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Transient stochastic analysis of nonlinear response of earth and rock-fill dams to spatially varying ground motion

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Abstract. The main purpose of this paper is to investigate the effect of transient stochastic analysis on nonlinear response of earth and rock-fill dams to spatially varying ground motion. The dam models are analyzed by a stochastic finite element method based on the equivalent linear method which considers the nonlinear variation of soil shear moduli and damping ratio as a function of shear strain. The spatial variability of ground motion is taken into account with the incoherence, wave-passage and site response effects. Stationary as well as transient stochastic response analyses are performed for the considered dam types. A time dependent frequency response function is used throughout the study for transient stochastic responses. It is observed that stationarity is a reasonable assumption for earth and rock-fill dams to typical durations of strong shaking.

Keywords: spatially varying ground motion; stationary response; transient response; equivalent linear method; earth-fill dam; rock-fill dam; stochastic response.

1. Introduction

For many years, considerable research has been directed towards the analytical and numerical techniques for estimating the response of earth and rock-fill dams subjected to earthquake ground motions. It is well known that the large structures such as earth and rock-fill dams obviously are subjected to different ground motions at its foundations. Spatial variation of earthquake ground motion, which includes the incoherence, wave passage and site response effects, has caused concern about the safety of the earth and rock-fill dams under seismic excitation. The effect of spatially varying ground motion (SVGM) on the response of the fill dams have been analyzed in the past few years. Dumanoglu and Severn (1984) carried out dynamic response of earth dams and other structures to differential ground motions. It was observed that for asynchronous dynamic analysis the velocity of the ground motion greatly influences the responses of earth dams. Haroun and Abdel-Hafız (1987) performed a parametric study to outline the effects of spatially varying nonuniform, in-phase and out-of phase ground motions on the response of earth dams by using a finite element method. Dakoulas and Hashmi (1992) presented an analytical model for steady-state lateral response of earth-fill dams in canyons subjected to asynchronous excitation consisting of SH

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wave's incident at an arbitrary angle. Using the finite element method, Maugeri *et al.* (1993) studied the dynamic response of earth dams to travelling seismic waves along the transverse direction of earth dams. Dakoulas and Hsu (1995) developed a new analytical solution for steady-state lateral response of earth and rock-fill dams built in semi elliptical canyons and subjected to obliquely incident harmonic SH waves. Comparisons were made with the response of the dam to rigid-base synchronous excitation. Chen and Harichandran (2001) analyzed the stochastic response of the Santa Felicia earth dam to spatially varying earthquake ground motion model including the incoherence and propagation of seismic waves. Haciefendioglu *et al.* (2004) investigated the influence of the incoherence effect on the stochastic response of nonlinear response of embankment dams induced by the spatially varying ground motions.

A classical stochastic method is applied only to linear elastic systems. However, the linear model may not be always enough to predict the response of earth and rock-fill dams during earthquake motions. Therefore, material nonlinearity must be included in the analyses. Faccioli (1976) developed an equivalent linearization random vibration formulation to investigate the one-dimensional amplification of soil deposits. Singh and Khatua (1978) studied to determine the seismic stability of earth dams by using stochastic linearization technique and performing iterative analysis which is related to finite element method. Gazetas *et al.* (1982) have developed a new method to analyze the stochastic response of nonlinear systems subjected to earthquake excitation. Haciefendioglu *et al.* (2004) computed stochastic dynamic analyses of linear and nonlinear responses of earth-fill dams due to the uniform ground motion case.

Whereas the solutions of the studies above mentioned are based on the stationary behavior of earthquake motions, this paper presents the results obtained from the stationary and transient stochastic analysis of nonlinear response of an earth-fill dam and a rock-fill dam under spatially varying ground motion models with using the equivalent linear method. The equivalent linear method is used to estimate the nonlinear hysteretic response of earth and rock-fill dams to stochastic excitation characterized by the filtered Kanai-Tajimi spectrum (Clough and Penzien 1993). Stationary and transient responses of an earth-fill dam and a rock-fill dam are calculated and shear strains, displacements and stresses are compared throughout this study.

2. Numerical method for stochastic analysis of nonlinear response

The equivalent linear method (Idriss *et al.* 1973) is carried out to represent the strain-dependent transient stochastic analysis of nonlinear response of earth and rock-fill dams to spatially varying ground motions. Nonlinearity of earth and rock-fill dam materials is approached using an iterative procedure. In this method, approximate nonlinear solutions can be obtained by series of linear analyses provided the updated stiffness and damping are compatible with current effective shear strain level. The equivalent effective strain is estimated as a fraction (i.e., 1.0 for stochastic analysis) of peak shear strain in order to define modulus and damping ratio for the each iteration from the experimentally achieved curves (Gazetas *et al.* 1982). Successive iterations are required until compatible dynamic parameters with strain level are acquired. The response of the last iteration is taken as an approximation to the nonlinear response.

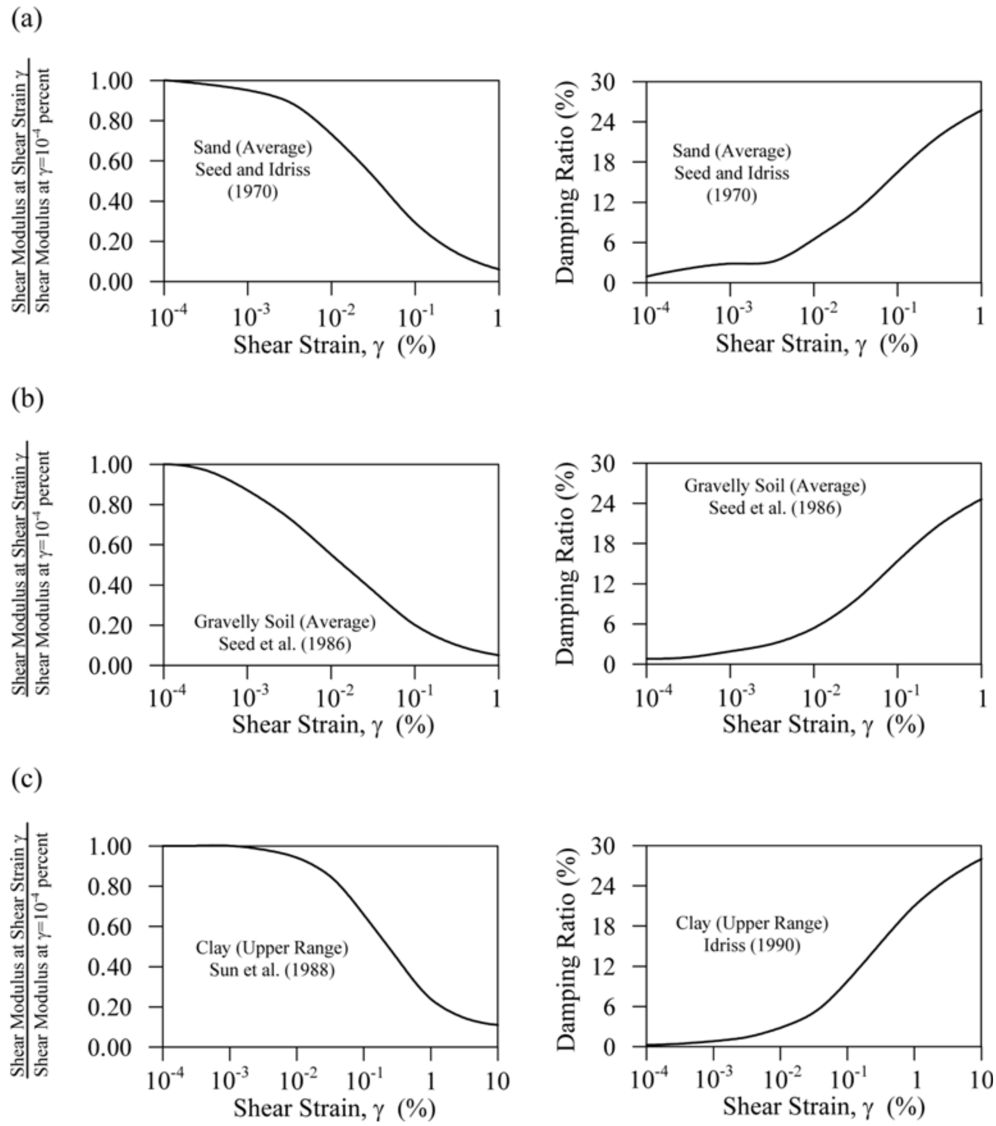

Fig. 1 The variation of shear modulus and damping ratios for (a) sand, (b) gravelly soil and (c) clay materials, respectively

2.1 Material properties

The dynamic behaviors of earth and rock-fill elements are described as the small strain shear modulus, $G_{\max}$, the decrease of secant modulus G with increasing strain γ, the hysteric damping ratio, β, which increase with increasing shear strain and Poisson's ratio ν. Experimental data from the literature on shear strain dependent moduli and damping for sand and rock-fill (gravel) materials are depicted Fig. 1(a)-(b) (Seed *et al.* 1986, Seed and Idriss 1970). $G_{\max}$ is estimated as a function of the effective confining pressures of cohesionless materials:

$$G_{\max} = 1000(K_2)_{\max}(\sigma_m)^{1/2} \tag{1}$$

Values of $(K_2)_{\max}$ (square-root of stress) determined by laboratory tests have been found to vary from 30 to about 75 for loose sands and from 90 to 188 for dense sands and is in the range of 150-250 for compacted gravels and rock-fill. Shear modulus values for saturated cohesive soils have been found to vary with the undrained shear strength level as

$$G = 2000 \cdot s_u \tag{2}$$

where $s_u = c + \sigma_m \cdot \tan\phi$, is the undrained shear strength, c is the cohesion factor and ϕ is the angle of internal friction. The variations of shear modulus and damping ratios with shear strain for clay materials are presented in Fig. 1(c) (Sun *et al.* 1988, Idriss 1990).

3. Finite element modelling and random vibration theory

The dynamic equations of motion of a structure discretized using the finite element method may be written in the partitioned form;

$$\begin{bmatrix} M_{rr} & M_{rg} \\ M_{gr} & M_{gg} \end{bmatrix} \begin{Bmatrix} \ddot{u}_r \\ \ddot{u}_g \end{Bmatrix} + \begin{bmatrix} C_{rr} & C_{rg} \\ C_{gr} & C_{gg} \end{bmatrix} \begin{Bmatrix} \dot{u}_r \\ \dot{u}_g \end{Bmatrix} + \begin{bmatrix} K_{rr} & K_{rg} \\ K_{gr} & K_{gg} \end{bmatrix} \begin{Bmatrix} u_r \\ u_g \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \tag{3}$$

Where $[M]$, $[C]$, $[K]$ are the mass, damping and stiffness matrices, respectively; $\{\ddot{u}\}, \{\dot{u}\}, \{u\}$ are the vectors of total accelerations, velocities and displacements, respectively. The subscript r denotes the response degrees of freedom and g denotes the ground degrees of freedom. It is possible to separate the total displacement vectors as quasi-static and dynamic parts as follow;

$$\{u_r\} = \{u_{sr}\} + \{u_{dr}\} \tag{4}$$

The quasi-static displacements of the structure may be obtained from Eq. (3) by not considering the first two terms on the left-hand side of the equation an replacing $\{u_r\}$ by $\{u_{sr}\}$:

$$\{u_{sr}\} = -[K_{rr}]^{-1}[K_{rg}]\{u_{sg}\} = [R_{rg}][u_{sg}] \tag{5}$$

in which $[R_{rg}] = -[K_{rr}]^{-1}[K_{rg}]$. Substituting Eqs. (4) and (5) in to Eq. (3), the equations of motion of the dynamic component of the response degrees of freedom can be written as

$$[M_{rr}]\{\ddot{u}_{dr}\} + [C_{rr}]\{\dot{u}_{dr}\} + [K_{rr}]\{u_{dr}\} = -[M_{rr}][R_{rg}]\{\ddot{u}_{sg}\} \tag{6}$$

Using the well-known modal analysis approach and letting $\{u_{dr}\} = [\phi]\{Y\}$ decouples the above equations to yield

$$\ddot{Y}_i + 2\xi_i\omega_i\dot{Y}_i + \omega_i^2 Y_i = G_i \tag{7}$$

in which Y_i are generalized displacements, ω_i and ξ_i are the natural frequencies and modal damping ratios and $G_i = (\Gamma_i)^T\{\ddot{u}_g\}$ are the modal loads. The modal participation factors are given by

$$\{\Gamma_i\} = [M_{rr}][R_{rg}]\{\phi_i\} \tag{8}$$

3.1 Stationary response

The total mean-square responses can be computed from,

$$\sigma_z^2 = \sigma_{z_d}^2 + \sigma_{z_s}^2 + 2\mathrm{Cov}(z_s, z_d) \tag{9}$$

where $\sigma_{z_d}^2$ and $\sigma_{z_s}^2$ present the dynamic and pseudo-static variances, respectively, and $\mathrm{Cov}(z_s, z_d)$ is the covariance between the dynamic and pseudo-static responses. The three components above Eq. (9) are given by

$$\sigma_{z_d}^2 = \int_{-\infty}^{\infty} \sum_{j=1}^{n} \sum_{k=1}^{n} \sum_{l=1}^{r} \sum_{m=1}^{r} \psi_{ij}\psi_{ik}\Gamma_{lj}\Gamma_{mk}H_j(-\omega)H_k(\omega)S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)d\omega \tag{10}$$

$$\sigma_{z_s}^2(\omega) = \sum_{l=1}^{r} \sum_{m=1}^{r} A_{il}A_{im} \int_{-\infty}^{\infty} \frac{1}{\omega^4} S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)d\omega \tag{11}$$

$$\mathrm{Cov}(z_s, z_d) = -\sum_{j=1}^{n} \sum_{l=1}^{r} \sum_{m=1}^{r} \psi_{ij}A_{il}\Gamma_{mj} \int_{-\infty}^{\infty} \frac{1}{\omega^2} H_j(\omega)S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)d\omega \tag{12}$$

where, ψ are the eigenvector, Γ stands for the modal participation factor, $S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)$ is the cross spectral density function of accelerations between supports l and m, $H(\omega)$ is the frequency response function, n is the number of free degrees of freedom and r is the number of restrained degrees-of-freedom. A_{il} and A_{im} are equal to the static displacements for unit displacements assigned to each support point (Harichandran and Wang 1990). Frequency response function is defined as,

$$H_k(\omega) = \frac{1}{\omega_k^2 - \omega^2 + 2i\xi_k\omega_k\omega} \tag{13}$$

where ω_k is the modal circular frequency and ξ_k is the modal damping ratio.

3.2 Transient response

For some linear structures, it may be important to consider transient response due to the structure initially being at rest (e.g., long-period structures such as suspension bridges and offshore platforms), and/or short-duration excitations (e.g., earthquakes). The aim of this study is to investigate the effect of transient response of nonlinear systems such as earth and rock-fill dams. In random vibration analysis, statistical averages are assumed to be independent of time for stationary excitation. In fact, earthquake motions cannot be stationary, because they initially grow from zero, then have a steady phase and eventually decay. Transient response due to stationary excitation beginning at time $t = 0$ can be easily accommodated in to the framework developed for stationary response, and in many cases such a model is sufficient to assess the impact of non-stationary. One simple way to do this is to replace the modal frequency response functions $H_k(\omega)$ for stationary response, with a time dependent modal frequency response function $H_k(\omega, t)$ given by (Lin 1963, Caughey and Stumpf 1961)

$$H_k(\omega, t) = H_k(\omega)\left[1 - e^{-\xi_k \omega_k t} e^{-i\omega t}\left(\cos\omega_{kd} t + \frac{(\xi_k \omega_k + i\omega)}{\omega_{kd}} \sin\omega_{kd} t\right)\right] \tag{14}$$

where $\omega_{kd} = \omega_k \sqrt{1 - \xi_k^2}$ is the damped modal frequency. By using $H_k(\omega, t)$ at a given instant of time t in place of $H_k(\omega)$ in the stationary formulation, the spectral moment of the transient response at time t can be computed.

3.3 Mean of maximum value

Depending on the peak response and standard deviation (σ_z) of $z(t)$ the mean of maximum value, μ, in the stochastic analysis may be obtained from

$$\mu = p \cdot \sigma_z \tag{15}$$

Standard deviation of μ is defined as

$$\sigma = q \cdot \sigma_z \tag{16}$$

where p and q are peak factors (Wung and Der Kiureghian 1989).

4. Ground motion model

Because earth and rock-fill dams are large, extended structures, the ground motion characteristics at the different dam supports may change. Der Kiureghian and Neuenhofer (1991) identified three phenomena that are responsible for the spatial variation of the ground motion as; the "wave passage" effect, resulting from the difference in arrival times of seismic waves at different stations, the "incoherence" effect, resulting from the reflections and refractions of seismic waves in the ground and the "local-site" effect due to the differences in soil conditions at each station. These effects are characterized by the coherency function, which is the normalized cross-power spectral density function of motions at two different stations.

Spatial variability of the ground motion is characterised with the coherency function domain. The coherency function for the accelerations $\ddot{u}_{g_l}$ and $\ddot{u}_{g_m}$ at the support points l and m is written as

$$S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega) = \gamma_{lm}\omega\sqrt{S_{\ddot{u}_{g_l}\ddot{u}_{g_l}}(\omega) * S_{\ddot{u}_{g_m}\ddot{u}_{g_m}}(\omega)} \tag{17}$$

where $S_{\ddot{u}_{g_l}\ddot{u}_{g_l}}(\omega), S_{\ddot{u}_{g_m}\ddot{u}_{g_m}}(\omega)$ and $S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)$ indicate the auto-power spectral densities of the accelerations and their cross-spectral density, respectively. In the case of homogeneous ground for which $S_{\ddot{u}_{g_l}\ddot{u}_{g_l}}(\omega) = S_{\ddot{u}_{g_m}\ddot{u}_{g_m}}(\omega) = S_{\ddot{u}_g}(\omega)$, the previous expression can be reduced to

$$S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega) = \gamma_{lm}(\omega) \cdot S_{\ddot{u}_g}(\omega) \tag{18}$$

where $\gamma_{lm}(\omega)$ is the coherency function and $S_{\ddot{u}_g}(\omega)$ is the power spectral density function of uniform surface ground acceleration.

The coherency function proposed by Der Kiureghian (1996) can be written as

$$\gamma_{lm}(\omega) = \gamma_{lm}(\omega)^i \exp[i(\theta_{lm}(\omega)^w + \theta_{lm}(\omega)^s] \tag{19}$$

where $\gamma_{lm}(\omega)^i$, $\gamma_{lm}(\omega)^w$ and $\gamma_{lm}(\omega)^s$ indicate the incoherence, wave passage and site-response effects, respectively.

For the incoherence effect the model developed by Harichandran and Vanmarcke (1986), based on the statistical analysis of strong ground motion data from the SMART-1 dense array, is considered.

$$\gamma_{lm}(\omega)^i = A \cdot e^{\frac{-2d_{lm}}{\alpha\theta(\omega)}(1 - A + \alpha A)} + (1 - A) \cdot e^{\frac{-2d_{lm}}{\alpha\theta(\omega)}(1 - A + \alpha A)} \tag{20}$$

$$\theta(\omega) = k \cdot \left[1 + \left(\frac{\omega}{2\pi\omega_0}\right)^b\right]^{-\frac{1}{2}} \tag{21}$$

where d_{lm} is the distance between support points l and m. A, a, k, f_0 and b are model parameters and in this study the values obtained by Harichandran (1991) are used ($A = 0.636$, $\alpha = 0.0186$, $k = 31200$, $f_0 = 1.51$ Hz and $b = 2.95$).

The wave passage effect due to the differences in the arrival times of waves at support points is defined as

$$\theta_{lm}(\omega)^w = -\frac{\omega d_{lm}^L}{v_{app}} \tag{22}$$

where v_{app} is the apparent wave velocity and d_{lm}^L is the projection of d_{lm} of the ground surface along the direction of propagation of seismic waves (Der Kiureghian and Neuenhofer 1991).

The site response effect due to the differences in the local site conditions is obtained as

$$\theta_{lm}(\omega)^s = \tan^{-1}\frac{\text{Im}[H_1(\omega)H_m(-\omega)]}{\text{Re}[H_1(\omega)H_m(-\omega)]} \tag{23}$$

where $H_1(\omega)$ is the local soil frequency response function (Der Kiureghian and Neuenhofer 1991). In this study, as the foundation of the dams are considered as the homogeneous soil, the site response effect will not be taken into account in the analyses.

The power spectral density function of ground acceleration characterizing the earthquake process can be modeled as Filtered White Noise (FWN) model modified by Clough and Penzien (1993) as

$$S_{\ddot{u}_g}(\omega) = S_0\left(\frac{\{1 + 4\xi_g^2[\omega/\omega_g]^2\}}{\{1 - [\omega/\omega_g]^2\}^2 + 4\xi_g^2[\omega/\omega_g]^2}\right)\left(\frac{[\omega/\omega_f]^4}{\{1 - [\omega/\omega_f]^2\}^2 + 4\xi_f^2[\omega/\omega_f]^2}\right) \tag{24}$$

where ω_g, ξ_g are the resonant frequency and damping ratio of the first filter, ω_f, ξ_f are those of the second filter, and S_0 is the amplitude of the white-noise bedrock acceleration.

In this study, the medium soil type is used at the support points of the example dams and the filter parameters corresponding to this soil condition are obtained. For this purpose Erzincan earthquake acceleration in 1992, recorded at the medium soil condition is used. Acceleration power spectral

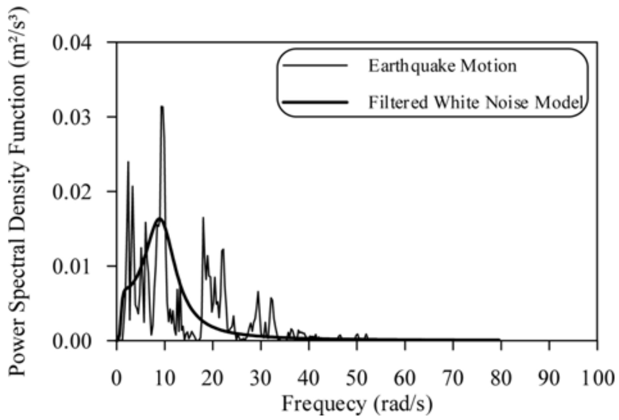

Fig. 2 Power spectral density function at medium soil

density function of this ground motion for the medium soil type is shown in Fig. 2. The spectral density function for the Filtered White Noise ground motion models is also given in this figure. The amplitude of the white-noise bedrock acceleration S_0 is obtained for the medium soil type by equating the variance of the ground acceleration (Eq. (17)) to the variance of the components of the Erzincan earthquake acceleration in 1992 record. The calculated intensity parameter value for medium soil type is, S_0(medium) = 0.00593 m^2/s^3. Filter parameter values (ω_g, ξ_g, ω_f, ξ_f) proposed by Der Kiureghian and Nevenhofer (1991) are utilized as $\omega_g = 10.0$, $\xi_g = 0.4$, $\omega_f = 1.0$ and $\xi_f = 0.6$.

5. Dam models

To investigate the stochastic analysis of nonlinear response of earth and rock-fill dams, two-dimensional mathematical model is used for calculations. It has been shown that a two-dimensional analysis of the fill dams provides natural frequencies and mode shapes which are in close agreement with those obtained by the three-dimensional analysis (Griffiths and Prevost 1988). Therefore, a two-dimensional analysis is carried out in the horizontal direction plane of the dams in order to achieve the stochastic response to spatially varying earthquake forces (Ramadan and Novak 1992). On the other hand, the ground motion may vary considerably along over the dam base for large dams. Thus, a three-dimensional analysis of the large dams may be called for but the computational requirements and costs of an analysis of such a system may be very high. The fact that this two-dimensional model has a relatively small number of degrees of freedom makes it more attractive by saving on computer time. Obviously, if actual design values for the responses are desired three-dimensional model should be taken into account.

5.1 Earth-fill dam model

In this study, Gordes Dam, which is a zoned earth-fill dam located in Manisa in Turkey, is selected as the first problem. The dam is 120 m high above its lowest foundation and 90 m above the original streambed. The crest has a width of 10 meters and a maximum length of 617.00 meters. The upstream and downstream slopes are 3:1 and 2.5:1, respectively. The dam is equipped with a

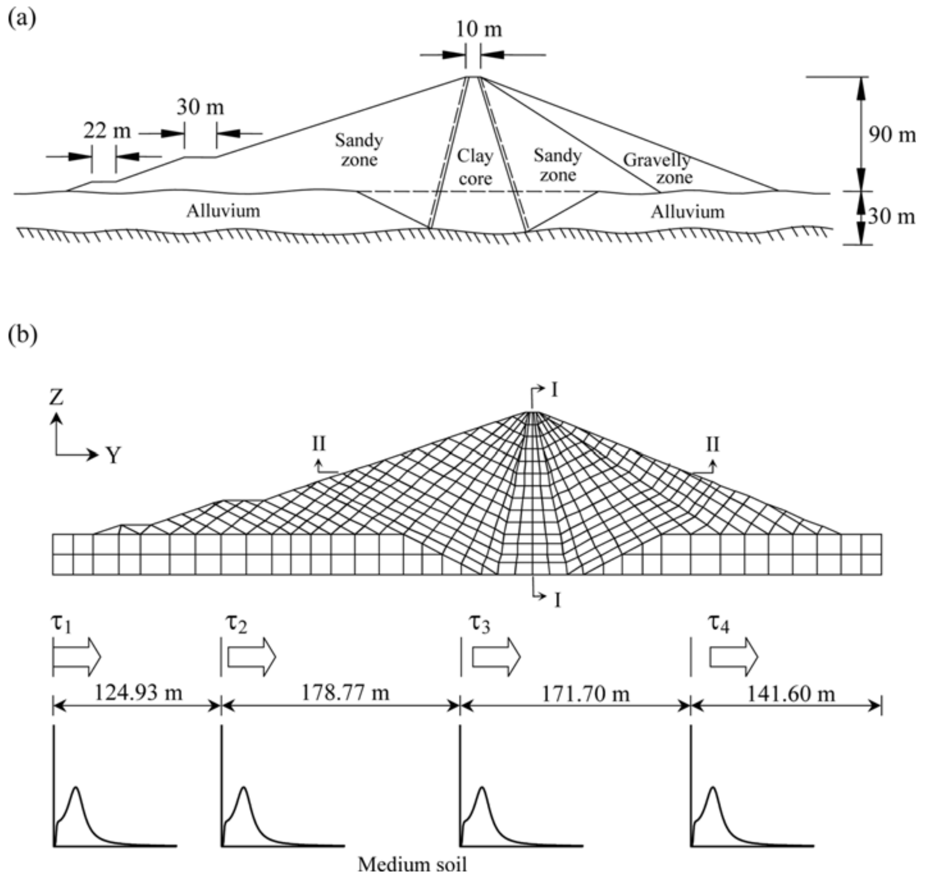

Fig. 3 (a) Dam cross section, (b) finite element model of earth-fill dam

clay core consisting of saturated clays that rise from bedrock with slopes of 0.2:1. Fig. 3(a) shows the cross section at the midlength of embankment dam. The finite element model subjected to different ground motions with 473 three and four-node isoparametric finite elements are shown in Fig. 3(b).

Two translational degrees of freedom are assigned to each node and a plane-strain assumption is used in the calculations. Gordes Dam is made of a clay core, sandy zone and sandy gravel zone, and the foundation material is made of alluvium (gravel and sand). Saturated unit weights of 20.3 kN/m^3, 21.0 kN/m^3, 21.5 kN/m^3 and 20.0 kN/m^3 are assumed for the clay core, foundation, sand and gravel-sand, respectively. In the analysis, Poisson ratios are selected as 0.45, 0.40, 0.35 and 0.43 for clay core, foundation, sand and gravel-sand, respectively. The cohesion constant is 15 kN/m^2 and the angle of friction is equal to 20^0 for the saturated clay core. $(K_2)_{\max}$ factor is given as 61, 122 and 140 at small-strains for the dynamic modulus coefficient of the sandy, gravel and alluvium (sand and gravel) materials, respectively. Maximum shear modulus for the central core is calculated depending on the G/s_u ratio. To evaluate the small-strain shear modulus of the core material, the average ratio $G_{\max}/s_u$ is taken as 2000. The initial damping value is selected as 5% for the stochastic analysis of nonlinear response of the earth-fill dam.

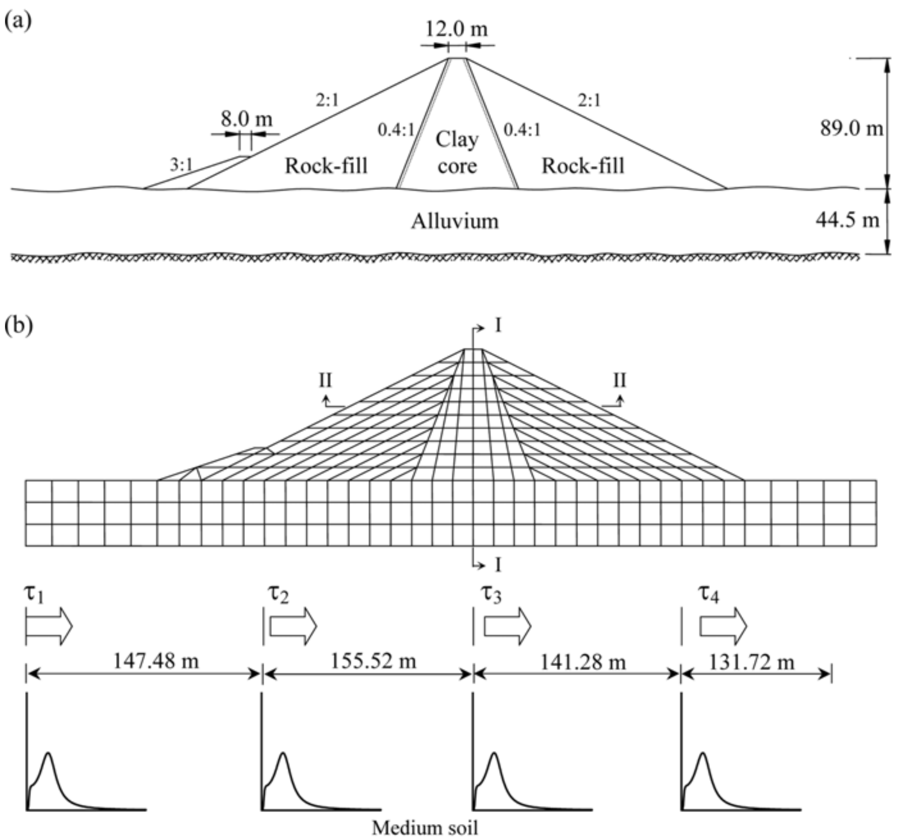

Fig. 4 (a) Dam cross section, (b) finite element model of rock-fill dam

5.2 Rock-fill dam model

Adatepe Dam is selected as the second problem, which has a clay core dam located in Kahramanmaras in Turkey. A typical dam cross section has a height of 89.0 meters above the base. The crest has a width of 12.0 meters and a maximum length of the dam itself of 398.0 meters. Upstream and downstream slopes are at 3:1. The cross section materials are grouped in two main categories: compacted rock-fill and impervious core which consists of saturated clays that arises from its lowest high with slopes of 0.4:1. The dam itself and foundation block are included together in the analyses. The height and length of the foundation block is 44.5 and 576.0 meters, respectively. The foundation block is made of alluvium material (sand and gravel). Fig. 4(a) shows the cross section at midlength of rock-fill dam. The weight of 21.0 kN/m^3, 21.0 kN/m^3 and 21.1 kN/m^3 are assumed for the rock-fill, foundation and clay core material, respectively. The Poisson's ratio is assumed to be equal to 0.35 at rock-fill material, equal to 0.45 for the clay core and equal to 0.40 for the alluvium material. In addition, the cohesion constant is 15 kN/m^2 and angle of friction is equal to 20^0 for the saturated clay core. $(K_2)_{\max}$ factor given in Eq. (1) is selected as 170 and 220 at small-strains for the dynamic modulus coefficient of the rock-fill and alluvium (sand and gravel) materials, respectively. Maximum shear modulus for the central core is calculated depending on the G/s_u ratio. To evaluate the small-strain shear modulus of the core material, the average ratio $G_{\max}/s_u$

is taken as 2000. The initial damping value is selected as 5% for the stochastic analysis of nonlinear response of the rock-fill dam.

The finite element model consists of 286 three and four-node isoparametric finite elements for the dam-foundation system as shown in Fig. 4(b).

6. Random vibration analyses

To investigate the importance of transient stochastic effects on the nonlinear behavior of the dams subjected to spatially varying ground motion the shear strains, displacements and stresses along the selected sections are computed at various times as well as the stationary excitation is applied to the dams initially. The mean of maximum values of the transient responses are calculated at times of 1, 5 s and compared with the stationary responses for the special cases of the ground motion model. These special cases can be categorized as: the wave-passage effect, the incoherence effect and the general excitation case including all the effects together.

For the incoherence effect, Harichandran and Vanmarcke's (1986) model is used. Soil condition is considered as medium soil throughout the study. The different ground motion models are applied to the dams in the horizontal direction as shown in Figs. 3-4(b). The apparent wave velocity is taken as v_{app} = 700 m/s for the medium soil case. The duration of the earthquake ground motions applied to the dams is taken as 20.94 seconds.

7. Lateral responses of the dams

For multi-DOF system, the rate at which the total transient response grows depends on the value of $\xi_i\omega_i$ for each mode, and on how much the lower modes contribute to the overall response. If the lower modes with small $\xi_i\omega_i$ do not contribute significantly. Since earth and rock-fill dams tend to be stiff and have high fundamental frequencies, it may be important to consider the transient response due to the structure initially being at rest when the duration of strong earthquake shaking is short.

Table 1 First 10 modal frequencies of the earth and rock-fill dams

Mode	Earth-fill dam frequencies (Hz)	Rock-fill dam frequencies (Hz)
1	4.71	5.36
2	6.24	9.06
3	7.27	9.80
4	7.32	11.15
5	7.55	12.28
6	7.77	13.58
7	8.15	13.89
8	8.52	14.31
9	8.71	14.89
10	9.36	15.01

The modal frequencies of the first ten lateral modes of the earth and rock-fill dams are given in Table 1. As can be seen in Table 1, the low modes of the dams have very high frequencies. Therefore, the total response for earth and rock-fill dams may reach stationarity rather quickly. These results seem more clearly in below analyses.

7.1 Earth-fill dam responses

In order to determine whether the earth-fill dam will reach its stationary response during typical durations of strong shaking; the total transient responses are calculated at times of 1, 5 seconds and compared with the stationary responses for the spatially varying ground motion (general excitation case including the wave passage and incoherence effects).

The mean of maximum stationary and transient shear strain and displacement values along the Section I-I are compared in Figs. 5-6, respectively. As the shear strain and displacement values near the base of the dam for the stationary and transient responses are close to each other, the stationary shear strain and displacement values near the dam crest are larger than the transient shear strain and displacement values for the 1st second. At the point where the maximum shear strain values take place 66, 100% of the stationary response is achieved at times of 1, 5 s of the transient response, respectively. Similarly, at the point where the maximum displacement values take place 96, 99% of the stationary response is achieved at times of 1, 5 s of the transient response, respectively. The mean of maximum total normal stresses in the horizontal and vertical direction and shear stress along the Section II-II are shown in Figs. 7-9. As the normal stresses and shear stress values obtained from the stationary analysis are larger than those of the transient analysis for the 1st second, the stress values for the stationary and transient analysis are close to each other at times of 5th second. At the point where the maximum values, the normal stress in the horizontal direction take place 93, 100%; in the vertical direction take place 91, 99% and shear stress take place 85, 100% of the stationary response is achieved at times of 1, 5 s of the transient response, respectively.

The variance of the total response comprises of three components: the variance of the pseudo-static response, the dynamic response, and the covariance between the pseudo-static and dynamic responses. Along the Section I-I of the dam, the normalized variance distributions of the pseudo-

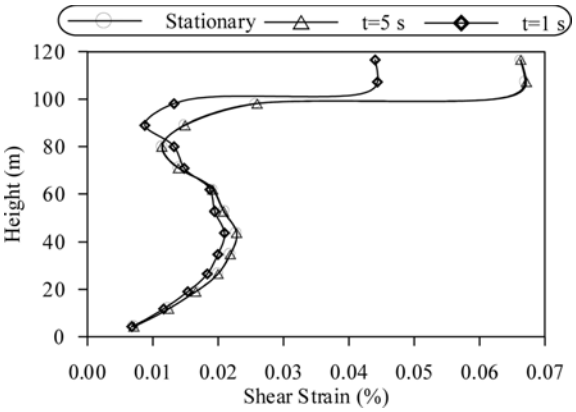

Fig. 5 Shear strain values along the Section I-I of earth-fill dam

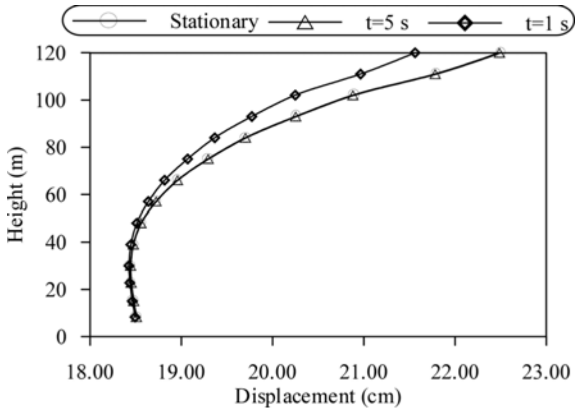

Fig. 6 Mean of maximum transient total horizontal displacements along the Section I-I of earth-fill dam

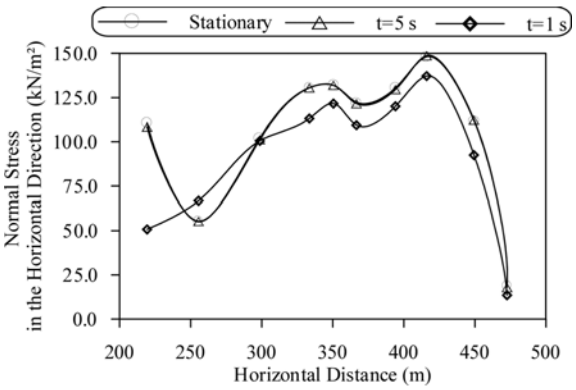

Fig. 7 Mean of maximum transient total normal stress in the horizontal direction along the Section II-II of earth-fill dam

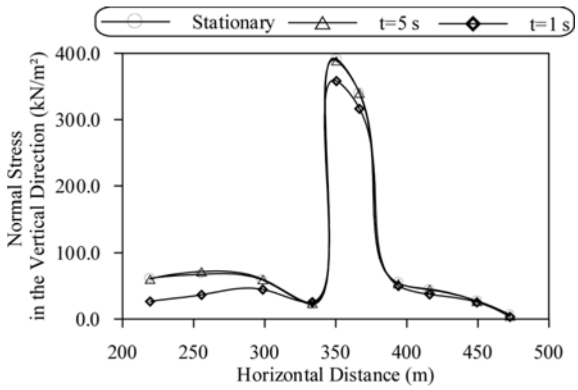

Fig. 8 Mean of maximum transient total normal stress in the vertical direction along the Section II-II of earth-fill dam

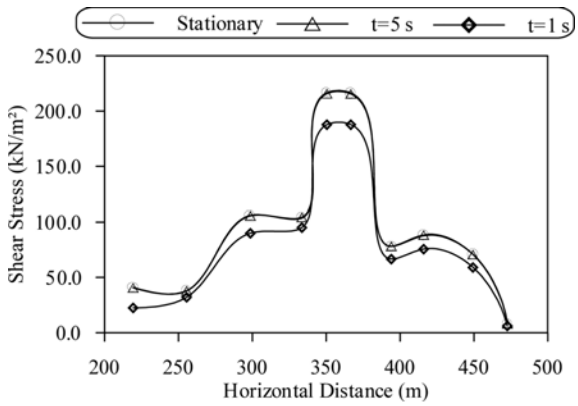

Fig. 9 Mean of maximum transient total shear stress along the Section II-II of earth-fill dam

static, dynamic, covariance and total normal stresses in the horizontal direction, which are obtained for the transient response at 1st second, of the selected earth-fill dam subjected to spatially varying ground motion model (general excitation case) are shown in Fig. 10. As can be seen in Fig. 10, while the normal stresses in the horizontal direction are dominated by the pseudo-static component, the contribution of the dynamic and covariance components to the total response are smaller than those of the pseudo-static component. The covariance component has also smallest contribution to the total response for the general excitation case. Examining the components of the total normal stress response in the horizontal direction at point where the maximum values take place reveals that, the pseudo-static component contributes 99.81%, the dynamic component contributes 0.29% and the covariance component contributes −0.10% to the total response.

Mean of maximum total responses obtained for the transient analysis at 1st second along the Section I-I of the dam are depicted in Fig. 11 for the cases of the general excitation, wave-passage effect, incoherence effect and uniform ground motion. From Fig. 11, as the total normal stress responses in the horizontal direction due to the general excitation case and incoherence effect are generally the largest, the total response due to the uniform ground motion is generally the lowest. In

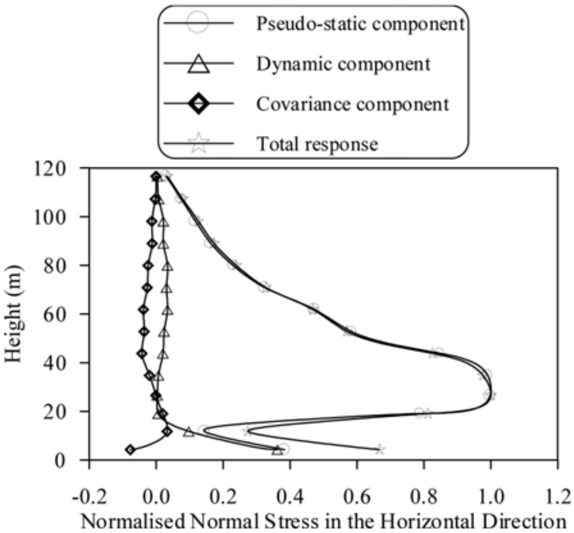

Fig. 10 Normalised variances of normal stress in the horizontal direction along the Section I-I of earth-fill dam

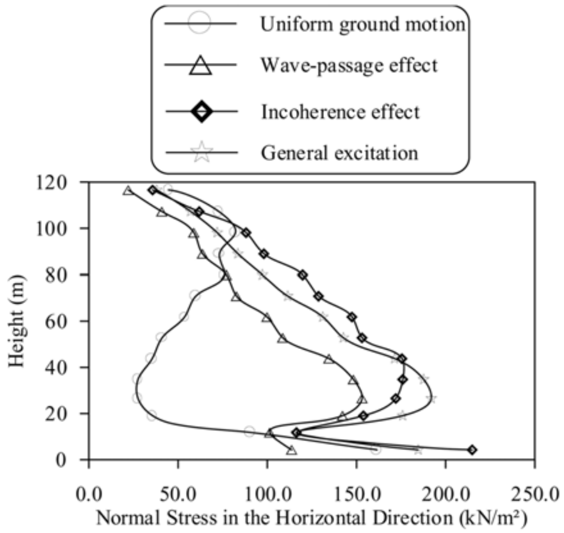

Fig. 11 Mean of maximum total normal stress in the horizontal direction along the Section I-I of earth-fill dam

addition, in general, the normal stress values due to the wave-passage effect are larger than those of the uniform ground motion case. The total stress response at the dam height of 26.5 meters for the general excitation case is larger than as much as 11.54%, 25.38% and 598.32% when compared to the response due to the incoherence, wave-passage effects and uniform ground motion cases, respectively.

7.2 Rock-fill dam responses

The comparison of the stationary and transient responses of the rock-fill dam is also investigated

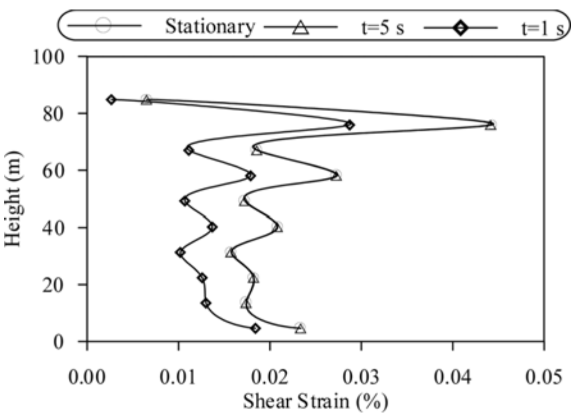

Fig. 12 Shear strain values along the Section I-I of rock-fill dam

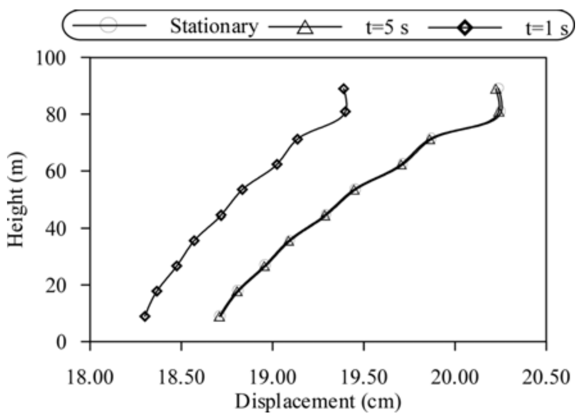

Fig. 13 Mean of maximum transient total horizontal displacements along the Section I-I of rock-fill dam

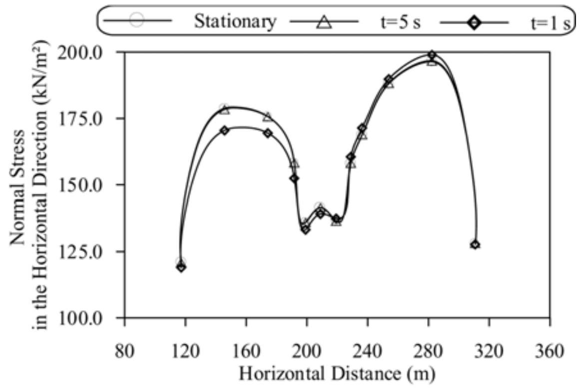

Fig. 14 Mean of maximum transient total normal stress in the horizontal direction along the Section II-II of rock-fill dam

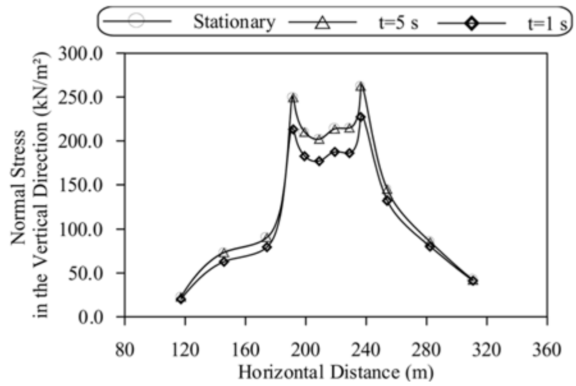

Fig. 15 Mean of maximum transient total normal stress in the vertical direction along the Section II-II of rock-fill dam

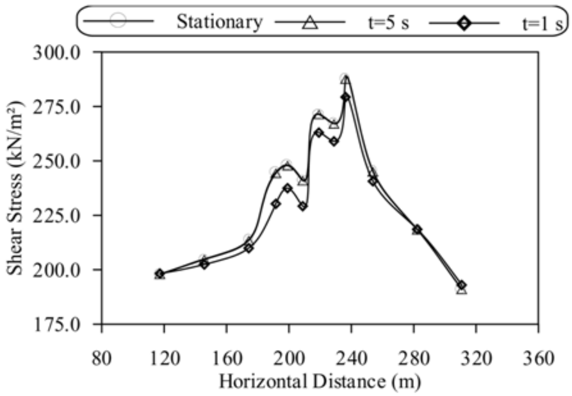

Fig. 16 Mean of maximum transient total shear stress along the Section II-II of rock-fill dam

for the general excitation case. Fig. 12 shows the shear strain computed along the Section I-I. At the point where the maximum values take place, 65, 100% of the stationary response is achieved at times of 1, 5 s of the transient response, respectively. Fig. 13 illustrates the horizontal displacements along the Section I-I. At node where the maximum values take place, 96, 100% of the stationary response is achieved at times of 1, 5 s of the transient response, respectively. Finally, Figs. 14-16 illustrate the normal stress in the horizontal and vertical direction and shear stress along the Section II-II. The stresses obtained from stationary analysis are generally close to the transient analysis. At the dam horizontal distance of 208.85 meters where corresponds to clay core, for the normal stress in the horizontal direction 98, 100%, for the normal stress in the vertical direction 88, 100% and for the shear stress 95, 100% of the stationary response is achieved at times of 1, 5 s of the transient response, respectively.

Fig. 17 presents the distributions of the pseudo-static, dynamic, covariance and total normal stress in the horizontal direction along the Section I-I of the dam. These variations due to the general excitation case are obtained for the transient analysis at 1st second. As the total normal stresses in the horizontal direction are dominated by the pseudo-static component, the dynamic and covariance components have small contributions for the general excitation case. Through these results, it is

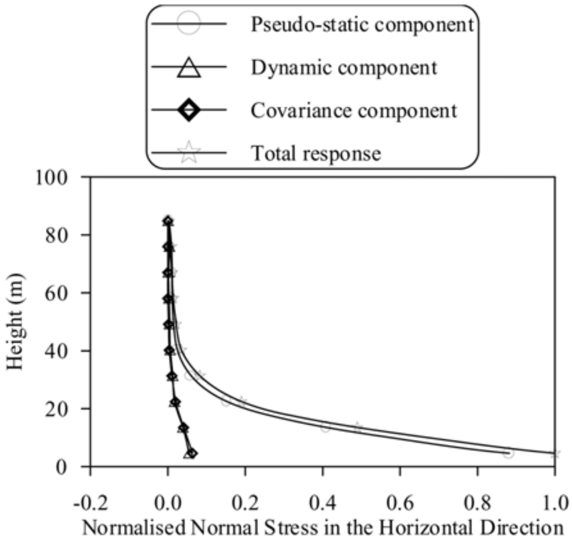

Fig. 17 Normalised variances of normal stress in the horizontal direction along the Section I-I of rock-fill dam

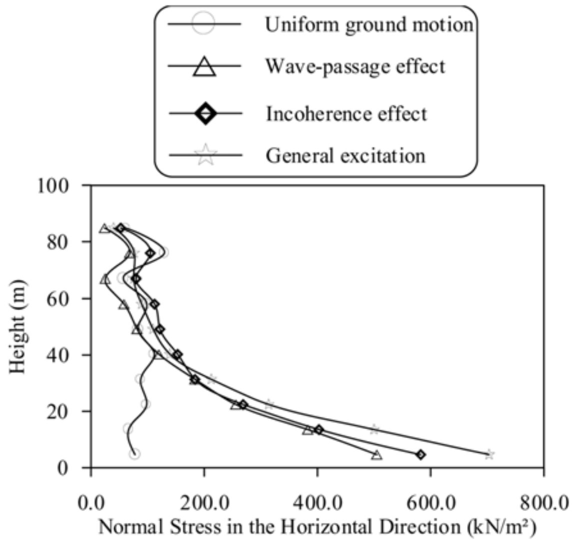

Fig. 18 Mean of maximum total normal stress in the horizontal direction along the Section I-I of rock-fill dam

clearly seen that the contribution of the pseudo-static component to the total response increases significantly with increasing depth of the dam. At the point where the maximum values take place, the pseudo-static component contributes 88.21%, the dynamic component contributes 5.43% and the covariance component contributes 6.36% to the total response.

Fig. 18 illustrates the mean of maximum normal stress in the horizontal direction for the transient analysis at 1st second, due to the general excitation case, wave-passage effect, incoherence effect and uniform ground motion. As at the point near the crest of the dam stress variations are close to each other for the ground motion models at the point near the base of the dam stress variations are showed more clearly. So, at the point near the base of the dam, while the stress values due to the general excitation case are the largest, the values due to the uniform ground motion are the smallest. In addition, the stress values for the incoherence effect are larger than those for the wave-passage effect. The total stress response at the point where the maximum values take place for the general excitation case is larger than as much as 39.37%, 20.76% and 809.09% when compared to the response due to the incoherence, wave-passage effects and uniform ground motion cases, respectively.

Like earth-fill dam, the rock-fill dam stationary response levels are reached within about 5 s. From the figures, while at $t = 1$ s the transient responses are smaller than the corresponding stationary ones, at $t = 5$ s, the transient responses are very close to the stationary ones. Thus, transient effects may be neglected for earth and rock-fill dams under most earthquakes.

8. Conclusions

The transient stochastic analysis of nonlinear response of earth and rock-fill dams subjected to spatially varying ground motion is presented in this paper and compared with the stationary analysis.

When comparing the transient responses obtained at the various durations of the earthquake ground motion with those of the stationary values, it is observed that the stationary assumption is reasonable for earth and rock-fill dams.

As the total transient stress responses at 1st second are dominated by the pseudo-static component, the dynamic and covariance components have insignificant contributions for the spatially varying ground motion (general excitation case). It can be also observed from the figures that, whereas the stress responses are generally the smallest for the uniform ground motion, the responses obtained from the spatially varying ground motion (general excitation case) are generally largest. The incoherence effect has generally more significant influence on the response of earth and rock-fill dams, comparing with the wave-passage effect.

Transient responses, which typically have very small contributions from the low modes, do not overshoot the stationary responses. So, it can be said that transient effects may be neglected for typical earth and rock-fill dams subjected to spatially varying ground motion.

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