INFLUENCE OF MATERIAL NONLINEARITY OF FOUNDATION IN THE DAM FOUNDATION INTERACTION ANALYSIS

A. Burman[*1], D. Chakravarty[2] and D. Maity[3]

[1] *Research Scholar, Dept. of Civil Engineering , Indian Institute of Technology, Guwahati, India*
[2] *Assoc. Professor, Dept. of Miningl Engineering , Indian Institute of Technology, Kharagpur, India*
[3] *Assoc. Professor, Dept. of Civil Engineering , Indian Institute of Technology, Kharagpur, India*
Email: avijit@iitg.ernet.in, dmaity@civil.iitkgp.ernet.in

ABSTRACT :

The effects of material nonlinearity of the foundation domain have been investigated in connection with the iterative analysis procedures of soil-structure interaction problems. For the purpose of analysis, a concrete gravity dam has been selected as the structure. An algorithm for the iterative analysis of SSI problems has been proposed and verified with the published results. The effects of different impedance ratios (the ratio of foundation stiffness and dam stiffness) have been investigated in some details in the present paper. The dam has been modeled as linear, elastic material. The foundation material has been modeled as both linear, elastic material and nonlinear elastic material. The widely known Duncan-Chang soil model has been chosen to represent the nonlinear material behavior of soil/rock foundation. The comparison of results of the interaction analysis for the linear foundation with those of the nonlinear foundation has been presented. The results clearly display pronounced effects of displacements and stresses for the structure compared to the case when both dam and foundation are assumed to be composed of linear, elastic materials. For the time integration of the dynamic equation of motion, the implicit-explicit Newmark scheme has been adopted. The results clearly emphasize the need for the consideration of a proper nonlinear treatment of the foundation material during the soil-structure interaction analysis, especially when the structure is subjected to severe ground motion during any earthquake.

KEYWORDS: Koyna gravity dam, earthquake excitation, dam-foundation interaction, material nonlinearity, nonlinear elastic Duncan-Chang model.

1. INTRODUCTION

A concrete gravity dam is a heavy and large structure with a complicated geometry and complex behavior under seismic condition. Considerable research has been conducted in the area of dynamic interaction analysis of dam-foundation coupled system in recent years. The majority of the work is restricted to linear analysis. The primary reason for this limitation is mainly the fact that most of the analyses have been carried out in frequency domain. It has long been recognized that the time domain method are the most suitable methods for nonlinear finite element analysis. Wilson (2002) devised a method for doing nonlinear soil structure interaction analysis using fast nonlinear analysis (FNA) method based on FEM exclusively.

It is necessary to consider the infinite nature of foundation domain during dam foundation interaction analysis. The first local boundary was proposed by Lysmer and Kuhlemeyer (1969). It is known as absorbing boundary since it places viscous dashpots at the boundary to absorb the energy of traveling waves.

Felippa and Park (1980) discussed, in detail, staggered solution procedure for solution of for a variety of coupled field dynamic problems. Maity and Bhattacharya (2003) suggested an iterative scheme in conjunction with the staggered solution procedure for the dam-reservoir interaction problems. However, the stability of this procedure is conditional on the size of time step. If a corrective iteration at each time step is employed, where the interface boundary conditions are iteratively updated until convergence is achieved, one obtains an iterative coupling method. Within the iteration procedure at every time step, a relaxation operator may be applied to the interface boundary conditions in order to enable or speed up convergence.

In this paper, a time domain staggered solution approach with iterative scheme has been used to solve dynamic dam-foundation interaction problems. The substructure techniques, while computing the interaction forces require applying either Laplace or Fourier transforms and then evaluating the convolution integrals which are computationally very demanding and complex processes (Wolf 1985). The present iterative method avoids calculation of transform and convolution integrals. The advantage of this staggered solution procedure is that there is no need to calculate the coupled mass, damping and stiffness matrices which appear in direct coupling equations of interaction problems. The soil and structure domain has been solved sequentially and the interaction is enforced by the proposed iterative method. In order to represent the infinite nature of the foundation domain, viscous dashpots as suggested by Lysmer and Kuhlemeyer (1969) has been adopted and its performance has been observed. For characterizing the behaviors of geologic materials, a popular elastic, nonlinear model namely Duncan-Chang (1970) model has been used. Also the nonlinear response of the whole structure has been compared with that of the linear dam-linear foundation interaction analysis.

2. MODELLING OF DAM

The dam has a long body whose geometry and loading conditions do not vary in the longitudinal direction. Therefore, the dam structure can be analyzed under plane strain idealization. Therefore, the constitutive relation for elastic isotropic material can be written as

$$\{\sigma\} = [D]\{\varepsilon\} \tag{2.1}$$

In the above equation, $\{\sigma\}^T = \{\sigma_x, \sigma_y, \tau_{xy}\}$ and $\{\varepsilon\}^T = \{\varepsilon_x, \varepsilon_y, \gamma_{xy}\}$ are the vectors of stress and strain respectively , and $[D]$ is the constitutive matrix defined as

$$[D] = \frac{E}{(1+\mu)(1-2\mu)}\begin{bmatrix} (1-\mu) & \mu & 0 \\ \mu & (1-\mu) & 0 \\ 0 & 0 & \frac{(1-2\mu)}{2} \end{bmatrix} \tag{2.2}$$

for a material with elastic modulus E and Poisson's ratio μ.

3. MODELLING OF FOUNDATION

In the present work, the foundation material is assumed to be of rock in nature. A nonlinear elastic constitutive model proposed by Duncan and Chang (1970) has been chosen to simulate the stress vs. strain behavior of soil/rock which is nonlinear in nature. For this model, the tangent modulus and the Poisson's ratio is given by:

$$E_T = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1 - \sigma_3)}{2(C\cos\phi + \sigma_3\sin\phi)}\right]E_i \tag{3.1}$$

Here, R_f is the failure ratio, C and ϕ are the cohesion and angle of internal friction respectively. The major and minor principal stresses are denoted by σ_1 and σ_3 respectively. The failure ratio, R_f is defined as

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \tag{3.2}$$

However, for the present analysis, only the Young's modulus of the rock material was varied according to eqn (3.1). The Posisson's ratio was kept constant throughout the analysis. The value of failure ratio was taken to be 0.80 (Bose and Das, 1997).

4. MODELLING OF RESERVOIR

Assuming the reservoir water to be inviscid and incompressible and its motion to be of small amplitude, the governing equation for hydrodynamic pressure is as follows (Westergaard, 1933):

$$\nabla^2 p = 0 \tag{4.1}$$

Here, $\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2}$ and is called the Laplacian operator and p is the hydrodynamic pressure. The solution of Laplace equation (eq. 4.1) can be expressed in equation (4.2) with the following assumptions:

i. The bottom of the fluid domain is horizontal and rigid.

ii. The fluid-structure interface is vertical.

iii. The fluid domain extends to infinity and its motion is two dimensional.

$$p = 2a_n \rho H \sum_{n=1}^{\infty} \frac{(-1)^{n+1}}{\lambda_n^2} e^{\left(-\lambda_n \frac{x}{H}\right)} \cos\left(\lambda_n \frac{y}{H}\right) \tag{4.2}$$

where, $$\lambda_n = \frac{(2n-1)\pi}{2} \tag{4.3}$$

5. VISCOUS OR ABSORBING BOUNDARY

A way to eliminate waves propagating outward from the structure is to use boundary condition suggested by Lysmer and Kuhlemeyer (1969). This method consists of simply connecting dashpots to all degrees of freedom of the boundary nodes and fixing them on the other end. Lysmer boundaries are derived for an elastic wave propagation problem in a one-dimensional semi-infinite bar.

The damping coefficient C of the dash pot equals

$$C = A\rho c \tag{5.1}$$

where A is the section of the bar, ρ is the mass density and c the wave velocity that has to be selected according to the type of wave that has to be absorbed (shear wave velocity C_s or compressional wave velocity (C_p). In the present case, shear wave velocity C_s has only been used formulating the model. The shear wave velocity C_s is given by

$$C_s = \sqrt{\frac{G}{\rho}} \tag{5.2}$$

Where G is the shear modulus of the medium and is expressed as

$$G = \frac{E}{2(1+\mu)} \tag{5.3}$$

Here E is the Young's modulus and μ is the Poisson's ratio.

6. ITERATIVE SCHEME

The equations of motion for dam and foundation system are written separately as follows:

$$M_d \ddot{x}_d + C_d \dot{x}_d + K_d x_d = f_d \tag{6.1}$$
$$M_f \ddot{x}_f + C_f \dot{x}_f + K_f x_f = f_f + f_{if} \tag{6.2}$$

Here, M, C, K and f are the mass, damping, stiffness and the applied load matrices respectively. The subscripts d and f represents the dam and the foundation domain respectively. The load vector f_d includes the hydrodynamic forces developed from reservoir water, the earthquake forces, self weight of the dam body and any other external forces if present. The vector f_{if} is the vector of interactive force for the soil region exerted by the dam body. The vector f_{if} is generated during successive iteration at the interface nodes of dam and foundation. The vectors of acceleration, velocity and displacement for the dam part at any time step t are represented by $\ddot{x}_d, \dot{x}_d$ and x_d. The vectors $\ddot{x}_f, \dot{x}_f$ and x_f also carry similar meaning for the foundation part.

The iterative scheme has been developed to determine the responses of the dam-foundation coupled system. At any instant of time t, the equation of motion for the dam part eq. (6.1) is solved first with the applied load f_d considering dam to be fixed at the bottom i.e. at the dam-foundation interface nodes. The exerted forces give rise to reaction forces at the common nodes of dam-foundation interface. The reaction forces generated at the common interface nodes of structure and foundation are then applied in the opposite direction at the common nodes of the foundation system to solve eq. (6.2) at the same time instant t. These reaction forces applied in the opposite direction for the foundation part is termed as f_{if} .

After solving the foundation part against the applied load of ($f_f + f_{if}$), the common interface nodes for the foundation part will undergo some displacement. These displacements are then fed into eq. (6.1) in the next instant of time ($t + \Delta t$) as the known displacements at the common interface nodes of dam and foundation. Subsequently, the response of the dam needs to be solved again with the changed boundary conditions which will be different from the earlier step. As a result, the developed reaction forces at the common interface nodes will be different from earlier step and therefore, the foundation domain is solved again with the modified reaction forces developed at the dam-foundation interface at the same time step. In this way iterations are continued for a particular time step t until the displacements and stresses for both the dam and foundation part are found to be converged with a certain level of tolerance. The displacements (x) of both the domain at a particular time step are assumed to converge if the following relationships are satisfied:

$$\frac{\left|\{x_d\}_{i+1}^t - \{x_d\}_i^t\right|}{\left|\{x_d\}_i^t\right|} \le \varepsilon \quad \text{and} \quad \frac{\left|\{x_f\}_{i+1}^t - \{x_f\}_i^t\right|}{\left|\{x_f\}_i^t\right|} \le \varepsilon \tag{6.3}$$

Here, i is the number of iteration, ε is a small pre-assigned tolerance value and t is a particular time instant.

Thus this iteration process goes on until the displacements and stresses between two successive iterations converge in both the dam and foundation domain.

7. NUMERICAL RESULTS

7.1 Validation of the Proposed Algorithm

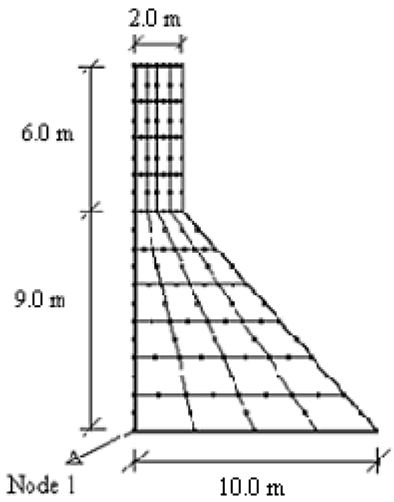

Fig. 1 The geometry of dam (Yazdchi et al, 1999)

In order to validate the proposed algorithm, a concrete gravity dam-foundation prototype resting on rock foundation was analyzed previously analyzed by Yazdchi et al (1999). The dam and the foundation domain has been discretized using two-dimensional, plane strain, eight nodded isoparametric finite elements. The dimensions of the dam prototype are depicted in Fig. 1. The width of the base of the dam is 10.0 m. The height of the dam is 15.0 m out of which the crest portion is of 6.0 m in length. The width of the crest is considered as 2.0 m. The width and the depth of the foundation part are considered as 250.0 m and 100.0 m respectively. The material properties of the dam and the foundation part are taken same and are as follows:

The Young's modulus is considered to be 30000000.0 kN/m^2. The Poisson's ratio is 0.20 and the mass density of both dam and foundation is considered to be 2600.0 kg/m^3. The discretized domain of the dam body is shown in fig. 1 which rest on a foundation of dimensions as stated above.

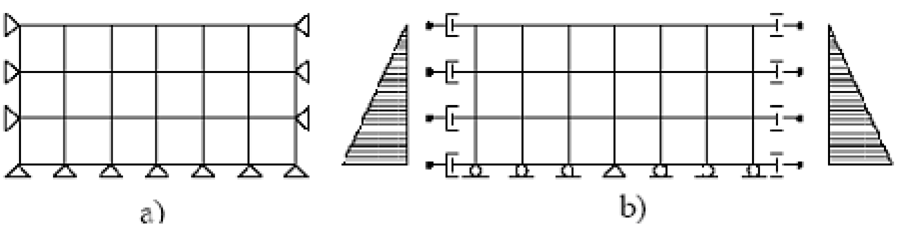

Fig. 3 Boundary conditions for 2-D model

During analysis of the dam-foundation prototype (Yazdchi, 1999), viscous boundary conditions were applied at the truncated boundaries. The Koyna earthquake acceleration is applied with a scaling factor of 2.5. The effect of viscous damping and the hydrodynamic pressure is also considered as Yazdchi et al (1999). Also, initially the dam is analyzed considering the effects of its self weight and the hydrostatic pressure.
The boundary condition applied to the side nodes of the foundation domain is shown in fig. 3. First, the side boundary nodes are fully kept fixed (fig. 3a). Then, these nodes are released and fitted with dashpots in both normal and tangential direction (fig. 3b). The perpendicular dashpots to the boundary absorb the p-waves,

whereas the dashpots tangential to the boundary absorb the s-waves. In these models, no displacement constraints are used. Therefore, the horizontal at rest earth pressure is applied at the boundary nodes situated at the both sides of the foundation domain. This is done by recording the reaction forces in the model with fixed boundaries and applying them with opposite sign to the model with absorbing boundaries.

This configuration of boundary conditions has no fixed point in x-direction. Because the dash pots only provide resistance to high velocity motions, the model is very sensitive to low frequency components of the motion. The slightest imbalance in acceleration causes the entire model to move as a rigid body in x-direction. To avoid this, the node at the center of the base is fully fixed in the following analyses.

Table 1 Comparison of maximum horizontal crest displacements

Horizontal crest displacements (mm)		Yazdchi (et al, 1999)	Staggered method with boundary conditions as per Fig. 8 (b)	% of deviation
Impedance ratio (E_f/E_d)	0.5	6.89	6.99	1.45
		-7.53	-7.20	4.38
	1.0	4.38	4.72	7.76
		-4.41	-4.28	2.95
	2.0	4.27	4.11	3.74
		-3.85	-3.97	3.12
	4.0	4.11	3.90	5.11
		-3.70	-3.57	3.51

Table 1 shows the comparison between the results of Yazdchi et al (1999) and that of the proposed method for different E_f/E_d (impedance ratio) ratios. The maximum crest displacement of the dam under seismic excitation by both the method has been tabulated in the table 1 for a comparison purpose. The obtained displacements by the proposed interaction scheme are in very close agreement with the results obtained by Yazdchi et al (1999).

8. NONLINEAR RESPONSE OF KOYNA GRAVITY DAM

The seismic response of Koyna dam has been investigated considering the interaction behavior of a linear concrete dam and a nonlinear, elasto-plastic foundation subjected to Koyna earthquake (1967) acceleration. The foundation material is assumed to be of hard rock. The width and the depth of the foundation are assumed to be 250.0 m and 100.0 m respectively. The geometry of the dam-foundation system chosen for the analysis purpose is shown in Fig. 4. The material properties of the dam are as follows:

The Young's modulus of dam body is assumed to be 3.15e+10 N/m^2. The Poisson's ratio and the mass density are assumed to be 0.20 and 2415.816 kg/m^3 respectively. The nonlinearity of the foundation domain was modeled by Duncan-Chang model (1970). The material properties of the foundation are as follows:
The Young's modulus of foundation rock is considered to be 1.75e+10 N/m^2. The Poisson's ratio is assumed as 0.2. The mass density of the foundation material is assumed to be 1800.0 kg/m^3. The cohesion of the rock foundation is considered to be 150.0e+4 N/m^2 and angle of internal friction is taken as 40.0^0.

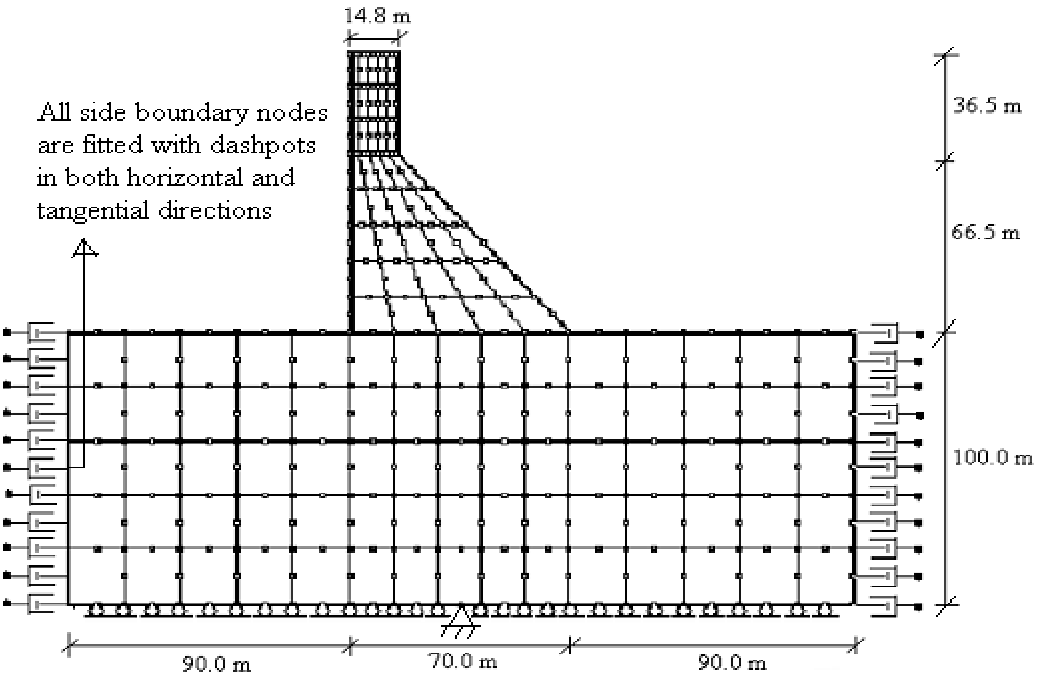

Fig. 4 The geometry and boundary condition of the Koyna dam-foundation system

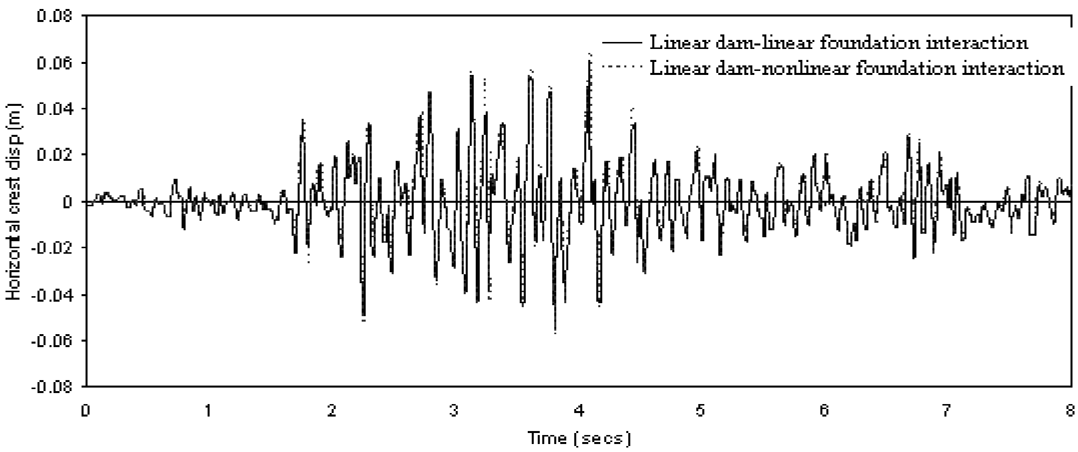

Fig. 5 Horizontal crest displacement vs. time for linear dam and linear/nonlinear interaction

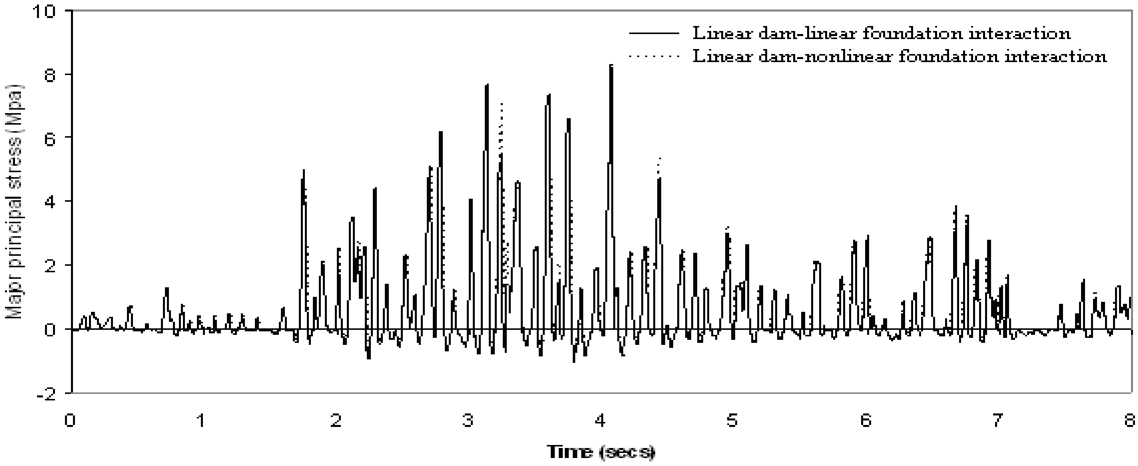

Fig. 6 Major principal stress vs. time for linear dam and linear/nonlinear foundation at the heel

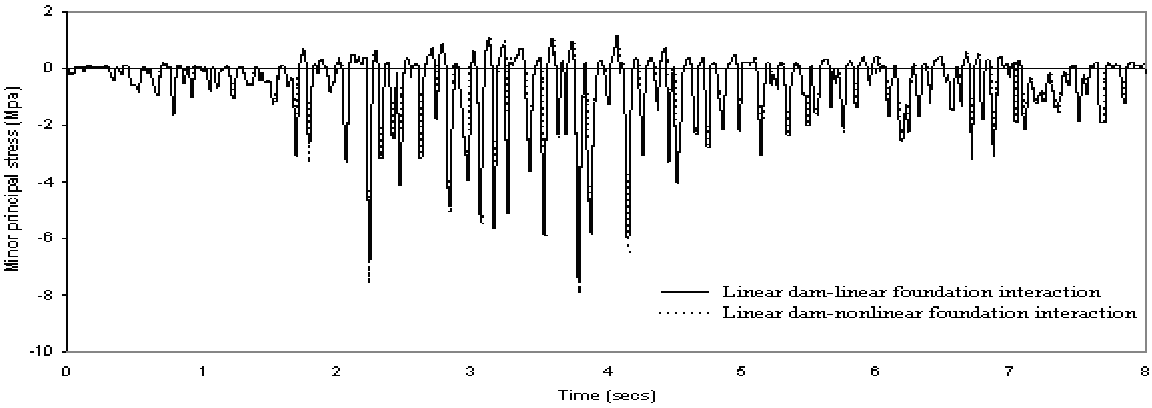

Fig. 7 Minor principal stress vs. time for linear dam and linear/nonlinear foundation at the heel

Fig. 5 shows the comparison of interaction analyses with dam and linear/nonlinear foundation material. The linear dam and linear foundation interaction produced maximum +ve and –ve displacement of 6.03 cm and -5.58 cm respectively. The interaction analysis with nonlinear foundation produced maximum +ve and -ve crest displacements as 6.33 cm and -5.75 cm respectively. Therefore, a variation of 4.98% and 3.05% occurred between these two results. Fig. 6 plots major principal stress vs. time for dam and linear/nonlinear foundation interaction analyses. The maximum values were observed to be 8.17 Mpa and 8.26 Mpa respectively for interaction analyses of dam with linear and nonlinear foundation material marking an increase of 1.11% for nonlinear case. Fig. 7 plots minor principal stress vs. time for dam and linear/nonlinear foundation material. The maximum values of principal stress were observed to be -7.42 Mpa and -7.94 Mpa respectively for interaction analyses of dam and linear/nonlinear foundation showing a rise of 7.01% for nonlinear case.

9. CONCLUSIONS

The present paper illustrates a simple iterative method for the dynamic analysis of nonlinear soil-structure interaction problems. The two individual systems are solved separately and the interaction effects are incorporated through an iterative manner. The proposed method is validated from the literature which shows the accuracy of the developed algorithm. The infinite nature of the foundation domain is simulated through viscous dashpots placed at the boundaries. The Koyna dam-foundation system has been solved against Koyna earthquake acceleration. The nonlinear nature of the foundation material is simulated by considering Duncan-Chang model. The consideration of nonlinear material behavior of the foundation domain produces higher displacements and stresses compared to the case when the foundation is assumed to be linear and elastic. Therefore, it is always advisable to consider the nonlinearity of the foundation during interaction analysis for a very important structure.

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