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Direct frequency domain analysis of concrete arch dams based on FE-BE procedure

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Abstract. A FE-BE procedure is presented for dynamic analysis of concrete arch dams. In this technique, dam body is discretized by finite elements, while foundation rock is handled by three dimensional boundary element formulation. This would allow a rigorous inclusion of dam-foundation rock interaction, with no limitations imposed on geometry of canyon shape. Based on this method, a previously developed program is modified, and the response of Morrow Point arch dam is studied for various ratios of foundation rock to dam concrete elastic moduli under an empty reservoir condition. Furthermore, the effects of canyon shape on response of dam, is also discussed.

Keywords: concrete arch dams; boundary element; dam-foundation rock interaction.

1. Introduction

One of the main complexities of dynamic analysis of concrete arch dams, is the modeling of dam-foundation rock interaction by accurate methods. In the initial studies, this modeling was simplified by using a massless foundation model, such as the work of Fok and Chopra (1986). In that case, the finite element discretization was utilized for both dam body and foundation rock domains (FE-FE approach). Later on, Tan and Chopra (1995) improved this methodology and they presented a technique, which considers dam-foundation rock interaction rigorously. In this work, the dam was modeled by finite element, while the foundation rock was represented by a two-dimensional (2D) boundary element formulation combined with a series expansion along the canyon axis direction (FE-BE technique). However, the main limitation of this work, is that the foundation rock geometry must be that of a uniform canyon extending to infinity. In other studies, i.e., the work of Maeso and Dominguez (1993) or its enhanced versions (Maeso *et al*. 2002, 2004), both dam and foundation rock domains were treated by boundary element formulations (BE-BE method).

In this paper, the problem is analyzed by FE-BE technique. That is, dam body and foundation rock domains are discretized by finite and boundary elements, respectively. Meanwhile, the foundation rock is represented by applying a three-dimensional (3D) boundary element formulation. Therefore, the geometry of canyon could be quite arbitrary and no limitations are required to be imposed. The formulation is discussed initially. A previous computer program (Lotfi 2001) is modified based on this method and the response of Morrow Point arch dam, is studied as a typical

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example. The investigation is carried out for various ratios of foundation rock to dam concrete elastic moduli, as well as different shapes of canyon and assumptions in regard to dam-foundation interaction.

2. Formulation

Let us consider a concrete arch dam-foundation rock system. The geometry is assumed completely arbitrary for the arch dam or the canyon shape. Although, the effects of hydrodynamic pressures could be also taken into account by well known incompressible or compressible water theories (Hall and Chopra 1983, Lotfi 2005), for simplicity, the present study is confined to the case of an empty reservoir.

2.1 Dam domain

Dam body is discretized by finite elements. The equations of motion for this region can be written as

$$\mathbf{M}\ddot{\mathbf{r}} + \mathbf{C}\dot{\mathbf{r}} + \mathbf{K}\mathbf{r} = -\mathbf{M}\mathbf{J}\mathbf{a}_g(t) + \mathbf{R} \tag{1}$$

in which **M**, **C**, and **K** are the total mass, damping and stiffness matrices of the dam body finite elements. **r** is the vector of nodal point displacements relative to the free field ground displacements. $\mathbf{a}_g(t)$ is a vector which includes three components of ground accelerations. Meanwhile, **J** is a matrix with each three rows equal to a 3×3 identity matrix, and **R** is a force vector which includes interacting forces, in this case only the forces corresponding to the dam-foundation rock interaction.

For harmonic ground excitation $\mathbf{a}_g(t) = \mathbf{a}_g(\omega)e^{i\omega t}$ with frequency ω, the displacements and forces would also behave harmonic, and the Eq. (1) can be expressed as

$$[-\omega^2\mathbf{M} + \mathbf{K}(1 + 2\beta_d i)]\mathbf{r} = -\mathbf{M}\mathbf{J}\mathbf{a}_g(\omega) + \begin{bmatrix} \mathbf{0} \\ \mathbf{R}_b(\omega) \end{bmatrix} \tag{2}$$

where β_d is the constant hysteretic damping factor for the dam body, and the force vector is partitioned into degrees of freedom above the dam-foundation interface and those on the interface. The first part being a null vector because of an empty reservoir consideration.

2.2 Foundation impedance matrix

The foundation surface can be discretized by boundary elements. It is well known that for a time harmonic excitation, the nodal displacements and tractions of this domain for the scattered field are related by the following matrix equation (Gaitanaros and Karabalis 1988)

$$\sum_{(f+q)} \mathbf{H}^j(\mathbf{U}^j)^s = \sum_{(f+q)} \mathbf{G}^j(\mathbf{P}^j)^s \tag{3}$$

in which $\mathbf{H}^j$, $\mathbf{G}^j$ are the usual matrices obtained in the boundary element formulations (Gaitanaros and Karabalis 1988). It should be noted that $\mathbf{H}^j$, $\mathbf{G}^j$ are contributions for the jth element, and they are obtained by considering the source point at all nodes related to foundation rock boundary element mesh. Therefore, assuming that there are N nodes and second order 8-node boundary elements are utilized, the size of these matrices would be $3N \times 24$. Meanwhile, it should be mentioned that summation in the above relation is over two domains f, q corresponding to dam-foundation interface (zone f) and the rest of the foundation rock surface (zone q), respectively. The latter part includes portions of the rock surface at the half space surface and the canyon wall discretized. Furthermore, $(\mathbf{U}^j)^s$, $(\mathbf{P}^j)^s$ are the scattered field nodal displacement and traction vectors for element j, defined as the difference of the total and free field motions of the corresponding element

$$(\mathbf{U}^j)^s = \mathbf{U}^j - (\mathbf{U}^j)^f \tag{4}$$

$$(\mathbf{P}^j)^s = \mathbf{P}^j - (\mathbf{P}^j)^f \tag{5}$$

In general, there could be different traction quantities at each node for various boundary elements connected to that node. However, there are unique displacement quantities at each node. Therefore, $\mathbf{H}^j$ matrices could be assembled and form the total $\mathbf{H}$ matrix. In that case, the relation (3) is written as

$$\mathbf{H}\mathbf{U}^s = \sum_{(f+q)} \mathbf{G}^j(\mathbf{P}^j)^s \tag{6}$$

In the above equation, the scattered displacement vector corresponds to the whole boundary element domain, and it can be defined in terms of the total and free field displacement vectors of the whole domain, similar to relation (4)

$$\mathbf{U}^s = \mathbf{U} - \mathbf{U}^f \tag{7}$$

Assuming uniform free field motion, the free field displacement vector is written as a combination of rigid body excitations in three directions

$$\mathbf{U}^f = \mathbf{J}\mathbf{u}_g(\omega) \tag{8}$$

In this relation, $\mathbf{u}_g(\omega)$ is a vector composed of three components of ground excitation. By substituting Eq. (8) into Eq. (7), it would be obvious that scattered displacement vector is the same as the vector of relative nodal displacement vector.

$$\mathbf{U}^s = \mathbf{r} = \mathbf{U} - \mathbf{J}\mathbf{u}_g(\omega) \tag{9}$$

Furthermore, by assuming uniform free field excitations, it is clear that the corresponding element traction vectors are null vectors

$$(\mathbf{P}^j)^f = \mathbf{0} \tag{10}$$

By employing relations (5), (9) and (10), the Eq. (6) would be

$$\mathbf{H}\mathbf{r} = \sum_{(f+q)} \mathbf{G}^j\mathbf{P}^j \tag{11}$$

Denoting the nodal degrees of freedom of the foundation at the dam-foundation interface by f (related to zone f), and the rest by q, the relation (11) can be written in the following partitioned form

$$[\mathbf{H}_f \ \ \mathbf{H}_q]\begin{bmatrix}\mathbf{r}_f \\ \mathbf{r}_q\end{bmatrix} = \mathbf{G}_f\mathbf{P}_f + \sum_q \mathbf{G}^j\mathbf{P}^j \tag{12}$$

In the above relation, it is assumed that dam-foundation interface is relatively smooth, which leads to adopt an average unknown traction value for each direction at a certain node. Therefore, the $\mathbf{G}^j$ matrices are also assembled over the domain (f) to form the $\mathbf{G}_f$ matrix.

Having in mind that the total traction vectors at the free surface are zero for elements j which correspond to the domain (q), the Eq. (12) can be rearranged as below

$$[-\mathbf{G}_f \ \ \mathbf{H}_q]\begin{bmatrix}\mathbf{P}_f \\ \mathbf{r}_q\end{bmatrix} = -\mathbf{H}_f\mathbf{r}_f \tag{13}$$

If one considers $\mathbf{r}_f$ as being a vector corresponding to a unit displacement for one of the degrees of freedom and zero for all others, this means that the corresponding tractions vector is obtained through Eq. (13) by considering the right hand side as the negative of the corresponding column of matrix $\mathbf{H}_f$. Therefore in general, for an arbitrary $\mathbf{r}_f$ vector, one can say that the tractions vector, is calculated by the following relation

$$\mathbf{P}_f = \mathbf{Q}\,\mathbf{r}_f \tag{14}$$

in which $\mathbf{Q}$ is a matrix obtained through a modified version of (13) as

$$[-\mathbf{G}_f \ \ \mathbf{H}_q]\begin{bmatrix}\mathbf{Q} \\ \mathbf{r}'\end{bmatrix} = -\mathbf{H}_f \tag{15}$$

It should be also mentioned that $\mathbf{r}'$ in the above relation is a matrix form of the $\mathbf{r}_q$ vector. This is due to the fact that the right hand side of that equation is now a matrix itself. The integration of the tractions at the dam-foundation interface can be achieved by the help of the $\mathbf{B}$ matrix, which yields the vector of nodal forces at that interface ($\mathbf{R}_f$). The form of the $\mathbf{B}$ matrix is well known as it is quite often written in the context of finite elements by using shape functions. Thus, by employing Eq. (14), the vector $\mathbf{R}_f$ would be

$$\mathbf{R}_f = \mathbf{B}\mathbf{Q}\,\mathbf{r}_f \tag{16}$$

The matrix Eq. (16) can also be written in the familiar form

$$\mathbf{R}_f = \hat{\mathbf{S}}_f\mathbf{r}_f \tag{17}$$

in which $\hat{\mathbf{S}}_f$ is the foundation impedance matrix defined as below

$$\hat{\mathbf{S}}_f = \mathbf{B}\mathbf{Q} \tag{18}$$

The foundation impedance matrix determined by Eq. (18), will not be exactly symmetric owing to the approximations inherent in the boundary element procedures, however, it is nearly symmetric. By minimizing the differences in the unsymmetric off-diagonal terms in a least square sense (Tan and Chopra 1995a), a symmetric impedance matrix is given by the following equation.

$$\mathbf{S}_f(\omega) = 1/2(\hat{\mathbf{S}}_f(\omega) + \hat{\mathbf{S}}_f^T(\omega)) \tag{19}$$

2.3 Dam-foundation rock system

In previous sections, the necessary equations for the two regions of dam and foundation rock, was obtained by finite and boundary element discretization respectively. To combine the equations, we note that the equilibrium of the interaction forces between the dam and foundation rock at the interface requires that

$$\mathbf{R}_b(\omega) + \mathbf{R}_f(\omega) = \mathbf{0} \tag{20}$$

and compatibility of interaction displacements at the interface also requires that

$$\mathbf{r}_b(\omega) = \mathbf{r}_f(\omega) \tag{21}$$

By utilizing relations (21), and the one obtained by replacing the symmetric form of foundation impedance matrix Eq. (19) in Eq. (17), the relation (20) becomes

$$\mathbf{R}_b(\omega) = -\mathbf{S}_f \mathbf{r}_b(\omega) \tag{22}$$

This relation can be employed in Eq. (2) to obtain

$$[-\omega^2\mathbf{M} + \mathbf{K}(1 + 2\beta_d i) + \overline{\mathbf{S}}_f(\omega)]\mathbf{r} = -\mathbf{MJa}_g(\omega) \tag{23}$$

in which $\overline{\mathbf{S}}_f$ is defined below.

$$\overline{\mathbf{S}}_f(\omega) = \begin{bmatrix} \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \mathbf{S}_f(\omega) \end{bmatrix} \tag{24}$$

The relation (23) is a system of equations, which can be solved for nodal displacements vector at specified frequencies for different forms of ground accelerations vector $\mathbf{a}_g(\omega)$ corresponding to upstream, vertical or cross-canyon excitations.

It should be also mentioned that in this process, most of the computational time is spent for calculation of foundation impedance matrix at each frequency. However, this could be remedied by calculating the impedance matrix only at certain frequency points and interpolating this matrix for intermediate frequencies similar to the work of Tan and Chopra (1995a). In the present study, 25 equally spaced frequency points are used and cubic interpolation scheme is implemented.

Meanwhile, it should be noted that as a special case, the dam-foundation rock interaction could be applied partially by considering the foundation flexibility only. In these cases, the foundation impedance matrix in relation (23) must be replaced by static stiffness matrix of foundation rock, which is calculated once. This is obtained similarly by applying relation (3) with the exception that elasto-static fundamental solution must be utilized for calculation of $\mathbf{H}^j$ and $\mathbf{G}^j$ matrices. Furthermore, the boundary element discretization must be closed for this case in contrast to an open domain used for calculation of foundation impedance matrix (Dominguez 1993).

3. Modeling and basic parameters

A computer program (Lotfi 2001) was enhanced based on the theory presented on the previous section. In this program, the dam body and the foundation rock, are treated by finite and boundary elements, respectively. Both domains are considered as linearly viscoelastic materials with isotropic behavior.

3.1 Models

An idealized symmetric model of Morrow Point arch dam is considered. The geometry of the dam may be found in Hall and Chopra (1983). The main model is prepared based on the finite and boundary elements discretization for the dam body and foundation rock respectively, which is depicted in Fig. 1. The dam is discretized by 40 isoparametric 20-node finite elements, while the foundation rock is modeled by 178 isoparametric 8-node boundary elements considered at the foundation surface. In this model, the canyon shape is also assumed uniform (FE-BE model with uniform canyon shape).

A second model is also considered in which both the dam and the foundation rock is discretized by finite elements. In this case, 40 and 276 isoparametric 20-node elements are utilized for the dam and foundation rock, respectively (FE-FE model with uniform canyon shape). This is used to verify the previous model in the special case where foundation flexibility is only included.

Finally, the third model is similar to the first model except that a non-uniform canyon shape is assumed. This could be utilized to study the effects of canyon shape on the response.

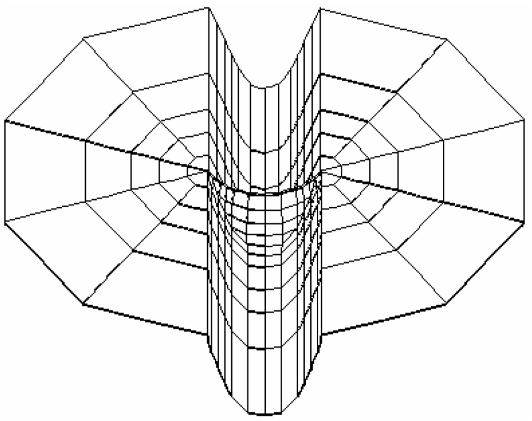


Fig. 1(a) Dam-foundation system discretization (model 1: FE-BE model with uniform canyon shape)

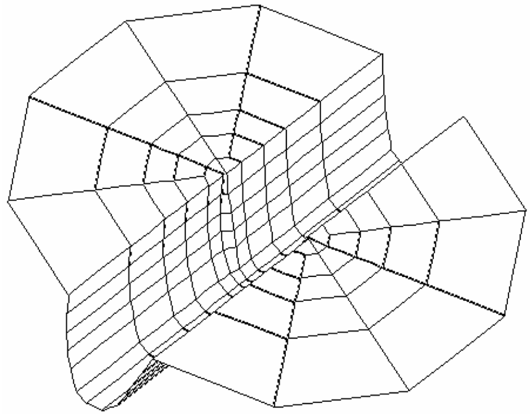


Fig. 1(b) Boundary element discretization of foundation rock for model 1

3.2 Basic parameters

The dam concrete is assumed to be homogeneous with isotropic linearly viscoelastic behavior and the following main characteristics:

Elastic modulus (E_d) = 27.5 GPa
Poisson's ratio = 0.2
Unit weight = 24.8 kN/m^3
Hysteretic damping factor (β_d)= 0.05

The foundation rock, is idealized by a homogeneous, viscoelastic domain. The basic properties of this region are:

Poisson's ratio = 0.2
Unit weight = 26.4 kN/m^3
Hysteretic damping factor (β_f) = 0.05

As for the foundation rock elastic modulus (E_f), it was varied to cover a wide range of foundation materials. In particular, E_f/E_d ratios of ∞ (rigid foundation), 2, 1, 0.5, and 0.25 are considered in the initial part of the investigations. In later stages, the elastic moduli ratio was fixed as E_f/E_d= 1, to compare the FE-BE and FE-FE procedures, as well as to study the effects of canyon shape on the response.

4. Results

The FE-BE model with uniform canyon shape (Fig. 1) is considered initially. The responses of dam crest are obtained due to upstream, vertical and cross-stream excitations (Figs. 2-4) and in each case for several ratios of foundation rock to dam concrete elastic moduli (E_f/E_d).

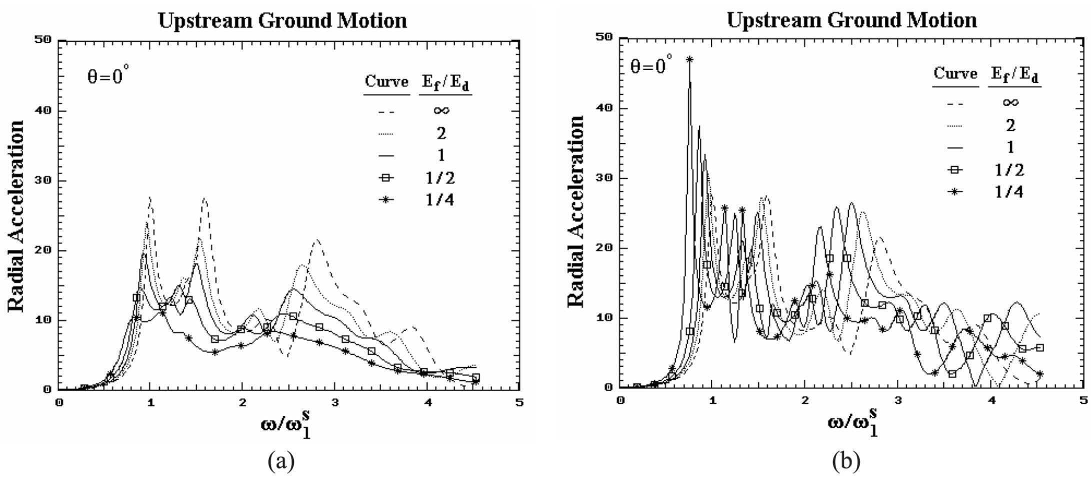
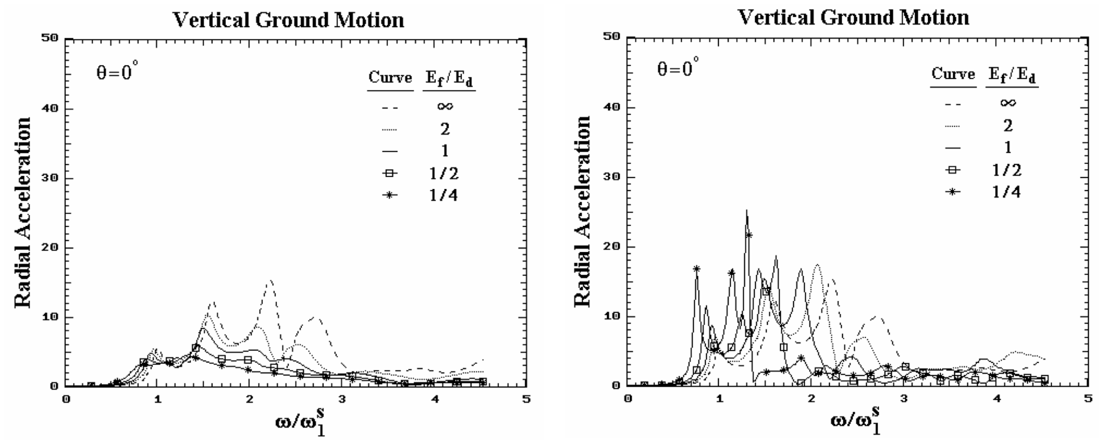
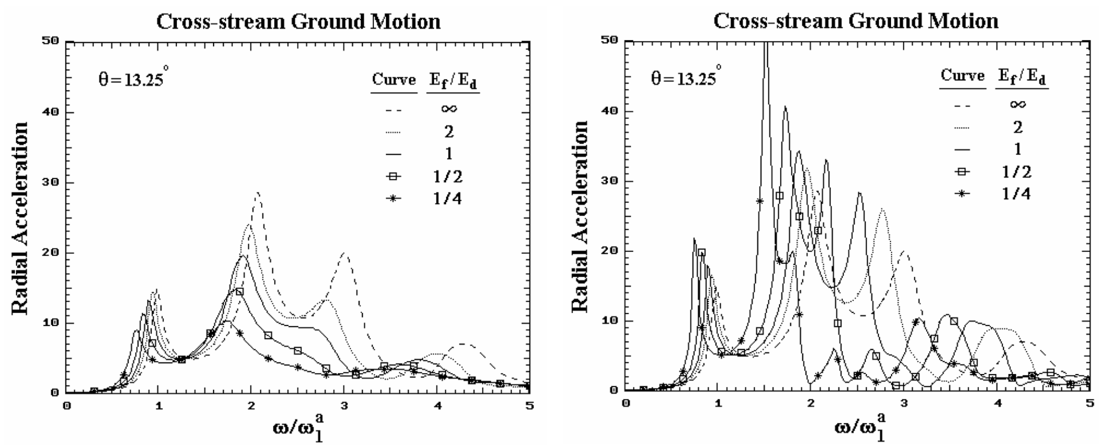


Fig. 2 Influence of moduli ratio E_f/E_d on response of dam due to harmonic upstream ground motion: (a) complete interaction, (b) foundation flexibility only



(a) (b)

Fig. 3 Influence of moduli ratio E_f/E_d on response of dam due to harmonic vertical ground motion: (a) complete interaction, (b) foundation flexibility only



(a) (b)

Fig. 4 Influence of moduli ratio E_f/E_d on response of dam due to harmonic cross-stream ground motion: (a) complete interaction, (b) foundation flexibility only

The response quantities plotted are the amplitudes of the complex valued radial accelerations for two points located at dam crest. These are either the mid-crest point ($\theta = 0^o$) selected for upstream or vertical excitations or a point located at ($\theta = 13.25^o$) which is used for the case of cross-stream excitation. This is due to the fact that radial acceleration is diminished at mid-crest for the cross-stream type of ground motion.

In each case, the amplitude of radial acceleration is plotted versus the dimensionless frequency for a significant range. The dimensionless frequency for upstream and vertical excitation is defined as ω/ω_1^S where ω is the excitation frequency and ω_1^S is the fundamental frequency of the dam on rigid foundation with empty reservoir for a symmetric mode. For the cross-stream excitation cases, the dimensionless frequency is defined as ω/ω_1^a, where ω_1^a is the fundamental resonant frequency of

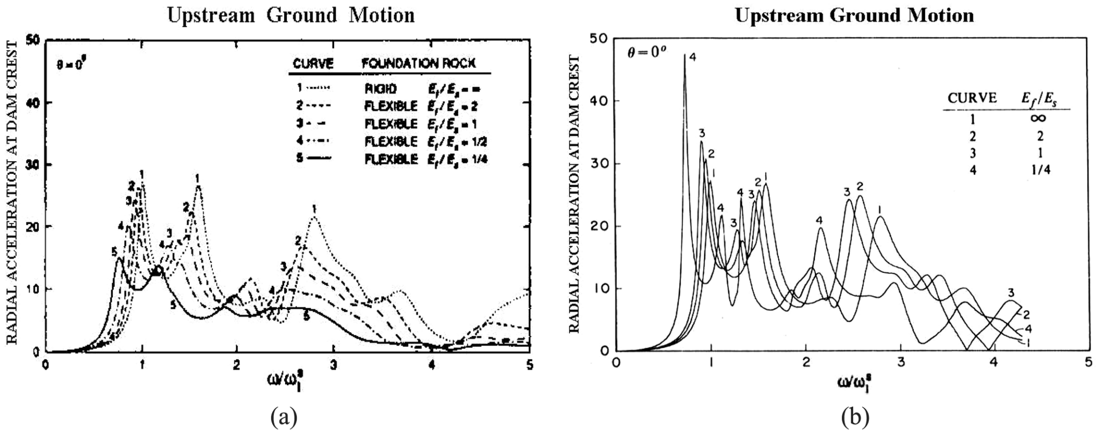


Fig. 5 Influence of moduli ratio E_f/E_d on response of dam due to harmonic upstream ground motion: (a) complete interaction (result taken from the work of Tan and Chopra (1995) for comparison), (b) foundation flexibility only (result taken from the work of Fok and Chopra (1986) for comparison)

the dam on rigid foundation with empty reservoir for an anti-symmetric mode.

It should be noted that for each type of excitation, the response is obtained for two different types of dam-foundation rock interaction treatment. The one that complete interaction is imposed (normal FE-BE procedure), and the case where interaction is treated partially by considering the foundation flexibility only (special case of FE-BE procedure).

It is observed that when complete interaction is applied (Figs. 2(a), 3(a) and 4(a)), the natural frequencies of the system, as well as the magnitude of the peaks of the response decrease as the ratio of foundation rock to dam concrete elastic moduli (E_f/E_d) reduces. The decrease in the response is due to radiation damping phenomena, which is caused by energy taken from system and traveling to far-field.

In other cases, the interaction is treated partially by including the foundation flexibility only. In these cases (Figs. 2(b), 3(b) and 4(b)), the reduction in the natural frequencies of the system is still present. However, the amplitudes of the peaks increase as the E_f/E_d ratio decreases.

Similar results taken from works of Fok and Chopra (1986) and Tan and Chopra (1995), are also illustrated in Fig. 5 for comparison purposes in the case of upstream ground motion. These results could be compared with the corresponding results of the present study (Fig. 2).

It is noticed that trend is similar and relatively good agreement exists between the results obtained herein (Fig. 2) and the ones taken from the above mentioned references (Fig. 5). The main difference is the amplitude of the peaks which are slightly higher in the work of Tan and Chopra (Fig. 5(a)) in comparison with the present study results related to the case of complete interaction treatment (Fig. 2(a)). This is mainly due to the fact that in that work, the foundation impedance matrix is obtained by applying a fine mesh and implementing static condensation to condense the extra degrees of freedom.

At this stage, it could be claimed that the modified computer program (Lotfi 2001) and the solution procedures are verified. However, to be certain, it was also decided to compare the FE-BE procedure with the FE-FE approach. Of course, this comparison must be carried out for the case where foundation flexibility is only included, otherwise there would be distortions in the results of FE-FE approach due to wave reflections at the boundaries. For this purpose, the second model (FE-

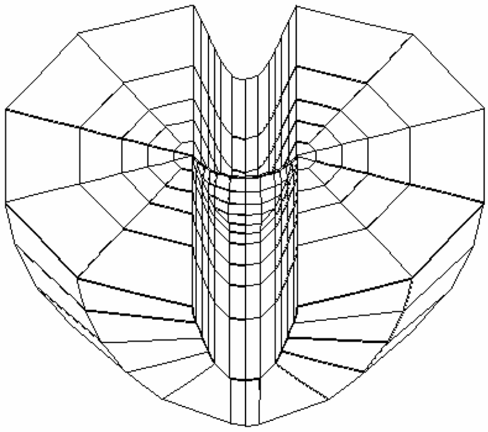


Fig. 6 Dam-foundation system discretization (model 2: FE-FE model with uniform canyon shape)

FE model with uniform canyon shape) is constructed (Fig. 6). The results of this case are compared against the FE-BE procedure in Fig. 7 for different types of excitation. It should be mentioned that for these results, the elastic moduli ratio $E_f/E_d = 1$ is selected, and only foundation flexibility is included in the analyses for the reasoning explained above.

It is observed that perfect agreement exist in the response of both procedures (Fig. 7), and this can be taken as an additional verification of the developed program and the solution procedures.

Finally, it was decided to show the effects of canyon shape on the response. For this aim, the third model is defined (FE-BE model with non-uniform canyon shape). This model is exactly similar to the first model, except from the geometry of canyon standpoint (Fig. 8). In this case, it is assumed that the topography at the crest level makes an angle of 60° with the stream direction. This is in both upstream and downstream directions at both left and right banks. At lower elevations, this

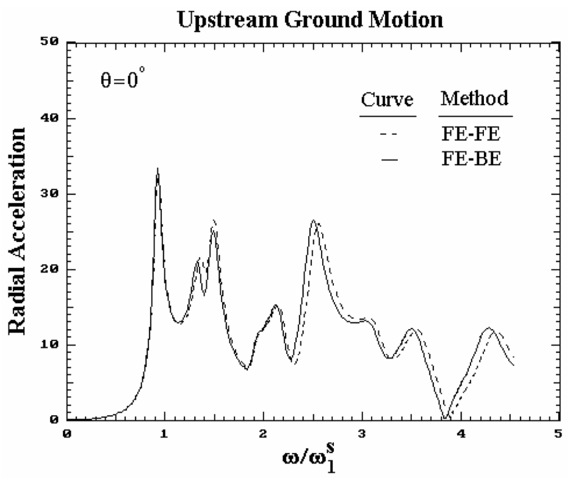


Fig. 7(a) Comparison of response at dam crest for FE-BE and FE-FE procedures due to upstream ground motion (foundation flexibility is only included)

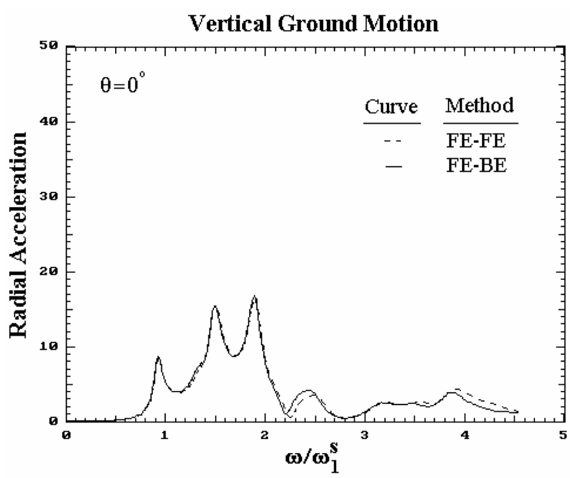


Fig. 7(b) Comparison of response at dam crest for FE-BE and FE-FE procedures due to vertical ground motion (foundation flexibility is only included)

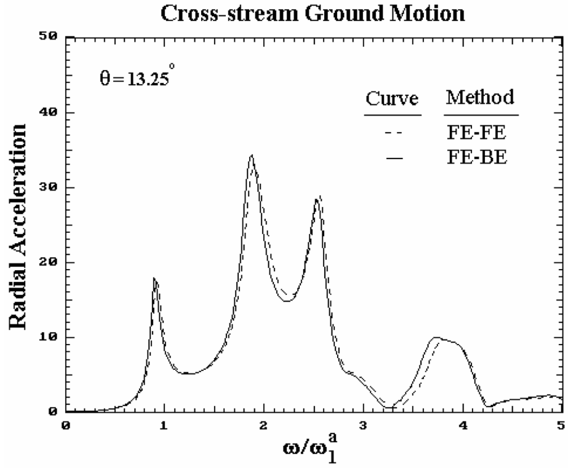


Fig. 7(c) Comparison of response at dam crest for FE-BE and FE-FE procedures due to cross-stream ground motion (foundation flexibility is only included)

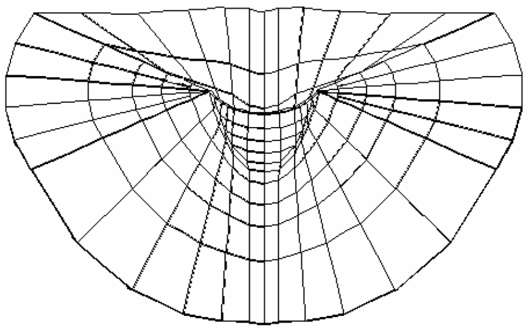


Fig. 8(a) Dam-foundation system discretization (model 3: FE-BE model with non-uniform canyon shape)

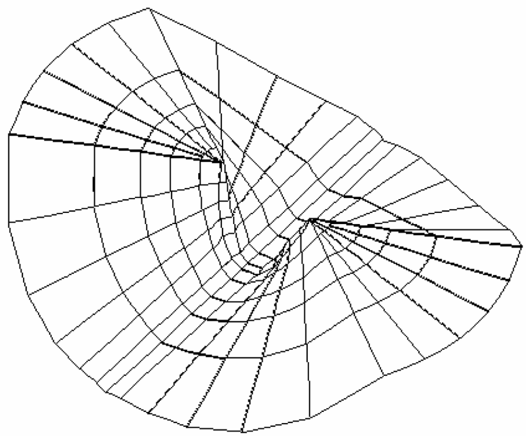
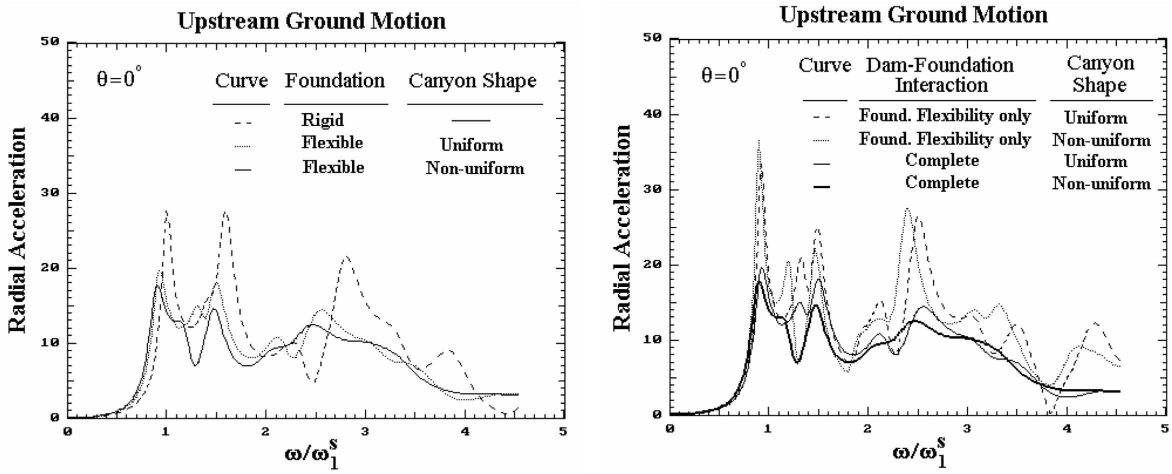


Fig. 8(b) Boundary element discretization of foundation rock for model 3

angle is reduced based on a quadratic function of height, such that it becomes zero at the base of the dam.

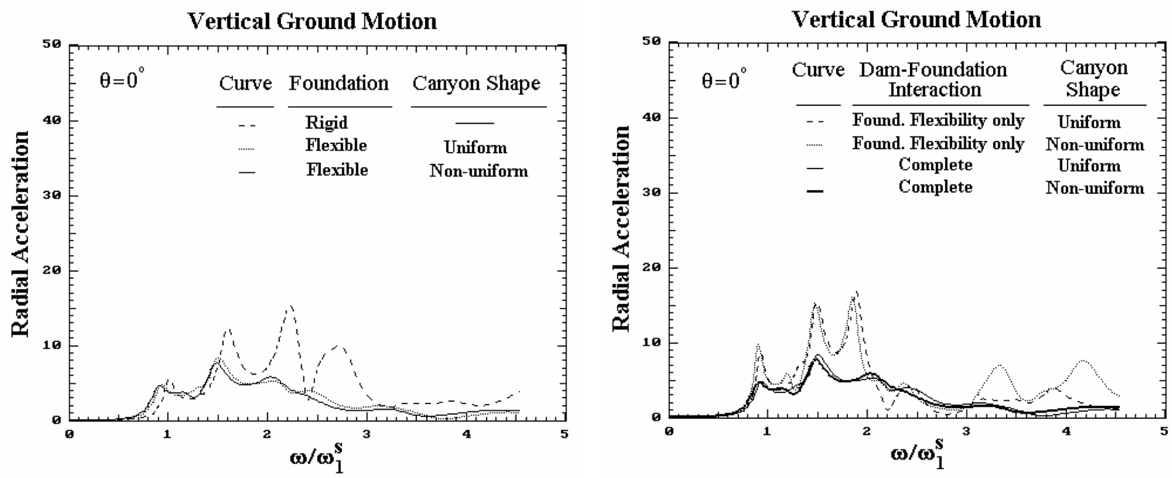
The results for uniform and non-uniform canyon shapes (model 1 versus model 3 for $E_f/E_d = 1$) are compared in Figs. 9-11 for different types of excitation. It should be also noted that in each figure, two graphs are presented. The Fig. 8(a) displays the effects of canyon shape when complete interaction is included in both models, and the response for the rigid foundation case is also given as a reference. The Fig. 8(b) includes the responses of both models for cases of complete interaction treatment as well as those in which foundation flexibility is only considered.

It is observed that the response is significantly affected due to change in canyon shape when complete interaction is considered (Figs. 9(a), 10(a) and 11(a)). The peaks of the response for the non-uniform canyon shape is lower than for the uniform canyon shape, and the natural frequencies



(a) (b)

Fig. 9 Influence of canyon shape on response of dam due to upstream ground motion: (a) comparison for complete interaction, (b) comparison for both complete interaction and foundation flexibility only

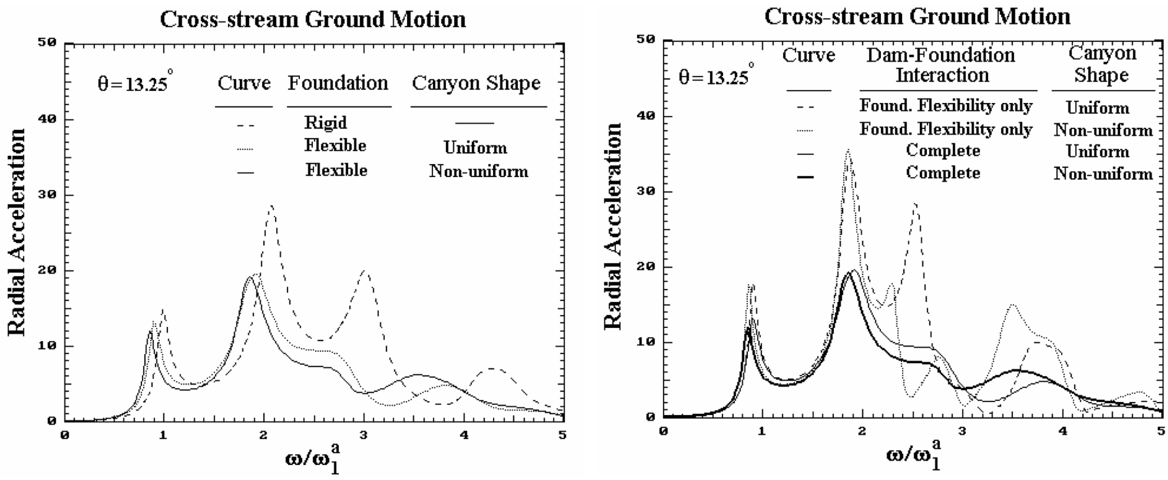


(a) (b)

Fig. 10 Influence of canyon shape on response of dam due to vertical ground motion: (a) comparison for complete interaction, (b) comparison for both complete interaction and foundation flexibility only

of the system are also reduced. This is because, the foundation domain has become more flexible as a result of thinner abutment, and the behavior is similar to reducing the foundation elastic modulus or decreasing the elastic moduli ratio (E_f/E_d). Of course, this decrease in the magnitude of the peaks is more pronounced for the upstream and cross-stream excitations than for vertical ground motion.

One can also observe from Figs. 9(b), 10(b) and 11(b) that when only foundation flexibility is included, the peak corresponding to the first symmetric or anti-symmetric mode, increases for the non-uniform canyon shape (thinner abutment) in comparison to the uniform canyon shape. From the same figures, it is noticed that the behavior is opposite when complete dam-foundation rock interaction is considered. Furthermore, it is observed that regardless of the canyon shape, the partial



(a) (b)

Fig. 11 Influence of canyon shape on response of dam due to cross-stream ground motion: (a) comparison for complete interaction, (b) comparison for both complete interaction and foundation flexibility only

interaction treatment (Foundation flexibility only) can model the shift in the natural frequencies correctly. However, the amplitude of the response is not estimated correctly, and the trend in change of response is actually opposite to the true behavior. The one obtained when complete dam-foundation rock interaction is considered.

5. Conclusions

The formulation based on FE-BE procedure for dynamic analysis of concrete arch dam-foundation rock systems, was explained. A computer program based on this methodology was prepared and the response of Morrow Point arch dam was studied for various ratios of foundation rock to dam concrete elastic moduli, as well as different shapes of canyon and assumptions in regard to dam-foundation rock interaction. Overall, the main conclusions obtained by the present study can be listed as follows:

- The initial results obtained herein shown to be in good agreement with the work of Tan and Chopra (1995) when complete dam-foundation rock interaction is included, as well as the work of Fok and Chopra (1986) for the special cases occurring when only foundation flexibility is considered. Meanwhile, similar observations as to previous studies due to effects of dam-foundation rock interaction are also noticed in the present results.
- The special case of the present FE-BE procedure (when only foundation flexibility is included) is also compared against a consistent FE-FE approach. It was observed that perfect agreement exist in the response of dam based on both methods, and this can be taken as an additional verification of the developed program and solution procedures.
- The main advantage of the FE-BE procedure of the present study over the method of Tan and Chopra (1995) is that there are no restrictions imposed as to the geometric shape of the canyon.
- The results are also compared for two cases having uniform and non-uniform canyon shapes. It is observed that the response is significantly affected due to change in canyon shape when

complete interaction is considered. The peaks of the response for the non-uniform canyon shape (thinner abutment) is lower than for the uniform canyon shape, and the natural frequencies of the system are also reduced. The behavior is similar to reducing the foundation elastic modulus or decreasing the elastic moduli ratio (E_f/E_d). Of course, this decrease in the magnitude of the peaks is more pronounced for the upstream and cross-stream excitations than for vertical ground motion. When only foundation flexibility is included, the peak corresponding to the first symmetric or anti-symmetric mode, increases for the non-uniform canyon shape (thinner abutment) in comparison to the uniform canyon shape. This behavior is opposite when complete dam-foundation rock interaction is considered.

- Regardless of the canyon shape, the partial interaction treatment (Foundation flexibility only) can model the shift in the natural frequencies correctly. However, the amplitude of the response is not estimated correctly, and the trend in change of response is actually opposite to the true behavior. The one obtained when complete dam-foundation rock interaction is considered.

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