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Seismic Response of Concrete Gravity Dams Considering Foundation Flexibility

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Introduction

The seismic behavior of concrete dams has been the subject of extensive research during the past decade because of concern for dam safety during earthquakes. Concrete dams are distinguished from other structures because of their large size and their interactions with the reservoir and foundation. The response of a dam during an earthquake depends on the characteristics of the ground motion, the surrounding soil and reservoir, and the dam itself. For the structure founded on soft soil, the motion of the base of the structure will be different from the free-field motion because of the coupling of the soil-structure system. This is due to the following reasons: First, the inability of the foundation to conform to the deformations of the free-field motion which would cause the motion of the base of the structure to deviate from the free-field motion. Second, the dynamic response of the supporting structure itself would induce deformation of the supporting soil. *i.e.,* the soil on which a structure is constructed may interact dynamically with the structure during earthquakes, especially when the soil is relatively flexible and the structure is stiff. This kind of dynamic soil-structure interaction (SSI) can sometimes modify significantly the stresses and displacements of the whole structural system from the values that could have been developed if the structure were constructed on a rigid foundation. If the foundation is rigid, the energy received by the structure from the base during an earthquake, can be dissipated only through material damping mechanisms, such as viscous damping. In the case of flexible soils, some energy is fed back to the base and radiated away giving rise to the so-called *geometric damping or radiation damping*.

Review of Previous Works

The general methodologies for soil-structure interaction are direct and substructure approaches, depending on the modelling method for the soil around the structure (Dutta and Roy 2002). In the substructure method, the soil-structure system is divided into two substructures: a structure that may include a portion of nonlinear soil adjacent to it and the unbounded soil. The unbounded soil region is usually represented by an impedance matrix, which may be

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attached to the dynamic stiffness matrix of the structure. Though simple, this method is restricted to simple geometry and linear soil (Wolf, 1985). In the direct method, the structure and the soil adjacent to it are modeled directly and analyzed in a single step. A consistent free-field ground motion is being applied to the boundaries of the discrete model and the response of the combined soil-structure system is computed. The response of the soil and the structure obtained was used as input in a second stage analysis to obtain the detailed structural response. Direct SSI analyses were more commonly performed using equivalent linear methods to approximate the effects of soil nonlinearity. This technique limits the extent of soil domain to be considered (Wolf 1985, 1987). Wolf and Song (1996) also developed consistent infinitesimal finite element cell technique which proved to be very useful in modeling the unbounded media. This method is exact in nature in radial direction and is able to simulate the nonlinear behavior of soil. The detailed description of this method can be found in this literature.

Much of the reported research on the dynamic analysis of dam foundations assumes linear behaviour of the foundation media. Adnan and Wilson (1990) developed an efficient computational technique for the dynamic analysis of large linear structural systems with local non-linearities. They developed a rational approach to the earthquake-resistant design of structure-foundation systems with predetermined non-linearities occurring along the structure-foundation interface. They had analyzed the dam-foundation model by considering local nonlinearities of soil. Yazdchi *et.al.* (1999) presented a computational method for the transient soil-structure interaction analysis using the coupled finite element-boundary element method. In the analysis, the half space soil was represented by the boundary element method (BEM) with linear material properties. Estorff and Firuziann (2000) investigated the transient inelastic response of structures coupled with a half space using a general coupled boundary element and finite element formulation. The inhomogenieties and elastoplastic behavior with hardening effects were accounted for in the near field of the surrounding soil. The far field was modeled using boundary elements. At the interface, the nodal forces resulting from the BE were treated as additional loads in each iteration. Since non-linearities may occur only in the FE sub domain, the geometrical linearity was checked along the interfaces by observing the strains at the interface nodes. There is a common belief that boundary element method is superior over finite element for the modelling of infinite or semi-infinite domains. However, in the reported literature, (Yang et al. 1993) the efficiency of boundary element method in time domain analysis is not ascertained. This is because of the presence of the convolution integral and singularity of the kernels of the formulation which requires large storage space and computational time for the evaluation of the effect of past time history and numerical integration of the kernels. Moreover, use of boundary element method requires the solution of an unsymmetrical and unbounded matrix. Hence this method does not possess any significant advantage over the finite element method. On the other hand, the FEM are well-established procedures (Zienkiewicz and Taylor 1991, Bathe 1996). These methods routinely accommodate complex geometries and non-linear phenomena. Kocak and Mengi (2000) proposed a simple three-dimensional soil-structure interaction model in which the layered soil medium was divided into thin layers and each thin layer was represented by a separate parametric model. The parameters of this model were determined, in terms of the thickness and elastic properties of the sub layer.

Two important characteristics that distinguish the dynamic soil-structure interaction system from other general dynamic structural systems are the unbounded nature (Yun *et al* 2000) and the nonlinearity (Halabian *et al* 2002) of the soil medium. The coupled response of the dam-foundation system not only depends on their material properties, but also on the characteristics of the ground motion (Maity and Reddy 2007). That is why it is important to model the structure and the soil in such a way so that the actual behavior of the structure and the soil is represented. While the structural models are well established in the literature, soil models involve complicated analysis due to their unbounded nature. Soil being heterogeneous, anisotropic and nonlinear in force-displacement characteristics is very difficult to model physically (Kim and Roesset 2004) and at the same time to represent mathematically.

The focus of the present paper is to analyze concrete gravity dam considering soil-structure interaction for dynamic excitation considering nonlinear material properties for the foundation. The interaction effect due to foundation has been adopted by the direct method of soil-structure interaction. The results show the need of consideration of foundation flexibility while analyzing massive structures like concrete gravity dam.

Theoretical Formulation

Modeling of Dam

The structural system considered for the present investigation, has been analyzed using two dimensional plane strain formulations. Since the problem involved here is a long body, whose geometry and loading do not vary in the longitudinal direction, can be analyzed by this idealization appropriately. The equation of motion of the dam under seismic excitation in time-domain can be expressed as:

$$M\ddot{u} + C\dot{u} + Ku = -M\,\ddot{u}_g + F_f \qquad (1)$$

Where, M and K are the mass and stiffness matrices of the dam respectively. The parameter u, $\dot{u}$ and $\ddot{u}$ are displacement, velocity and acceleration vectors respectively, $\ddot{u}_g$ is the vector of ground acceleration, and F_f is the force vector generated from the foundation-dam interaction. The damping matrix C represents viscous damping in the structure. In this formulation, a popular spectral damping scheme, called Rayleigh or proportional damping is adopted. The damping matrix C is formed as a linear combination of the stiffness and mass matrices as

$$C = \alpha M + \beta K \qquad (2)$$

where, α and β are called proportional damping constants. The use of this damping matrix is equivalent to damping effects that vary with frequency (Bathe 1996).

Modeling of Foundation Domain

The foundation nonlinearity is incorporated through Wen (1976) elasto-plastic model. The model is described in some details in the following section.

Wen Plasticity Model

The plasticity model is based on the hysteretic behavior proposed by Wen (1976) as shown in Figure 1.

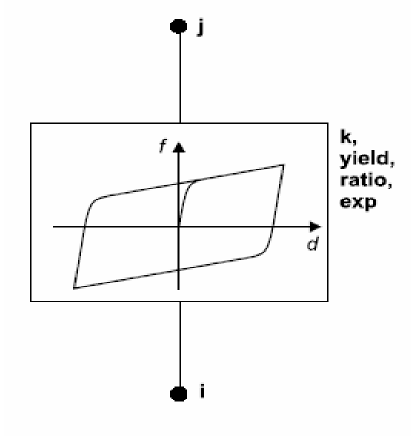

Fig. 1 Wen Plasticity Property Type for Uniaxial Deformation

All internal deformations are independent. Therefore, the yielding at one degree of freedom does not affect the behavior of the other deformations. The nonlinear force-deformation relationship is given by

$$f = r k \, d + (1 - r) y z \tag{3}$$

where *k* is the elastic spring constant, *y* is the yield force, *r* is the specified ratio of post-yield stiffness to elastic stiffness (*k*), and *z* is an internal hysteretic variable.

This variable has a range of $|z| \leq 1$, with the yield surface represented by $|z| = 1$. The initial value of *z* is zero, and it evolves according to the differential equation

$$\begin{aligned} \dot{z} &= \frac{k}{y} \dot{d} \left(1 - |z|^{\exp}\right) \quad if \; \dot{d} \, \dot{z} > 0 \\ &= \frac{k}{y} \dot{d} \qquad\qquad otherwise \end{aligned} \tag{4}$$

where *exp* is an exponent greater than or equal to unity. Larger values of this exponent increase the sharpness of yielding as shown in Figure 2. The practical limit for *exp* is about 20 (Wen, 1976). The equation for $\dot{z}$ is equivalent to Wen's model with *A* = 1 and α = β = 0.5. Nonlinear Model Time-History Analysis method is used to perform the analysis.

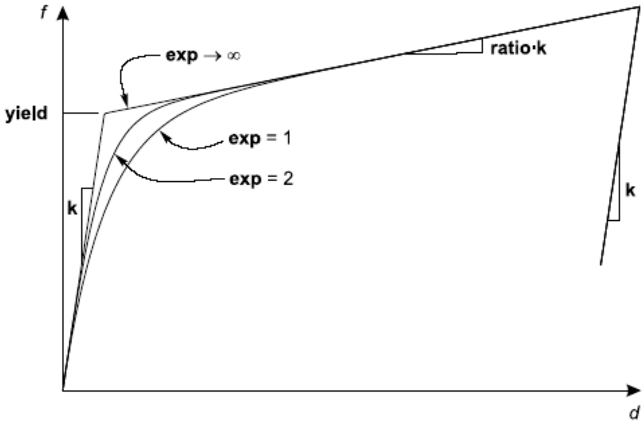

Fig. 2 Definition of Parameters for the Wen Plasticity Property

Replacing Foundation Material with Nonlinear Wen Link Elements

Foundation soil is replaced with nonlinear Wen elasto-plastic link elements (1976). The stiffness and damping values for Wen link elements are calculated by using the following formulas shown in Table 1.

Table 1 Empirical Formulas for Replacing Foundation Soil with Equivalent Springs

Direction	*Stiffness (K)*	*Damping*	*Mass*
Vertical	$4Gr/1-\nu$	1.79 $\sqrt{K \rho r^3}$	1.50 ρr^3
Horizontal	18.2Gr $(1-\nu^2)/(2-\nu)^2$	1.08 $\sqrt{K \rho r^3}$	0.28 ρr^3
Rotation	2.7Gr^3	0.47 $\sqrt{K \rho r^3}$	0.49 ρr^5
Torsion	5.3Gr^3	1.11 $\sqrt{K \rho r^5}$	0.70 ρr^5

Where,

ν - Poisson’s ratio, r - equivalent radius, G-shear modulus, ρ- mass density of foundation

Absorption Boundary on Foundation

In the conventional approach of foundation-structure interaction analyses, the effect of far field (unbounded media) is not considered. For wave propagation analysis, the usual finite boundary of the finite element model will cause the elastic waves to be reflected and superimpose with the progressing waves. Besides modeling the foundation stiffness up to infinity, reflections of the outgoing propagating waves on the artificial boundary at finite distance from the structure must be avoided also. In this case, some numerical treatment is needed to introduce artificial boundary so as to simulate the unbounded nature of the soil foundation and yet maintain a finite computational domain. An artificial boundary is required to be imposed at the truncated boundary using a non-reflecting, absorbing, radiating or transmitting condition that can prevent spurious reflections. Viscous damper components normal and tangent to a far field boundary are used to simulate the radiation condition. The dashpot coefficients are determined in terms of the material properties of the semi-infinite domain as proposed by Wilson (1995).

Solution Scheme for Coupled Dam-Foundation System

In dam-foundation interaction problems, the foundation and the dam do not vibrate as separate systems under external excitations, rather they act together in a coupled way. Therefore, these problems have to be dealt in a coupled way. To develop the fundamental SSI dynamic equilibrium equations, the two-dimensional foundation structure system is considered as shown in Figure 3.

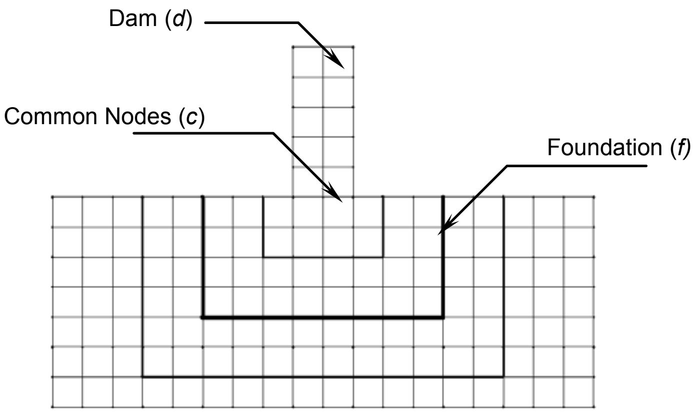

Fig. 3 Foundation Structure Interaction Model

The SSI model is divided into three sets of node points. The common nodes at the interface of the dam and foundation are identified with *"c"*; the nodes within the dam are *"d"* nodes; and the nodes within the foundation are *"f"* nodes. From the direct stiffness approach in structural analysis, the dynamic force equilibrium of the coupled system may be given in terms of the absolute displacements, *U*, by the following sub-matrix equation:

$$\begin{bmatrix} M_{dd} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix} \begin{Bmatrix} \ddot{U}_d \\ \ddot{U}_c \\ \ddot{U}_f \end{Bmatrix} + \begin{bmatrix} K_{dd} & K_{dc} & 0 \\ K_{cd} & K_{cc} & K_{cf} \\ 0 & K_{fc} & K_{ff} \end{bmatrix} \begin{Bmatrix} U_d \\ U_c \\ U_f \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix} \tag{5}$$

where the mass and stiffness at the contact nodes are the sum of the contributions from the dam *(d)* and foundation *(f)*, and are given by:

$$M_{cc} = M_{cc}^{(d)} + M_{cc}^{(f)} \text{ and } K_{cc} = K_{cc}^{(d)} + K_{cc}^{(f)} \tag{6}$$

In terms of absolute motion, there are no external forces acting on the system. However, the displacements at the boundary of the foundation must be known. To avoid solving this SSI problem directly, the dynamic response of the foundation without the structure is calculated. In many cases, this free-field solution can be obtained from a simple one-dimensional site model. The three-dimensional free-field solution is designated by the absolute displacements ***v*** and absolute accelerations $\ddot{v}$.

By a simple change of variables, it is now possible to express the absolute displacements $\dot{U}$ and accelerations $\ddot{U}$ in terms of displacements *u* relative to the free-field displacements *v***.** Or

$$\begin{bmatrix} U_d \\ U_c \\ U_f \end{bmatrix} = \begin{bmatrix} u_d \\ u_c \\ u_f \end{bmatrix} + \begin{bmatrix} v_d \\ v_c \\ v_f \end{bmatrix} \quad and \quad \begin{bmatrix} \ddot{U}_d \\ \ddot{U}_c \\ \ddot{U}_f \end{bmatrix} = \begin{bmatrix} \ddot{u}_d \\ \ddot{u}_c \\ \ddot{u}_f \end{bmatrix} + \begin{bmatrix} \ddot{v}_d \\ \ddot{v}_c \\ \ddot{v}_f \end{bmatrix} \tag{7}$$

The above equation can now be written as

$$\begin{bmatrix} M_{dd} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix} \begin{Bmatrix} \ddot{u}_d \\ \ddot{u}_c \\ \ddot{u}_f \end{Bmatrix} + \begin{bmatrix} K_{dd} & K_{dc} & 0 \\ K_{cd} & K_{cc} & K_{cf} \\ 0 & K_{fc} & K_{ff} \end{bmatrix} \begin{Bmatrix} u_d \\ u_c \\ u_f \end{Bmatrix} = R \tag{8}$$

where,

$$R = -\begin{bmatrix} M_{ss} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix} \begin{Bmatrix} \ddot{v}_d \\ \ddot{v}_c \\ \ddot{v}_f \end{Bmatrix} - \begin{bmatrix} K_{dd} & K_{dc} & 0 \\ K_{cd} & K_{cc} & K_{cf} \\ 0 & K_{fc} & K_{ff} \end{bmatrix} \begin{Bmatrix} v_d \\ v_c \\ v_f \end{Bmatrix} \tag{9}$$

But this approach is numerically inconvenient. Therefore, in order to reduce the numerical difficulties, the following change of variables is introduced as suggested by Wilson (1995).

$$\begin{bmatrix} U_d \\ U_c \\ U_f \end{bmatrix} = \begin{bmatrix} u_d \\ u_c \\ u_f \end{bmatrix} + \begin{bmatrix} 0 \\ v_c \\ v_f \end{bmatrix} \quad and \quad \begin{bmatrix} \ddot{U}_d \\ \ddot{U}_c \\ \ddot{U}_f \end{bmatrix} = \begin{bmatrix} \ddot{u}_d \\ \ddot{u}_c \\ \ddot{u}_f \end{bmatrix} + \begin{bmatrix} 0 \\ \ddot{v}_c \\ \ddot{v}_f \end{bmatrix} \tag{10}$$

Substitution of this change of variables into equation (8) yields the following dynamic equilibrium equation in terms of absolute displacements $\{u_d\}$ of the dam body. Therefore, equation (8) can be expressed as

$$\begin{bmatrix} M_{dd} & 0 & 0 \\ 0 & M_{cc} & 0 \\ 0 & 0 & M_{ff} \end{bmatrix} \begin{Bmatrix} \ddot{u}_d \\ \ddot{u}_c \\ \ddot{u}_f \end{Bmatrix} + \begin{bmatrix} K_{dd} & K_{dc} & 0 \\ K_{cd} & K_{cc} & K_{cf} \\ 0 & K_{fc} & K_{ff} \end{bmatrix} \begin{Bmatrix} u_d \\ u_c \\ u_f \end{Bmatrix} = R \tag{11}$$

The right hand side of equation (11) can be calculated as per the suggestions of Wilson (1995). Thus, the vector R is expressed as follows:

$$R = -\begin{bmatrix} M_{dd} & 0 & 0 \\ 0 & M_{cc}^{(d)} & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} 0 \\ \ddot{v}_c \\ 0 \end{Bmatrix} - \begin{bmatrix} K_{dd} & K_{dc} & 0 \\ K_{cd} & K_{cc}^{(d)} & 0 \\ 0 & 0 & 0 \end{bmatrix} \begin{Bmatrix} 0 \\ v_c \\ 0 \end{Bmatrix} \tag{12}$$

Validation of Proposed Algorithm

The results of the present model are compared with the results of dam model analyzed by Yazdchi *et. al.* (1999) for Koyna ground motion. A dam of height 15.0 m, crest-width 2.0 m and base width 10.0 m discretized with isoparametric elements. The foundation size of 250 m × 100 m has been considered in this study (Figure 4). The dam and the foundation are assumed to be linear elastic with the following material properties:

Poisson's ratio = 0.2

modulus of elasticity $E_d = 3 \times 10^7$ kN/m^2

mass density = 2600 kg/m^3

The modulus of elasticity of the foundation was varied from 0.5 to 4.0 times the modulus of dam as considered in the literature. The Poisson's ratio and the mass density of the foundation were assumed to be the same as those of the dam. The 1967 Koyna earthquake motion (Figure 5) has been used for the analysis.

The maximum displacements at dam crest for different foundation material has been calculated in time domain and presented in Table 2. The comparison of the results obtained by the present model and Yazdchi *et. al.* (1999) confirms the correctness of the proposed algorithm.

Table 2 Validation of Present Model with Yazdchi *et. al.* 1999 Model

Dam with →	***Flexible base for different E_f/E_d ratio***			
	0.5	1.0	2.0	4.0
Maximum displacement at crest (mm) (Present model)	8.6	4.6	4.5	3.9
Maximum displacement at crest (mm) Yazdchi *et. al.* (1999)	7.53	4.41	3.9	3.70
% of deviation	12	4	13	5

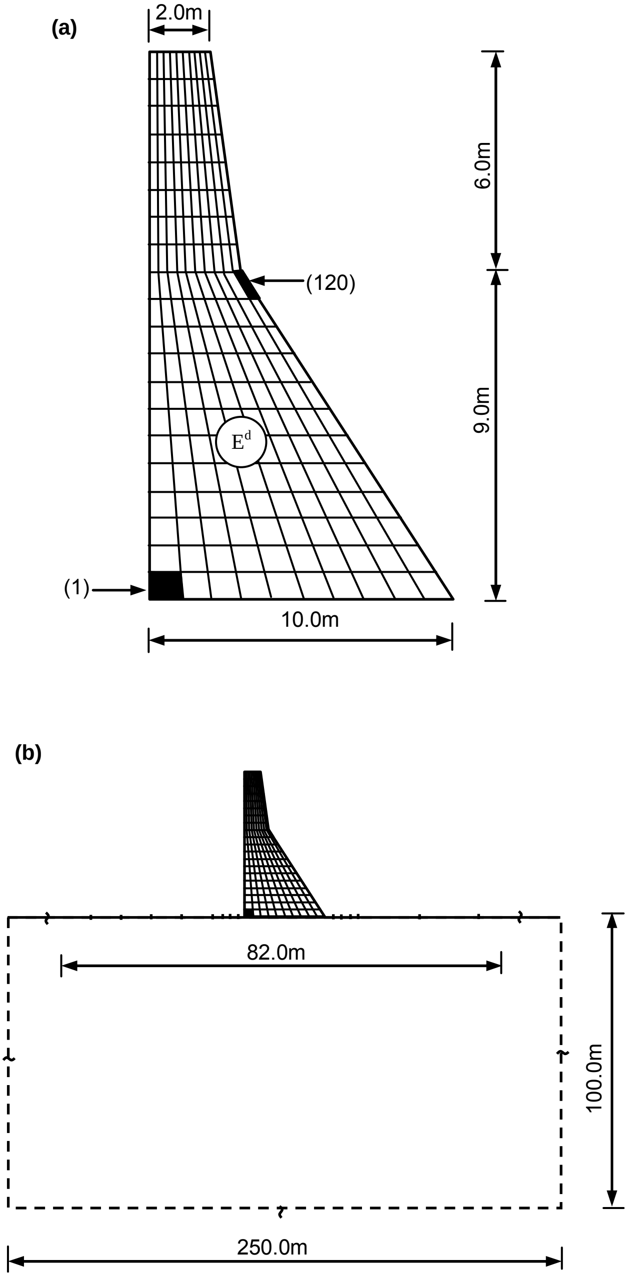

Fig. 4 Discretization of Coupled Dam-foundation System (Yazdchi *et. al.1999*)

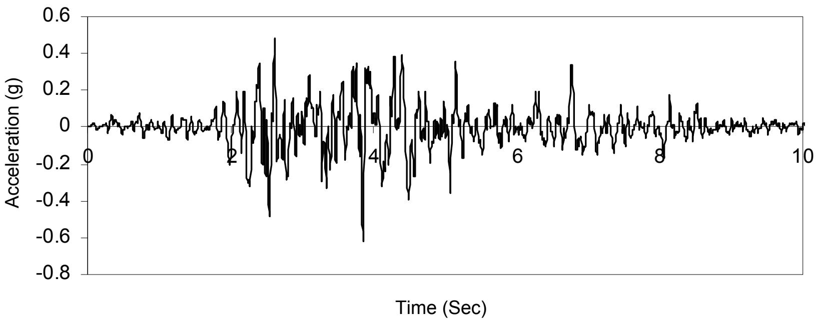

Fig. 5 Koyna - Longitudinal Earthquake Motion (1967)

Analysis of Dam-Foundation Coupled System

Statement of the Problem

The Koyna dam has been chosen in the present study for the extensive analysis using finite element technique. The tallest non-overflow monolith of height 103 m, width at the top of the dam 14.8 m and at the base 70.0 m is considered for the present study. The two-dimensional finite element idealization for this monolith is shown in Figure 6.

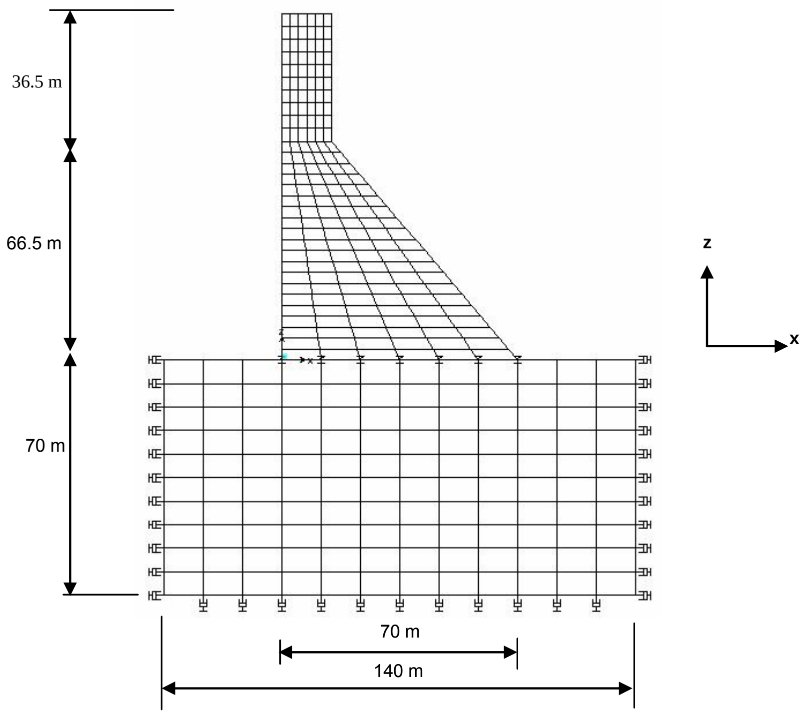

Fig. 6 FE Discretization of Dam-foundation System with Absorbing Boundary

The eight nodded, isoparametric finite elements have been used for the discretization of the structure and foundation domain throughout the present analysis. The mass concrete in the dam is assumed to be homogeneous, isotropic, linear elastic solid with the following properties. Modulus of elasticity (E_d) =3×10^7 kN/m^2; Poisson's ratio = 0.2 and mass density = 2400 kg/m^3. The material properties of the foundation is: Modulus of elasticity (E_f) =2.5×10^7 kN/m^2; Poisson's ratio = 0.33 and mass density = 2400 kg/m^3.

The dam is analyzed to get the response subjected to the seismic accelerations of 1940 El-Centro (N-S component) earthquake. The entire solution has been done in time domain with application of Direct Method of soil-structure interaction.

Selection of Size of Foundation Domain

The dam and its foundation are assumed to be in a state of plane-strain condition. A convergence study is carried out in order to arrive at a suitable foundation domain of finite dimension by varying horizontal and vertical extent of the foundation. The time periods and the maximum displacement at the crest have been calculated and compared for the convergence study. The material properties of dam and foundation are considered as stated in section 5.1.

In the first step, the foundation length in horizontal direction is taken arbitrarily equal to 1.5 times the width of dam (*b*) at its base, and the foundation length in vertical direction equal to half the width of dam at its base. By keeping height of foundation in vertical direction as constant, the length in horizontal direction is increased by 0.5 times the base width of dam (*b*), till the results are converged. It is observed that in between the foundation widths of 1.5*b* and 2*b* the variation in displacements of two models is negligible. Therefore, the horizontal length of the foundation is fixed as *2b*. In the second step, length of foundation in vertical direction is increased by keeping horizontal length as constant. It is observed that in between the foundation depths of 0.5b and *b* the variation in displacements of two models is negligible. So the vertical length of foundation is fixed as *b*. Thus, the size of the foundation domain may be considered for further analysis as $2b\times b$ (Figure 6) where the results are converging sufficiently. Some of the results of the convergence study are shown in Table 3. The above conclusions are made with an assumption that no incoming waves propagating from infinity towards the structure exist at the boundary *i.e.,* there is no energy associated with the waves may radiate from the infinity into this truncated area. The displacements at the edges of the foundation are taken care by the absorbing boundary conditions that are incorporated.

From the table it is observed that the time periods of dam with different foundation models are increased compared to that of the dam with fixed base condition. Similarly displacement at crest of dam is also increased in case of coupled system when compared to fixed base system. These results clearly show the importance of foundation flexibility to be taken into consideration during the analysis.

Variation of Time Periods and Frequencies

Table 4 shows the comparison of time periods as well as corresponding fundamental frequencies of rigid base and the flexible base system for different modes. It is observed that the frequencies of the dam are significantly reduced when the dam-foundation interaction effect is taken into account. The time periods of the coupled system are elongated with the introduction of foundation flexibility.

Table 3 Effect of Foundation Size on the Response of Koyna Dam (E_f/E_d=0.833)

Dam With	*Time Period (Sec)*		***Max. Disp. at Crest (m)***	***Direct and Shear stress values at heel (kN/m²)***		
	Mode1	Mode2		(σ_{xx})	(σ_{zz})	(σ_{xz})
Rigid foundation	0.358	0.132	0.032	315.38	1756.47	207.77
Foundation size 1.5 *b* × 0.5 *b*	0.393	0.157	0.042	913.87	2596.79	451.05
Foundation size 1.5 *b* × 1.0 *b*	0.397	0.161	0.043	1033.78	2697.52	495.99
Foundation size 2.0 *b* × 0.5 *b*	0.391	0.157	0.042	819.38	2605.31	435.02
Foundation size 2.0 *b* × 1.0 *b*	0.40	0.167	0.045	1093.48	2829.61	520.91
Foundation size 2.5 *b* × 1.0 *b*	0.402	0.169	0.047	1186.39	2906.15	593.02
Foundation size 2.0 *b* × 1.5 *b*	0.402	0.169	0.045	1027.22	2719.67	511.38
Foundation size 3.0 *b* × 1.0 *b*	0.403	0.172	0.049	1060.09	2895.72	543.3
Foundation size 3.0 *b* × 1.5 *b*	0.405	0.177	0.049	1162.54	2867.84	592.13
Foundation size 3.0 *b* × 2.0 *b*	0.406	0.180	0.049	1161.12	2822.23	582.95

Variation of Displacements and Stresses

By considering foundation material as elasto-plastic (Wen 1976) the corresponding variation in displacements are shown in Table 5 for different nonlinear parameters (*i.e.,* yield force and stiffness ratio). From the tabular data it is observed that for higher yield forces and stiffness ratios, the displacements become almost constant (37 mm in this case). Selection of these nonlinear parameters depends on the foundation material properties and one can get these values from experimental data. For the present case, these parameters have been considered on the basis of works done by Park *et.al.* (1986).

Table 6 shows the variation of crest displacement, direct and shear stresses for rigid and nonlinear foundation material properties. The material nonlinearity for foundation is accounted for by incorporating an advanced plasticity-based soil model called Bouc-Wen elasto-plastic modal (Sections 3.2.1 and 3.2.2). The results show that the maximum crest displacement in nonlinear case is more than that of rigid case.

Table 4 Variation of Frequencies and Time Periods

Mode	*Rigid-Base System* T *(Sec.)*	*Rigid-Base System* Ω *(rad/sec.)*	*Flexible System* T *(Sec.)*	*Flexible System* Ω *(rad/sec.)*
1	0.358	17.55	0.40	15.688
2	0.132	47.373	0.167	37.665
3	0.087	72.304	0.127	49.396
4	0.065	96.811	0.092	68.092
5	0.043	147.27	0.071	88.215
6	0.041	153.26	0.065	96.573
7	0.031	206.12	0.054	115.79
8	0.027	233.22	0.048	130.97
9	0.025	250.72	0.047	131.79
10	0.024	254.44	0.044	141.37
11	0.022	289.81	0.040	155.43
12	0.021	290.81	0.038	164.70

Table 5 Effect of Yield Force and Stiffness Ratio on Displacements

Yield force (kN)	***Displacement at crest (mm) for stiffness ratio of***			
	0.05	0.1	0.15	0.2
2×10^2	40	39	39	39
2×10^3	38	37	37	37
2×10^4	37	37	37	37
2×10^5	37	37	37	37

Table 6 Response of Dam with Nonlinear Foundation Material Properties

Dam with	***Max. disp. at crest (mm)***	***Direct and shear stress values at heel (kN/m^2)***		
		(σ_{xx})	(σ_{zz})	(σ_{xz})
Rigid foundation	32	315.38	1756.47	207.77
Nonlinear foundation material properties	40	491.04	2778.55	400.84

Figure 7 shows the crest displacement of dam considering the foundation as rigid and flexible. Due to the flexibility effect of the foundation, the displacement amplitudes are increased compared to that of the rigid base.

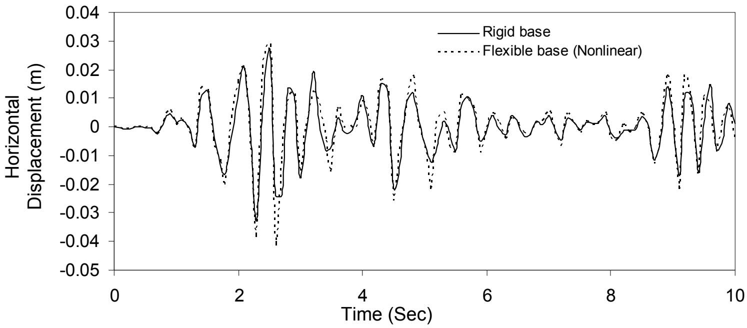

Fig. 7 Variation of crest displacement under El-Centro excitation

Effect of Modulus of Elasticity of Foundation

The modulus of elasticity of the foundation is varied from 0.25 to 5.0 times the modulus of elasticity of dam to study the influence of the foundation material properties on the response of the dam. The variation of horizontal displacements at the crest, direct and shear stress values at heel for different E_f/E_d ratio are shown in Table 7. It is observed from the tabular data that with the increase of foundation stiffness, the crest displacements, stresses and the vibration periods of the coupled system decrease significantly. This indicates that with the increase in stiffness of the unbounded foundation media, the system behaves like a structure on a rigid foundation. From Table 7, it is observed that the maximum crest displacement for the case of E_f/E_d = 5.0 is 36 mm, while for cases E_f/E_d = 0.5 and E_f/E_d = 0.25 is 45 mm and 56 mm, respectively. This indicates that there is about 25% and 56% increase in the magnitude with decrease of the rigidity of the foundation material. This indicates that assumption of a rigid base of the dam can underestimate the displacements in the dam when compared to dam with flexible base. Similarly the maximum direct stress value for the case of E_f/E_d = 5.0 is 2061.35 kN/m^2, while for cases of E_f/E_d = 0.5 and E_f/E_d = 0.25 is 2684.92 kN/m^{2}and 3596.64 kN/m^2 respectively. This indicates that there is about 30% and 74% increase in the magnitude of the stress value.

Table 7 Response of Dam for Different E_f/E_d Ratio

E_f/E_d ratio	*Time Period (Sec)*		*Max. Crest Displ. (mm)*	*Direct and Shear stress values at heel (kN/m^2)*		
	Mode 1	*Mode 2*		*(σ_{xx})*	*(σ_{zz})*	*(σ_{xy})*
0.25	0.468	0.215	56	993.71	3596.64	802.77
0.5	0.42	0.183	45	648.22	2684.92	539.54
1.0	0.395	0.162	39	464.95	2250.71	349.72
2.0	0.383	0.149	37	393.88	2123.4	254.99
3.0	0.378	0.144	36	372.09	2096.65	231.07
4.0	0.376	0.141	36	369.24	2076.42	219.4
5.0	0.375	0.140	36	366.84	2061.35	214.6

In general, it may be concluded that by including the effects of foundation flexibility in the analysis, the displacement and the stress distribution in the body of the dam will be significantly different. Therefore, the foundation stiffness should be included in the response analysis of dams to increase the accuracy in results. Figure 8 shows the variation of crest displacement with time for Ef/E_d ratio of 0.25 and 5.0.

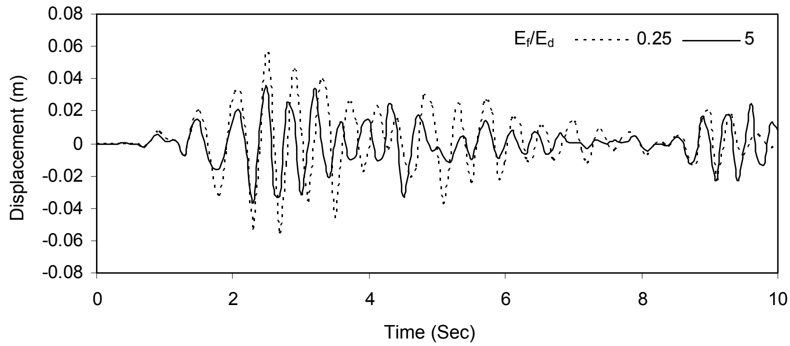

Fig. 8 Variation of Displacement for Different Ef/Ed Ratio (Nonlinear Analysis)

Base Shear and Base Moment

Table 8 shows the comparison of base shear (kN) and base moment (kN-m) for first three fundamental modes with different E_f/E_d ratio along with the rigid base. It is observed that the magnitudes of base shear and base moments obtained from the rigid base model are much higher than that of the elastic base. Consideration of flexibility of the supporting foundation reduces the magnitude of the base shear and base moment of the dam.

It is observed that the magnitudes of base shear and base moments get reduced as E_f/E_d ratio reduces. Incorporation of dam-foundation interaction effects has the direct result of reducing the base shear applied to the structure, and consequently the lateral forces and overturning moments.

Table 8 Comparison of Base Shear (kN) and Moment (kN-m) for Different E_f/E_d Ratios

Mode	E_f/E_d = 0.25		E_f/E_d =1.0		E_f/E_d =5.0		Rigid base	
	base shear	base moment	base shear	base moment	base shear	base moment	base shear	base moment
1	11093	16641	12849	19274	12960	19440	13922	20883
2	47140	70710	86495	129743	99683	49524	99902	149853
3	68593	102890	140442	210664	191227	286839	179771	269656

Conclusions

The effect of foundation flexibility on the seismic response of concrete gravity dam is investigated using the method described above. The proposed algorithm has been validated with the results available in the literature. The responses of the soil-structure system with absorbing boundary indicate that the most of the incident energy is absorbed at the truncation boundary. By the use of absorption boundary for the finite element analysis of unbounded foundation domain, a base size of $2b \times b$ will produce sufficiently accurate results compare to the large foundation size. The parametric study shows that the consideration of foundation flexibility may alter the response of the dam significantly. The magnitude of base shear and base moment reduces with the increase of foundation flexibility. The results show that the fundamental time period of the coupled system is being elongated if the foundation becomes more flexible. The magnitude of the displacements and stresses on dam under seismic excitation becomes less if the foundation becomes rock type in nature. Consideration of soil-structure interaction effect is necessary if the dam is founded on soft soil.

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