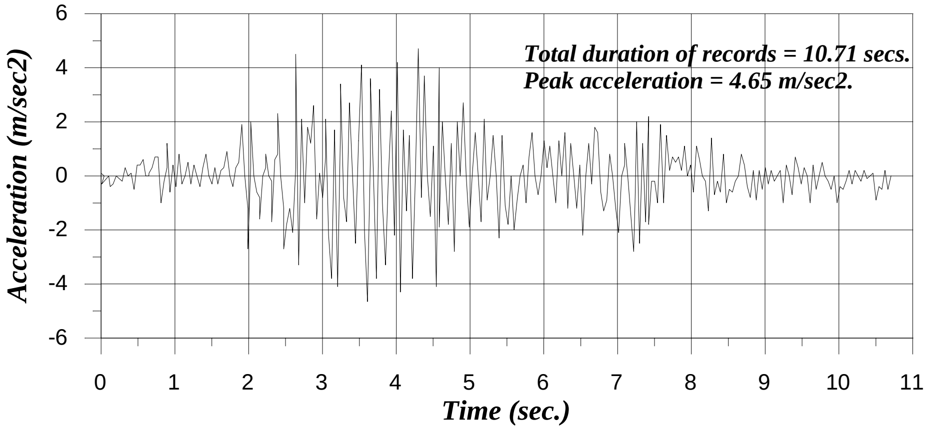


(a) Transverse component of El-Centro earthquake of May 1940.

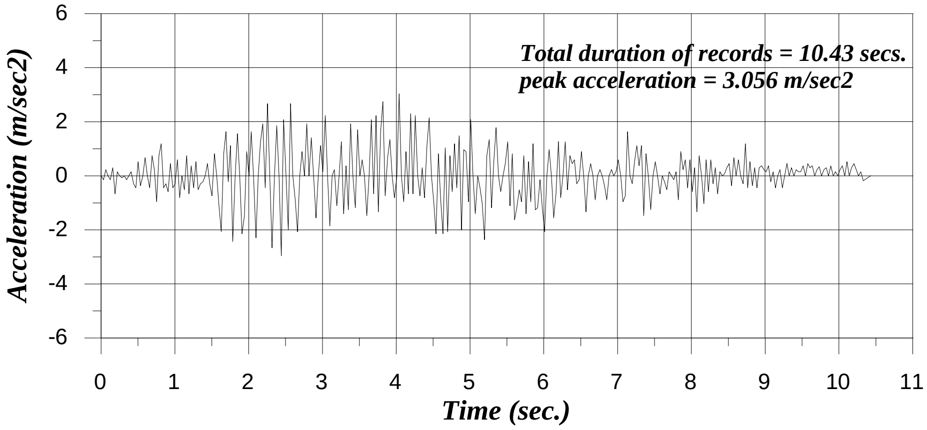


(b)Vertical component of El- Centro earthquake of May 1940.

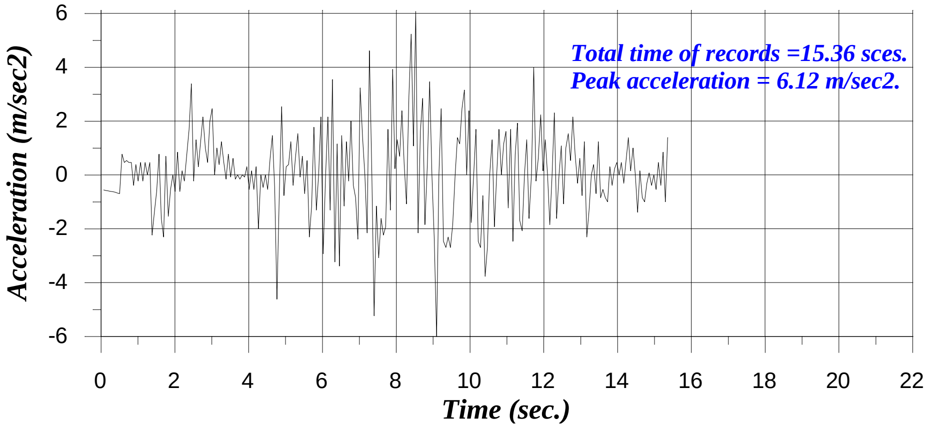
Fig (4): Earthquakes (from Paul, [31]).



(c) Transverse component of Koyna earthquake of Dec., 1967.



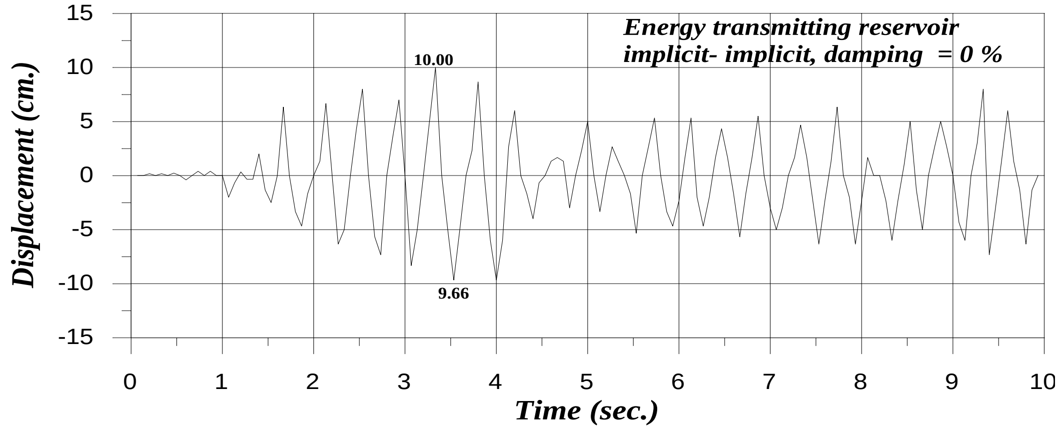
(d) Vertical component of Koyna earthquake of Dec., 1967.



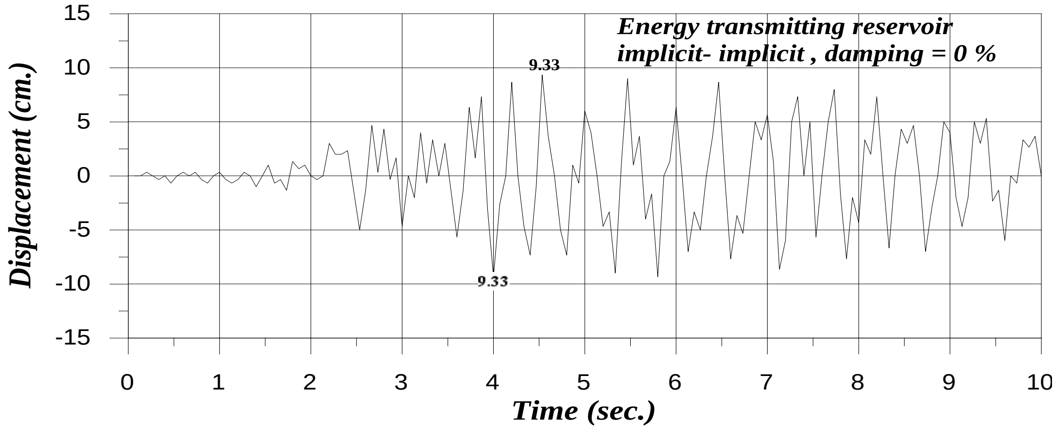
(e) San-Fernando earthquake N18E component Feb., 1971.

Figure (4): Continued

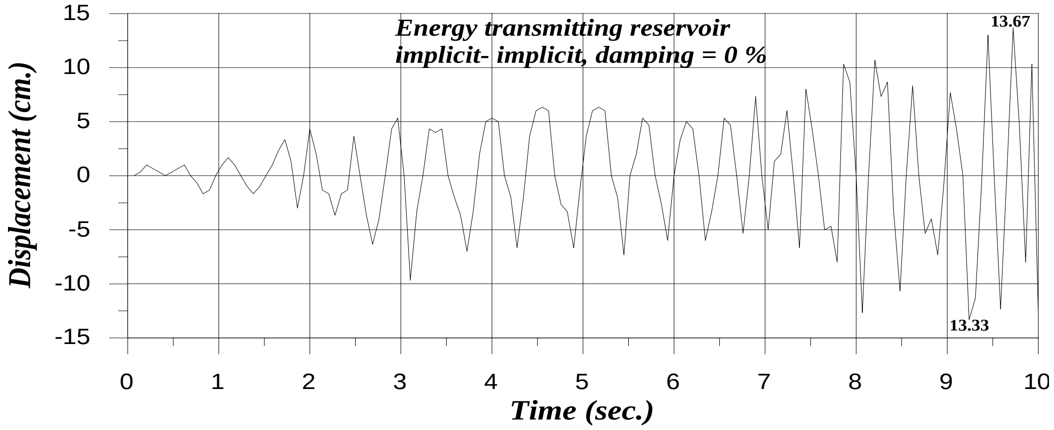
The undamped response (0 % damping) of the crest displacement (element 72), the stress at the dam heel (element 33) and the hydrodynamic pressure at the base of the dam (elements 33, 34 and 35) when subjected to both transverse and vertical components of either the El-Centro or Koyna or San Fernando earthquakes, simultaneously are shown in Figures (5), (6) and (7), respectively. It is noticed that the response characteristics are very much dependent on the type of earthquake excitation. This is because of the strong interaction between the impounded water and the foundation when the vertical component of the earthquake is considered in comparison with that due to only the transverse component of earthquake. The peak responses of the dam for various earthquake (transverse and vertical) excitations are summarized in Table (3).



(a) El-Centro earthquake.



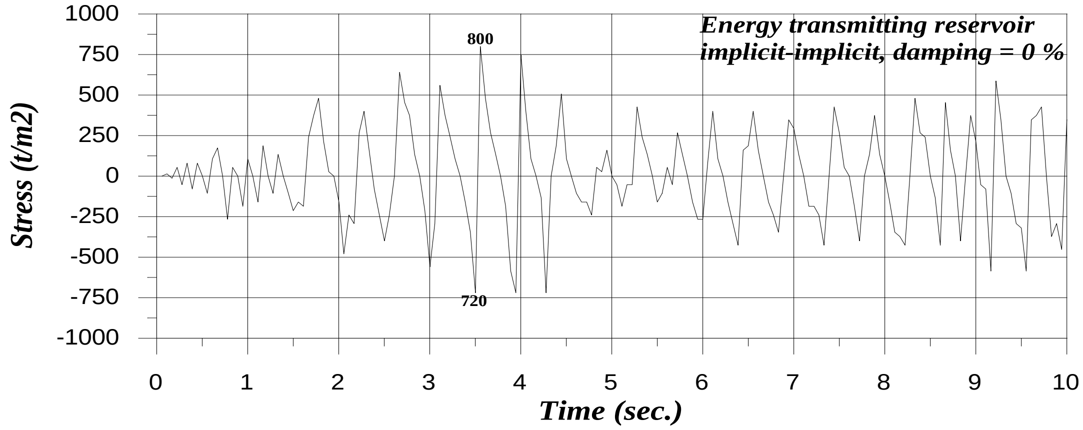
(b) Koyna earthquake.



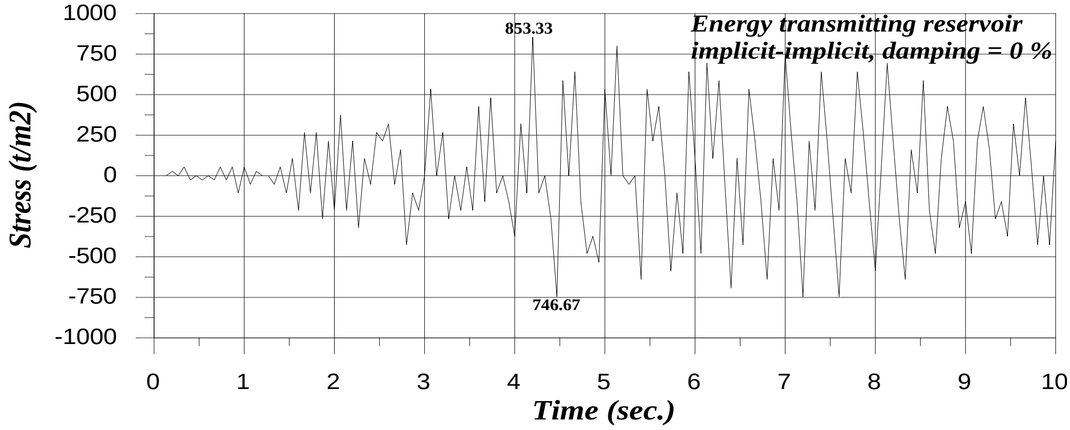
(c) San-Fernando earthquake.

Note : 1cm = 10 mm.

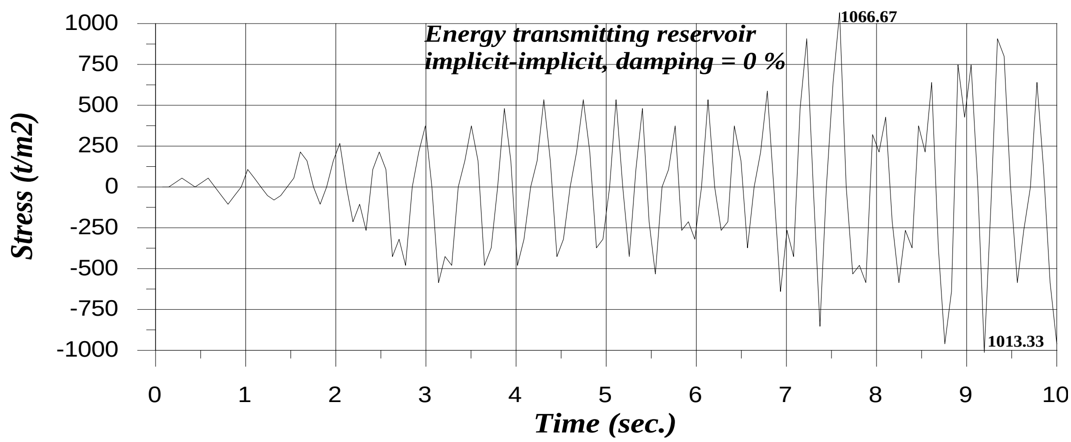
Figure (5): Response of dam crest displacement when subjected to various earthquakes (transverse and vertical) excitations.



(a) El-Centro earthquake.



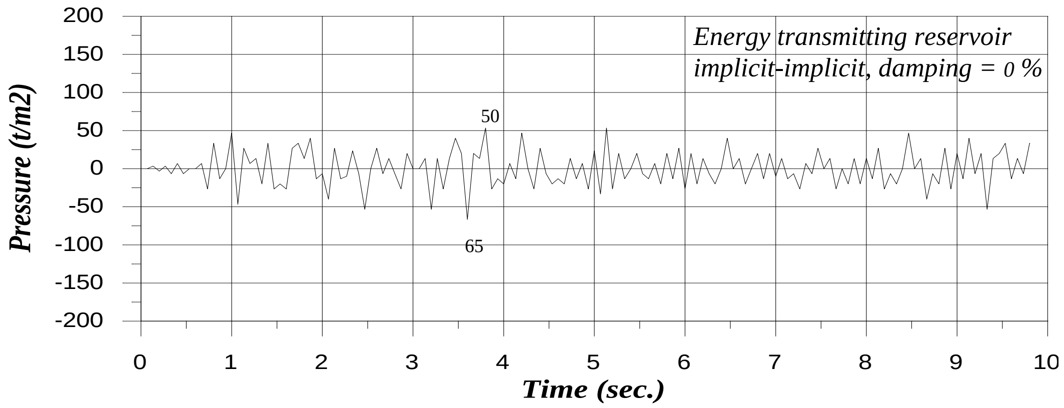
(b) Koyna earthquake.



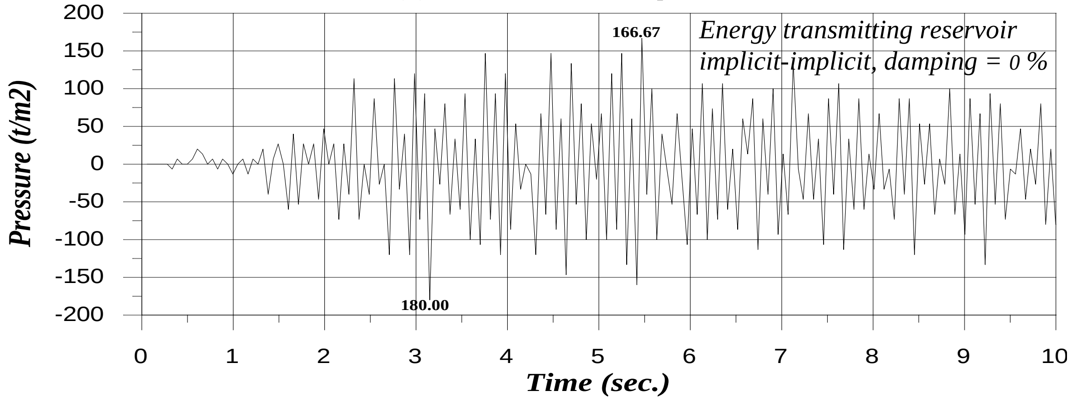
(c) San-Fernando earthquake.

Note: 1 t/m2 = 9.81 kN/m2

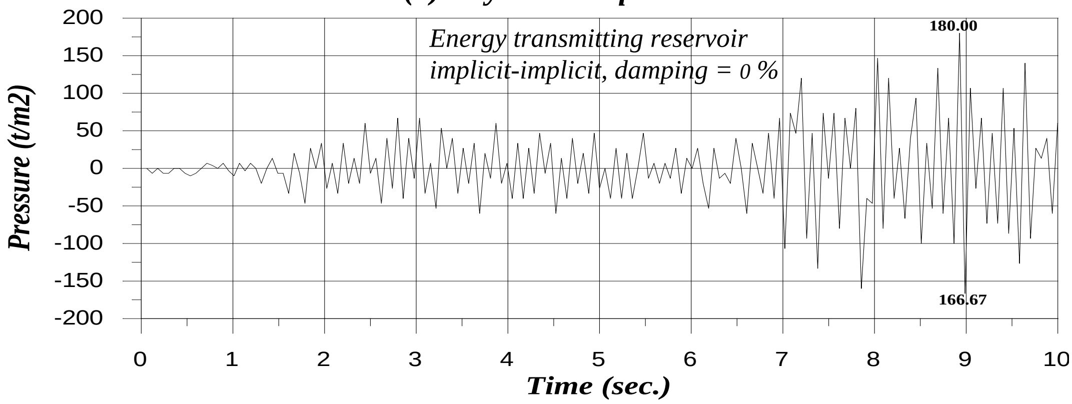
Figure (6): Response of normal stress at the dam heel when subjected to various earthquake (transverse and vertical) excitations.



(a) El-Centro earthquake.



(b) Koyna earthquake.



(c) San-Fernando earthquake.

Note: 1 t/m2 = 9.81 kN/m2

Figure (7): Response of pressure at the dam base when subjected to various earthquakes (transverse and vertical) excitations.

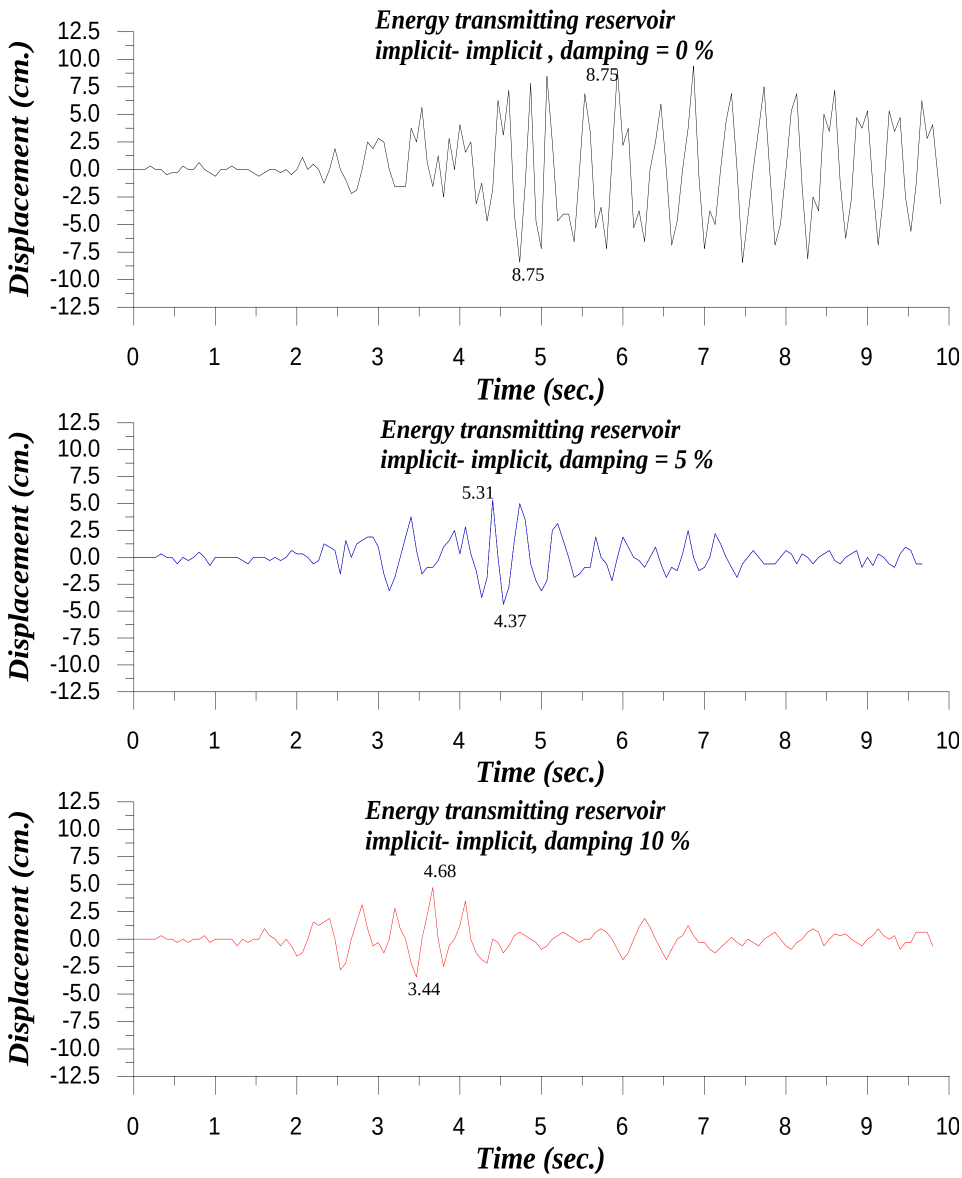
Table (3): Comparison of peak responses of dam for various earthquakes (transverse and vertical) excitations.

Response Description	*El-Centro Earthquake*	*Koyna Earthquake*	*San-Fernando Earthquake*
Dam crest displacement (cm)	*10.00 at 3.33 sec.*	*9.33 at 4.53 sec.*	*13.67 at 9.72 sec.*
	-9.66 at 3.53 sec.	*-9.33 at 4.00 sec.*	*-13.33 at 9.24 sec.*
Stress at the dam heel (T/m^2)	*800 at 3.55 sec.*	*853.33 at 4.20 sec.*	*1066.67 at 7.59 sec.*
	-720 at 3.50 sec.	*-746.67 at 4.47 sec.*	*-1013.33 at 9.19 sec.*
Pressure at the	*50 at 3.85 sec.*	*166.67 at 5.47 sec.*	*180.00 at 8.92 sec.*

Dam base (T/m^2)	*-65 at 3.60 sec.*	*-180.00 at 3.15 sec.*	*-166.67 at 8.98 sec.*

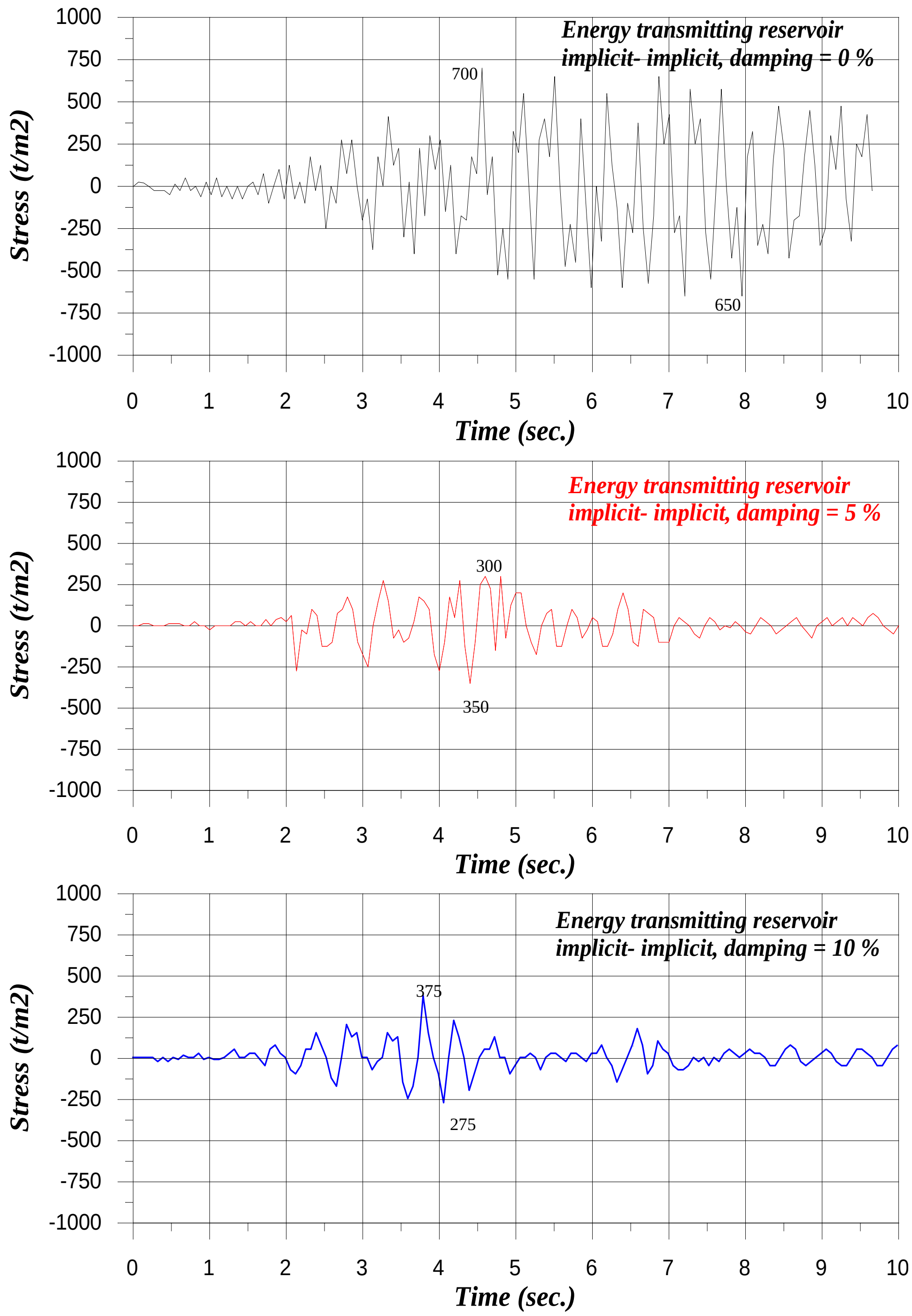
EFFECT OF STRUCTURAL DAMPING:

The responses of Koyna dam when subjected to the transverse component of the Koyna earthquake for 0%, 5% and 10% damping are shown in Figures (8), (9) and (10), respectively. It is observed that the effect of structural damping is significant and, therefore, estimation of damping in the evaluation of the response should be made carefully. The peak responses of the dam for different damping ratios are given in Table (4).



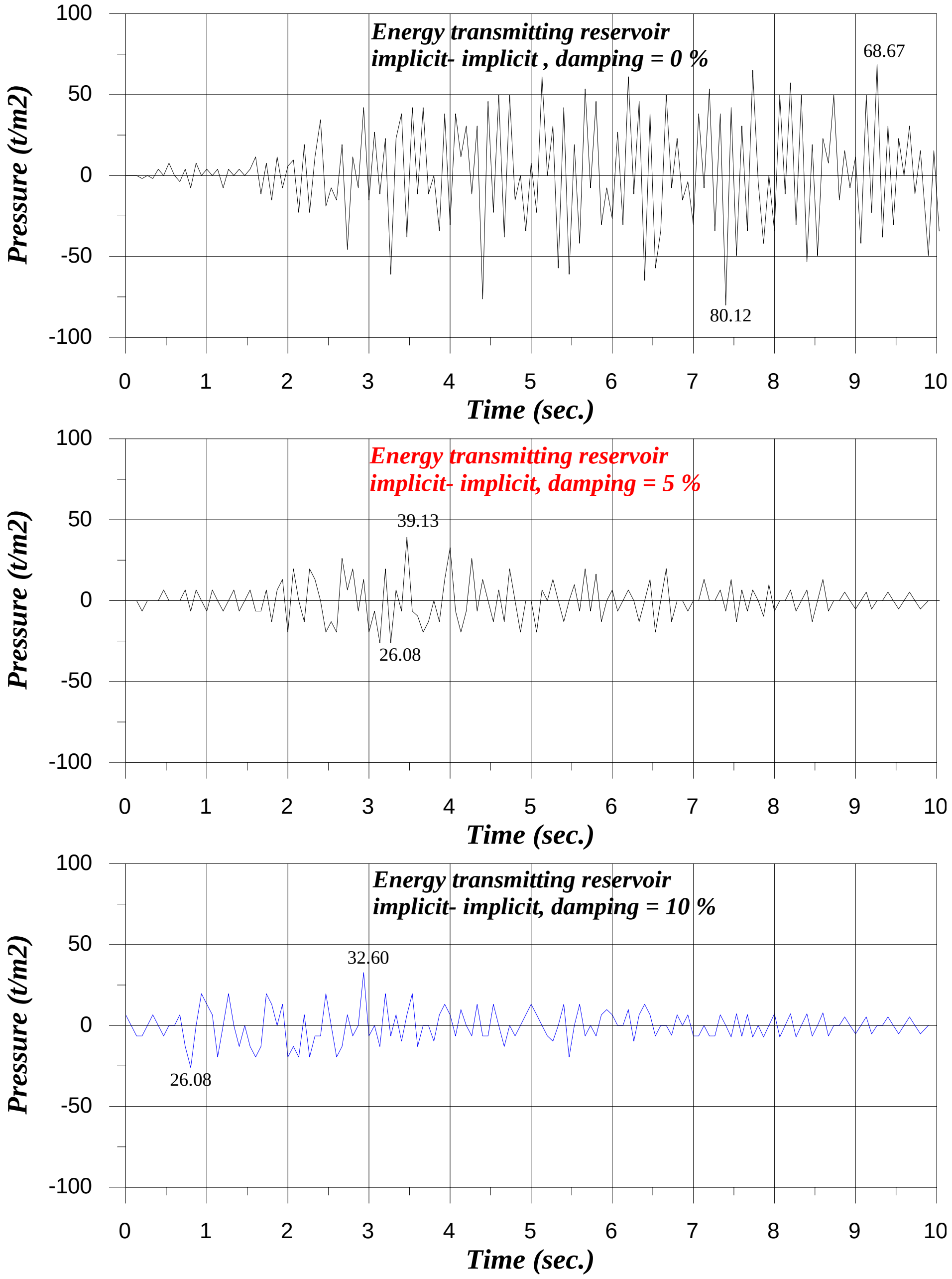
Note : 1cm = 10 mm.

Figure (8): Response of dam crest displacement when subjected to transverse component of Koyna earthquake.



Note: 1 t/m2 = 9.81 kN/m2

Fig (9): Response of normal stress at the dam heel when subjected to a transverse component of Koyna earthquake.



Note: 1 t/m2 = 9.81 kN/m2

Fig (10): Response of pressure at the dam base when subjected To a transverse component of Koyna earthquake.

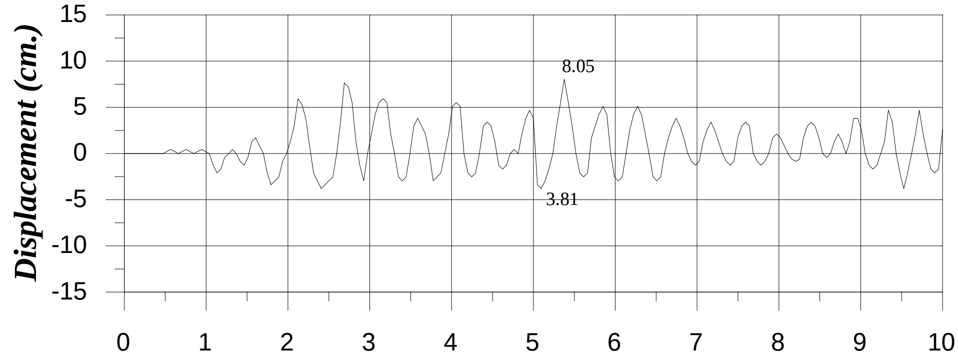
Table (4): Effect of structural damping on the response of dam when the transverse component of Koyna earthquake is applied.

Response Description	**0 % damping**	**5 % damping**	**10 % damping**
Dam crest displacement (cm)	8.75 at 5.8 sec.	5.31 at 4.4 sec.	4.68 at 3.66 sec.
	-8.75 at 4.8 sec.	-4.37 at 4.5 sec.	-3.44 at 3.46 sec.
Stress at the dam heel (T/m^2)	700 at 4.55 sec.	300 at 4.6 sec.	375 at 3.8 sec.
	-650 at 7.95 sec.	-350 at 4.4 sec.	-275 at 4.07 sec.

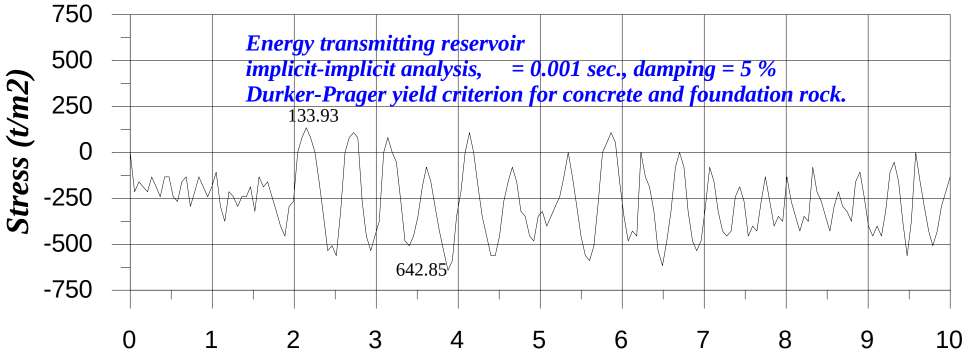
Pressure at the Dam base (T/m^2)	*68.67 at 9.26 sec.*	*39.13 at 3.5 sec.*	*32.60 at 2.93 sec.*
	-80.12 at 7.4 sec.	*-26.08 at 3.26 sec.*	*-26.08 at 0.8 sec.*

EFFECT OF MATERIAL NONLINEARITY:

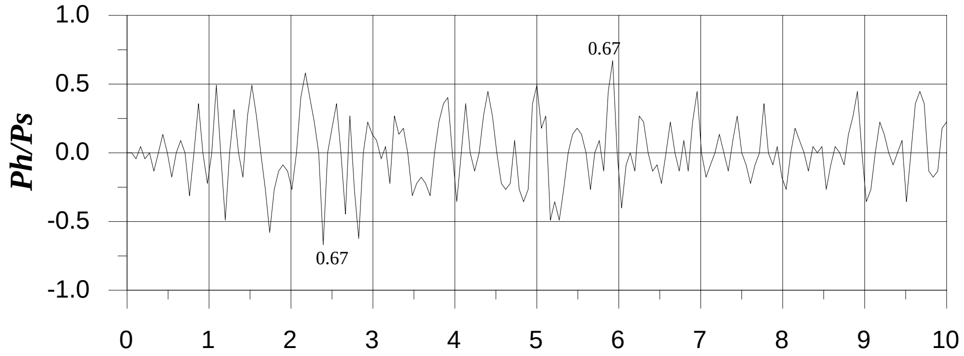
The nonlinear response of the Koyna dam when subjected to transverse and vertical Koyna earthquake components is shown in Figure (11). The concrete and foundation-rock-soil are represented by the Drucker-Parger yield criterion (Owen and Hinton [29]). The yield stress values of the concrete and the foundation rock are taken equal to be 323.94 T/m2 and 257.75 T/m2, respectively (1T/m2=9.81kN/m2). It is found that the effect of material nonlinearity is significant and when the nonlinearity in the dam structure is considered, the response reduces appreciably.



(a) Response of dam crest displacement.



(b) Response of normal stress at the dam heel.



(c) Response of hydrodynamic force on dam.

Note : 1cm = 10 mm and,

1 t/m2 = 9.81 kN/m2.

Fig (11): Nonlinear response of dam when subjected to transverse

and vertical components of Koyna earthquake.

CONCLUSIONS:

From this investigation, the following points can be drawn:

- The computer code developed is found to be very useful and can be used for a wide range of applications in many soil-fluid-structure interaction problems.
- The partitioned solution scheme in which the fluid, structure and soil-pore fluid is integrated in a staggered fashion is found to be very efficient.
- Two-phase materials subjected to dynamic loadings can be formulated with approximate numerical solutions and acceptable degrees of accuracy.
- Analysis of the actual behavior of constructions during dynamic loading exemplify the fact that the soil-structure interaction and, in the case of hydraulic structures, the fluid-structure interaction are phenomena which may have an important influence on the structural seismic response.
- The compressibility of water has a significant effect on the distribution of pressure on the rigid dam.
- As the flexibility of the dam system increases, the pressure distribution on the face of the dam also increases. The maximum effect is obtained when both the dam and the foundation are most flexible. Conversely, when the dam is rigid, foundation flexibility is not so important.
- The response characteristics of the dam-reservoir are very much dependent on the type of earthquake excitation, structural damping and material nonlinearity.

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LIST OF SYMBOLS:

$\underline{b}$ = Displacement of fluid relative to the solid skeleton.
B = Strain –displacement matrix.
c = Speed of sound.
$\underline{C}_s$ = Rayleigh damping matrix.
$\underline{C}_f$ =Compressibility matrix.
C_{ijkl} = Components of the elasticity tensor.
D_t = Constitutive matrix.
E = Modulus of elasticity specified by Table (1).
$\bar{E}$ = Adopted modulus of elasticity in analysis.
g = Gravitational acceleration.

G = Shear modulus.
$\bar{h}(L)$ = Heaviside step function defined as 1 for $l > 0$ and 0 at $l \le 0$.
H = Dam height.
$\hat{H}$ = Positive shape hardening scalar function of σ_{ij} and q_n .
$\hat{i}$, $\hat{j}$, $\hat{k}$ = Unit vectors in x, y and z directions, respectively.
k = Permeability coefficient.
K = Bulk modulus .
$\underline{K}_s$ = Stiffness matrix.
$\underline{K}_f$ =Flow matrix.
K_f = Bulk modulus of the fluid.
K_S= Bulk modulus of the solid phase.
K_T = Total bulk modulus of the solid skeleton.
$\underline{L}$ = Coupling matrix .
L = Loading index.
L_{ij} = Loading direction.
$\underline{M}_s$ = Solid skelton mass matrix.
$\underline{M}_f$ =Fluid mass matrix.
n = Porosity.
n = the direction of the unit normal at the radiating boundary.
$\underline{N}_p$ = Shape functions for pore pressure.
$\underline{N}_u$ = Shape functions for solid skeleton displacements.
P_f = Mass density.
P = Pressure above the hydrostatic value.
P_d^h = Hydrodynamic pressure.
P_s = Static pressure.
$1/Q = \left[\frac{n}{K_f} + \frac{\alpha_c - n}{K_s}\right]$
t = Surface traction.
T = Time.
$\underline{u}$ = Solid phase translation.
$\dot{u}_X$, $\dot{u}_y$ and $\dot{u}_Z$ = Velocity of solid phase components in x, y and z directions, respectively.
$\underline{\dot{w}}$ = Fluid velocity.
Y = Rise from dam base.
α and β are Rayleigh damping constants.
α_c= 1-K_T/ K_S.
$\underline{\varepsilon}^o$= Autogenous strains.
ε_{ij} = Strains due to stresses and the superscripts.
Γ = Boundary surface.
μ' = The dynamic viscosity of fluid.
ρ = Solid phase density.
ρ_f = Fluid density.
σ_{ij} = Stress tensor.
Ω = The domain.
$\langle\ \rangle$ = Macauley brackets defining the operation $\langle L\rangle = \bar{h}(L)L$.
A superposed dot indicates the rate.