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RESEARCH ARTICLE

Influence of uncertainty of structural parameters on the stochastic dynamic response of asphaltic lining dam-foundation systems to non-stationary random seismic excitation

Kemal Hacıefendioğlu*, Hasan Basri Başağa

Karadeniz Technical University, Department of Civil Engineering, 61080, Trabzon, Türkiye

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Abstract

The present work aims toward the effect of uncertain structural parameters on the stochastic dynamic response of an asphaltic lining dam-foundation system subjected to stationary as well as non-stationary random excitation. Uncertain structural parameters of interest are shear modulus and mass density, modeled using the lognormal distribution. The stochastic seismic response of the dam-foundation system to the random loads with uncertain structural parameters is carried out with the Monte Carlo simulation method. The spatial variability of ground motion is considered with incoherence and wave passage effects as stationary as well as non-stationary random excitation. A time-dependent frequency response function is used throughout the study for non-stationary responses. Obtained results indicate that the variability of shear modulus and mass density can be neglected in stochastic dynamic analysis of an asphaltic lining dam-foundation system. Also, it is seen from the results that stationarity is a reasonable assumption for asphaltic lining dams to typical durations of strong shaking.

1. Introduction

The recent development of stochastic finite element methods verifies that the uncertainties with the input motions and the dynamical properties of the materials in geotechnical earthquake engineering significantly affect the dynamic behaviors of soil structures. Firstly, the uncertainties of the material properties are mainly a consequence of the fact that the soil properties are naturally formed in many different depositional environments; therefore, their physical properties will vary from point to point. Secondly, the uncertainty of ground motion is related mainly to earthquake input which is caused by earthquake input source mechanism, transmission path, and others. There have been significant prior studies [1-9] related to stochastic finite element analysis of the geotechnical area including the parametric uncertainties as well as the random loads.

The dam–foundation interaction systems subjected to earthquake ground motion are geotechnical problems. The standard seismic analysis of these systems assumes that the same ground motion acts simultaneously at all points along the base of the dam. However, for large structures, such as pipelines, long-span bridges, dams, etc. it is obvious that ground motions will be subjected to significant variations because

* Corresponding author (kemalhaciefendioglu@ktu.edu.tr)

of the finite velocity of wave propagation, loss of coherency of seismic waves due to reflections and refractions in the medium of the ground and differences in local soil conditions. It is well known that earth and rock-fill dams are subjected to different ground motions at their foundations. The effect of spatially varying ground motion on the response of the fill dams has been analyzed in the past few years [10-17].

This paper aims to investigate the effect of uncertainties of structural parameters on the dynamic response of dam-foundation systems to stationary as well as non-stationary random excitation. For this purpose, the spatially varying ground motion including the wave-passage and incoherence effects together is taken into account in this study. The stationary process is represented by using the spectral density function. It is reasonable to represent the non-stationary process with a time-dependent modal frequency function [18, 19]. The uncertainties of the shear modulus and unit weight (or mass density) of the dam-foundation system are taken into account in the study. But, the structural parameters are completely assumed as uncorrelated. Due to the nature of the material, it is proposed to model the low-strain shear moduli as a lognormal random field, so the material nonlinearity is omitted in this study. In this frame, the Monte Carlo Simulation method using the stochastic finite element method is used to estimate the effect of the uncertainties of the structural parameters on the dynamic response analysis of an asphaltic lining dam-foundation system subjected to random loads taking into account the spatially varying ground motion. The main drawback of the simulation method used herein is its enormous computation requirement, but it is simple, direct, and quite powerful. Two-dimensional interface finite elements between the dam soil deposit and asphaltic lining layer as well as the foundation soil deposit and cut-off wall and Monte Carlo simulation method are coded in the computer program SVEM [20] by the authors.

2. Uncertainty of structural parameters

Asphaltic lining dam-foundation systems are non-homogeneous structures constructed from soil and asphalt materials. Since the structural parameters of such types of these dam structures are estimated from a limited set of data and vary from place to place resulting deposits have an uncertainty characteristic. Uncertain material properties are best defined as random variables described by a mean, a standard deviation, and a probability distribution function. To investigate the influence of randomness in structural parameters on the seismic response of the dam-foundation system to random loads, the randomness in the soil shear modulus and mass density are considered in the study. While the randomness of shear modulus affects the dynamic stiffness matrix, those of mass density influence the dynamic mass matrix of the structure finite element formulation. The shear modulus and mass density are modeled using the lognormal distribution. This choice is motivated by the fact that these parameters are positive, and lognormal distribution enables analyzing its large variability [9]. The random field for shear modulus and mass density are generated using simulations by the Monte Carlo method (MCS). Thus, a standard stochastic finite element analysis using spatially varying ground motion is carried out per MCS generating the random variables.

The Monte Carlo simulation technique (MCS) is the most effective and widely applicable method for handling large-scale probabilistic or stochastic finite element problems with complicated structural responses, despite involving expensive computations due to the successive finite element analyses required. The MCS method generates the random number z_i, by using the pseudo-static method based on recurrent procedures [21]. The stochastic variables u_i are obtained through the stochastic variable functions for lognormal distribution considering them as independent stochastic variables. The generation of a stochastic variable can be expressed as

$$u_i = m_x + s_x\sqrt{(-2)\ln z_i}\,cos(2\pi z_i + 1),$$
$$u_{i+1} = m_x + s_x\sqrt{(-2)\ln z_i}\,sin(2\pi z_i + 1) \quad (1)$$

The shear modulus expression is given by using a lognormal distribution

$$G(z) = e^{[G_{0,ln\,G}+(\sigma_{ln\,G})*u_i]} \tag{2}$$

with

$$\sigma_{ln\,G}^2 = ln\left(1+\frac{\sigma_G^2}{G_{0,G}^2}\right),$$

$$G_{0,ln\,G} = ln(G_{0,G}) - \frac{1}{2}\sigma_{ln\,G}^2 \tag{3}$$

where G_0 and σ_G^2 stand for the shear modulus mean and variance, respectively. When substituting ρ instead of *G* into Eqs. (2) and (3), mass density expression can be formulated in the same equations for the lognormal distribution.

After a set of *N* individual response functions, *X* are observed, the mean and standard deviation of *X* (mean of absolute maximum values) can be calculated by

$$E(X) = \frac{1}{N}\sum_{i=1}^{N} X_i \tag{4}$$

$$\sigma_X^2 = \frac{1}{N(N-1)}\left(N\sum_{i=1}^{N} X_i - \left(\sum_{i=1}^{N} X_i\right)^2\right) \tag{5}$$

The variation in the response becomes insignificant for a sample size larger than 6000, and therefore, in this study, a sample size of 6000 is chosen for the subsequent simulations.

3. Random vibration formulation

3.1. Stationary random vibration

Since the formulation of the random vibration theory for spatially varying ground motion is given previously by many researchers [17, 22, 23, 24], in this study only the required final equations will be considered. The random vibration theory provides an approximate estimate of the mean of the absolute maximum response of the structure in terms of the power spectral density function and a coherency function. The free-response can be decomposed into pseudo-static and dynamic parts, i.e. $z = z_s + z_d$when there is a differential excitation at the supports. Assuming the stationary excitation, the total variance responses can be obtained from:

$$\sigma_z^2 = \sigma_{z_d}^2 + \sigma_{z_s}^2 + 2Cov(z_s, z_d) \tag{6}$$

where $\sigma_{z_d}^2$ and $\sigma_{z_s}^2$ are the dynamic and pseudo-static variances, respectively, and $Cov(z_s, z_d)$ is the covariance between the dynamic and pseudo-static responses z_d and z_s [12]. The three components above Eq. (6) are given by

$$\sigma_{z_d}^2 = \int_{-\infty}^{\infty} \sum_{j=1}^{n}\sum_{k=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_{ij}\psi_{ik}\Gamma_{lj}\Gamma_{mk}H_j(-\omega)H_k(\omega)S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)\,d\omega \tag{7}$$

$$\sigma_{z_s}^2(\omega) = \sum_{l=1}^{r}\sum_{m=1}^{r} A_{il}A_{im}\int_{-\infty}^{\infty}\frac{1}{\omega^4}S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)d\omega \tag{8}$$

$$Cov(z_s, z_d) = -\sum_{j=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_{ij}A_{il}\Gamma_{mj}\int_{-\infty}^{\infty}\frac{1}{\omega^2}H_j(\omega)S_{\ddot{u}_{g_l}\ddot{u}_{g_m}}(\omega)d\omega \tag{9}$$

where ψ is 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. Ail and Aim are equal to the static displacements for unit displacements assigned to each support point [22]. The frequency response function is defined as

$$H_k(\omega) = \frac{1}{\omega_k^2 - \omega^2 + 2i\xi_k\omega_k\omega} \tag{10}$$

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

3.2. Non-stationary random vibration

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). In random vibration analysis, statistical averages are assumed to be independent of time for stationary excitation. Earthquake motions cannot be stationary, because they initially grow from zero, then have a steady phase and eventually decay. Non-stationary response due to stationary excitation beginning at time $t = 0$ can be easily accommodated into the framework developed for a stationary response, and in many cases, such a model is sufficient to assess the impact of non-stationary.

In most earthquake engineering applications, it is reasonable to represent the non-stationary ground acceleration by an envelope-modulated stationary random process given by

$$\ddot{u}_g(t) = a(t)w(t) \tag{11}$$

where $a(t)$ = a temporal modulating function; and $w(t)$ = stationary random process. The non-stationary mean-square responses at time t can be computed using frequency domain analysis by substituting the time-dependent modal frequency response function$H_k(\omega, t)$, defined by [18, 19]

$$H_k(\omega, t) = H_k(\omega)\left[1 - e^{-\xi_k\omega_k t}e^{-i\omega t}\left(\cos\omega_{kd}\, t + \frac{(\xi_k\omega_k + i\omega)}{\omega_{kd}}\sin\omega_{kd}\, t\right)\right] \tag{12}$$

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. Expected 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 can be expressed as

$$\mu = p\sigma_z \tag{13}$$

where p is a peak factor, which is a function of the time of the motion and the mean zero crossing rate [25].

$$p = \sqrt{2\ln(\nu_e T)} + \frac{0.5772}{\sqrt{2\ln(\nu_e T)}} \tag{14}$$

where

$$\nu_e = (1.9\xi^{0.15} - 0.73)\nu_0 \tag{15}$$

is the equivalent mean zero-crossing rate. In which, T is the duration of the motion, ξ is the modal damping ratio, and ν_0 is the frequency of occurrence.

The frequency of occurrence is described as the average number of times that the line ($y(t) = 0$) is crossed by the response in a unit of time. For the Gaussian process of zero average, the average number of times in the zero level crossed by the process in a unit of time is expressed as:

$$\nu_0 = \frac{1}{\pi}\sqrt{\frac{\lambda_2}{\lambda_0}} \tag{16}$$

Because the zero level is crossed two times for each cycle, the frequency of occurrence for the response process will be equal to $\nu_0/2$ as

$$f_0 = \frac{\nu_0}{2} = \frac{1}{2\pi}\sqrt{\frac{\lambda_2}{\lambda_0}} \tag{17}$$

where λ_0 and λ_2 are the zeroth and second spectral moments, respectively.

For any stochastic analysis, it is valuable to be able to calculate the probability of occurrence of a particular value of the selected structural response quantity, and this has been achieved by Vanmarkce [26] through a cumulative probability distribution function for the first crossing time of a symmetric barrier for a zero-mean stationary Gaussian process.

4. Spatially varying ground motion

The spatial variability of the ground motion is characterized by the coherency function$\gamma_{lm}(\omega)$. The cross-power spectral density function between the accelerations $\ddot{u}_{g_l}$ and $\ddot{u}_g$ at the support points l and m for homogeneous ground motion is written as

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

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 power spectral density function of ground acceleration is assumed to be in the form of the filtered white noise ground motion model originally proposed by Kanai-Tajimi [27, 28] and modified by Clough and Penzien [29] as

$$S_{\ddot{u}_g}(\omega) = S_0\,\frac{\omega_g^4+4\xi_g^2\omega_g^2\omega^2}{(\omega_g^2-\omega^2)^2+4\xi_g^2\omega_g^2\omega^2}\,\frac{\omega^4}{(\omega_f^2-\omega^2)^2+4\xi_f^2\omega_f^2\omega^2} \tag{19}$$

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 spectrum of the white-noise bedrock acceleration.

The E-W component of the Erzincan Earthquake recorded on March 13, 1992, in Erzincan, Turkey is chosen as ground motion and given in Fig.1 since it occurred nearby the dam site. The acceleration power spectral density function of this ground motion for the medium soil type is shown in Fig.1. The spectral density function for the Filtered White Noise ground motion models is also given in this figure. The calculated intensity parameter value for medium soil type is $S_0 = 0.00593$ m^2/s^3. Filter parameters $(\omega_g, \xi_g, \omega_f, \xi_f)$ proposed by Der Kiureghian and Nevenhofer [30] are utilized as $\omega_g = 10.0$ rad/s, $\xi_g = 0.4$, $\omega_f = 1.0$ rad/s, and $\xi_f = 0.6$.

The complex coherency function accounting for the incoherence, wave passage effects is defined as

$$\gamma_{lm}(\omega) = \rho(\xi,\omega)e^{\left(\frac{iwd_{lm}^L}{v_{app}}\right)} \tag{20}$$

in which, $\rho(\xi,\omega)$ characterizes the incoherence effect and the exponential term represents the wave passage effect, v_{app} is the apparent wave velocity and d_{lm}^L is the projection of d_{lm} for the ground surface along the direction of propagation of seismic waves.

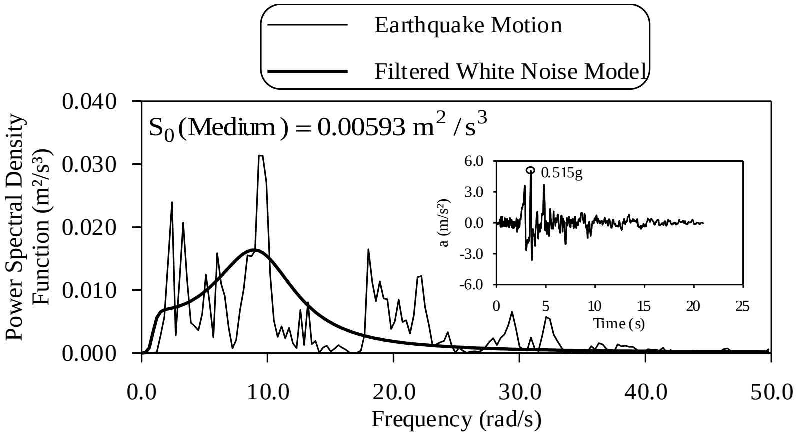


Fig. 1. Power spectral density function at the medium soil

Fig. 2. Muratlı Dam constructed in Artvin, Turkey

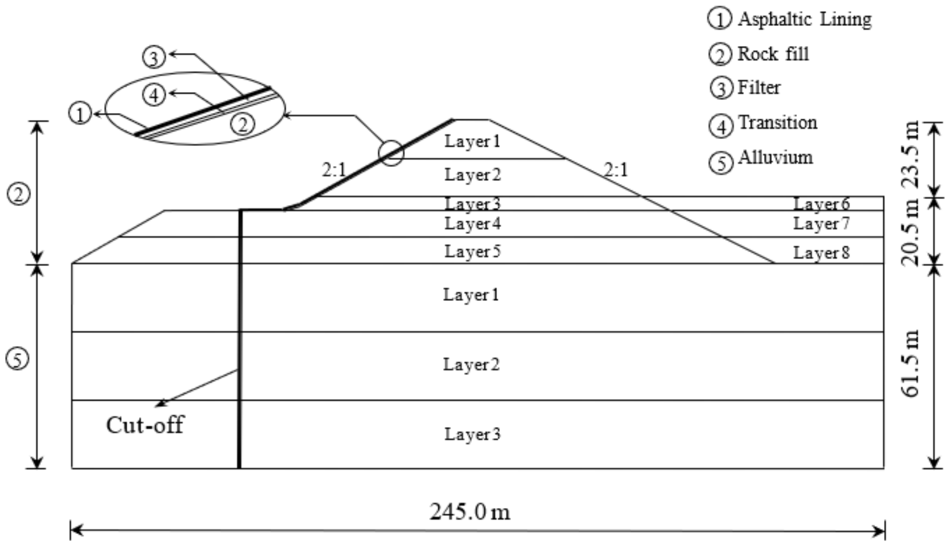
5. Application

Muratlı Dam (Fig. 2) located in Artvin in Turkey is selected as the asphaltic lining dam-foundation interaction problem. A typical dam cross section has a height of 44.0 meters above the base. The crest has a width of 10.0 meters and a maximum length of the dam itself of 213.0 meters. Upstream and downstream slopes are at 2:1. The dam itself and the foundation block are included together in the analyses. The height and length of the foundation block are 61.5 and 245.0 meters, respectively. The foundation block is made of alluvium material (sand and gravel). Fig. 3(a) shows the cross-section at midlength of the dam-foundation interaction system. The dam soil and foundation soil are considered to be layered systems that are homogeneous in a horizontal plane and have different soil properties. The properties of layers of the dam-foundation system and interface elements are given in Table 1. Also, the coefficient of variation of shear modulus and mass density are shown in Table 1. The initial damping value is selected as 5% for the stochastic analysis of the asphaltic lining dam-foundation system. The finite element model consists of 161 three and four-node isoparametric finite elements and interface finite elements for the dam-foundation system as shown in Fig. 3(b). The nodes representing the extreme left and right sides of the foundation block were allowed a horizontal degree of freedom. In this study, structural shear modulus and mass density are assumed to be random variables. This study neglects the influence of the correlation for the material properties of the dam-foundation system.

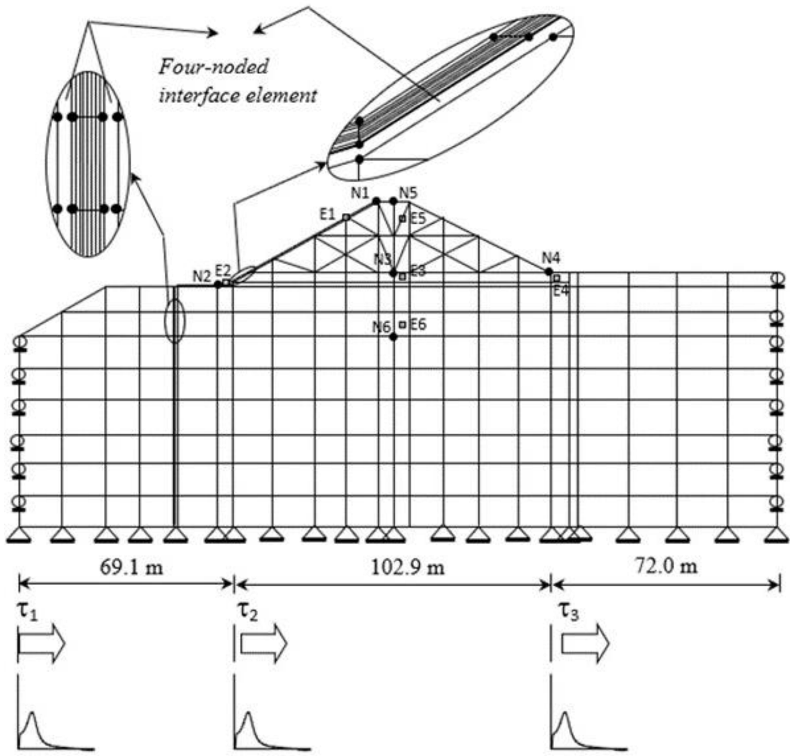
Figure 3(b) shows the model for spatially varying ground motion, which takes into account the dam-soil interaction. The dam-soil part is divided into three zones 69.1 m, 102.9 m, and 72.0 m. Eq. (18), which includes correlation and wave effect for these three regions, has been taken into account.

Table 1. Material properties for the cross-section of the Muratlı Dam

Material	Layer	Shear modulus (kN/m^2)		Mass density (kN.sn^2/m^4)		Poisson's ratio
		Mean	COV	Mean	COV	
Rockfill	1	3.04E+05	10%	2.19	10%	0.35
	2	4.50E+05				
	3	4.91E+05				
	4	5.00E+05				
	5	6.18E+05				
	6	2.06E+05				
	7	4.07E+05				
	8	5.66E+05				
Foundation (Alluvium)	1	9.89E+05	10%	2.14	10%	0.40
	2	1.28E+06				
	3	1.51E+06				
Asphalt		1.09E+04	10%	2.40	10%	0.35
Interface Element (Asphalt)		1.53E+05		2.40		0.30
Cut-off (Concrete)		1.08E+07	10%	2.45	10%	0.15
Interface Element (Cut-off)		8.33E+06		2.45		0.20



(a)



(b)

Fig. 3. (a) Dimensions and materials, and (b) finite element model of the dam-foundation system

6. Numerical results

6.1. Stationary responses

In this part of the study, the stationary stochastic responses of an asphaltic lining dam-foundation system subjected to spatially varying ground motion (SVGM) are calculated for uncertainties of structural parameters. To investigate the effect of the randomness in structural parameters on the stochastic behavior of the dam-foundation system, four different cases of the structural parameters are considered in the study. These special cases can be categorized as the average values of parameters (Case A), randomness in shear modulus (Case B), randomness in mass density (Case C), separately, and overall (Case D)(randomness in shear modulus and mass density together). Spatially varying ground motion includes the wave-passage effect and the incoherence effects. For the incoherence effect, Harichandran and Vanmarcke's model [31] is used. Soil conditions are considered homogeneous throughout the study. The SVGM is applied to the dams in the horizontal direction as shown in Fig. 3(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.

To compare the results due to the randomness in structural parameters, the means of horizontal displacement values on selected nodal points are shown in Figs.4 for different special cases of structural parameters. As can be seen from Fig.4, the displacement value obtained from the average values of parameters (Case A), randomness in shear modulus (Case B), and randomness in mass density (Case C), separately, and overall are very close to each other for selected nodal points. While the difference for horizontal displacement values at Node 1 (N1) between the average values of parameters (Case A) and randomness in shear modulus (Case B) is 0.09%, those of the average values of parameters (Case A) and randomness in mass density (Case C) is 0.01%, and also the displacement values for the average values of parameters (Case A) and overall (Case D) have 0.09% difference.

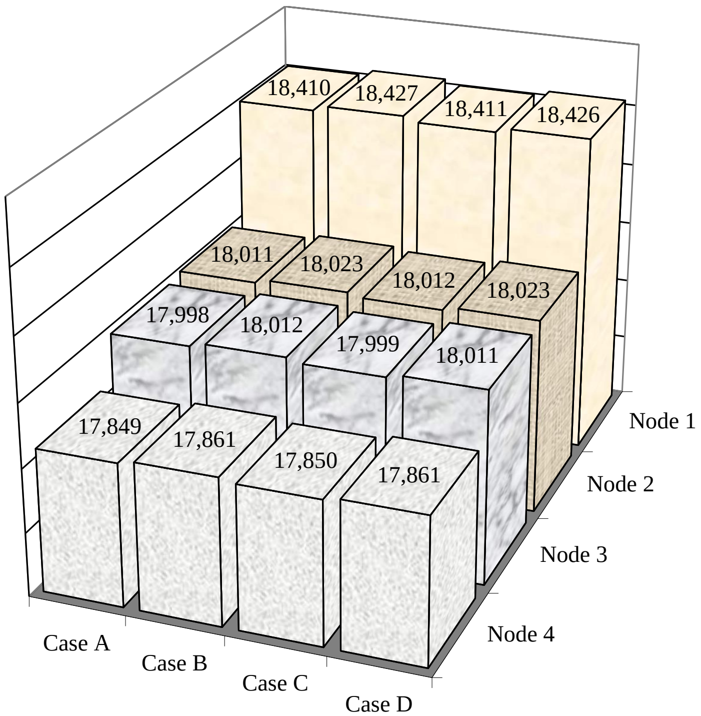


Fig. 4. Comparison of mean horizontal displacements (cm) at nodes 1-4 for Case A, B, C, and D

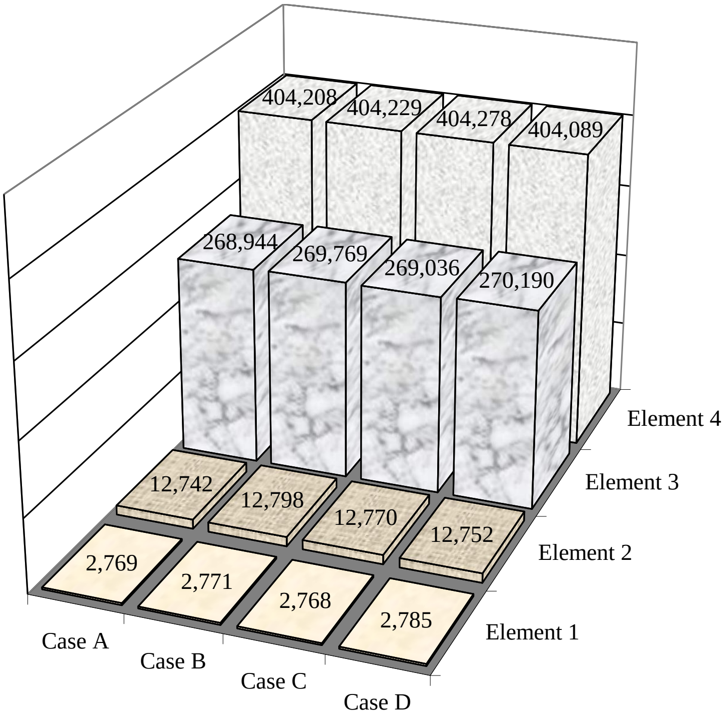


Fig. 5. Comparison of mean horizontal stresses (kN/m^2) at the center of elements 1-4 for Case A, B, C, and D

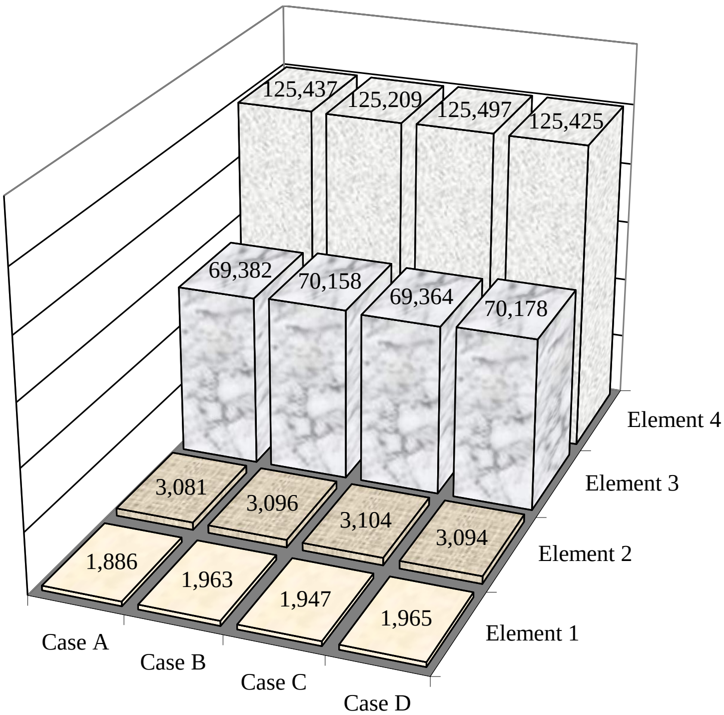


Fig. 6. Comparison of mean vertical stresses (kN/m^2) at the center of elements 1-4 for Case A, B, C, and D

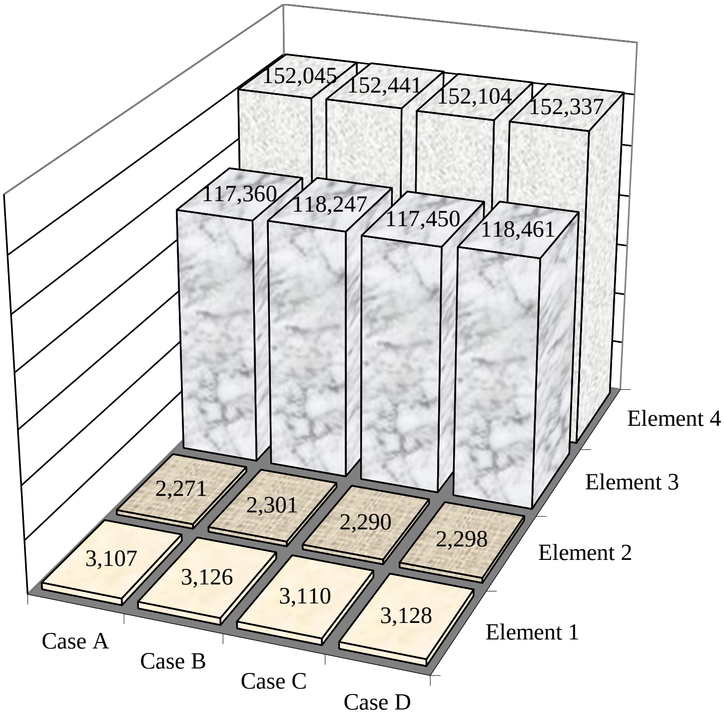


Fig. 7. Comparison of mean shear stresses (kN/m^2) at the center of elements 1-4 for Case A, B, C, and D

The means of stress values are calculated at the center of points of the elements for four different cases of the structural parameters. The finite elements (E1, E2, E3, E4) on the dam-foundation system are selected for the comparison of the mean stress values. The horizontal, vertical, and shear stress obtained on selected finite elements for the different cases are shown in Figs. 5-7, respectively. It is observed from the figures that the values obtained from the average values of parameters (Case A), randomness in shear modulus (Case B), randomness in mass density (Case C), separately, and overall (Case D) are very close to each other for horizontal, vertical and shear stresses, separately. At Element 1 (E1) the stress values obtained from the randomness in shear modulus (Case B), randomness in mass density (Case C), separately, and overall (Case D)) are 0.10%, -0.02% and 0.58% for horizontal stress value, 4.05%, 3.22% and 4.18% for vertical stress value, 0.60%, 0.11% and 0.68% for shear stress value larger than those from the average values of parameters (Case A), respectively. So, it can be said from the figures that the displacements and stresses due to Case A, B, C, and D have the smallest difference values. Consequently, it can be said from the figures the mean values of the stochastic response of the selected asphaltic lining dam-foundation system are not significantly affected by the randomness in structural parameters.

6.2. Non-stationary responses

For a multi-DOF system, the rate at which the total non-stationary 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. The first 5 natural frequencies of 1.99, 2.98, 3.65, 4.05, and 4.29 of the dam-foundation system are calculated by using the finite element method. Since asphaltic lining dams tend to be stiff and have high fundamental frequencies, it may be not important to consider the transient response due to the structure initially being at rest when the duration of strong earthquake shaking is short.

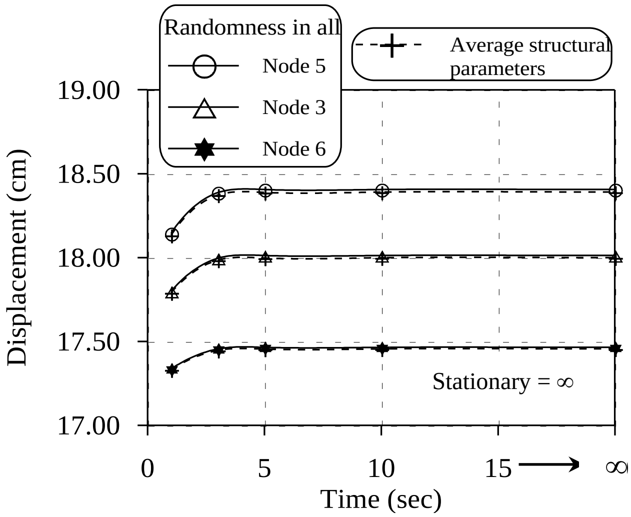


Fig. 8. Means of non-stationary horizontal displacement at nodes N5, N3, and N6, respectively

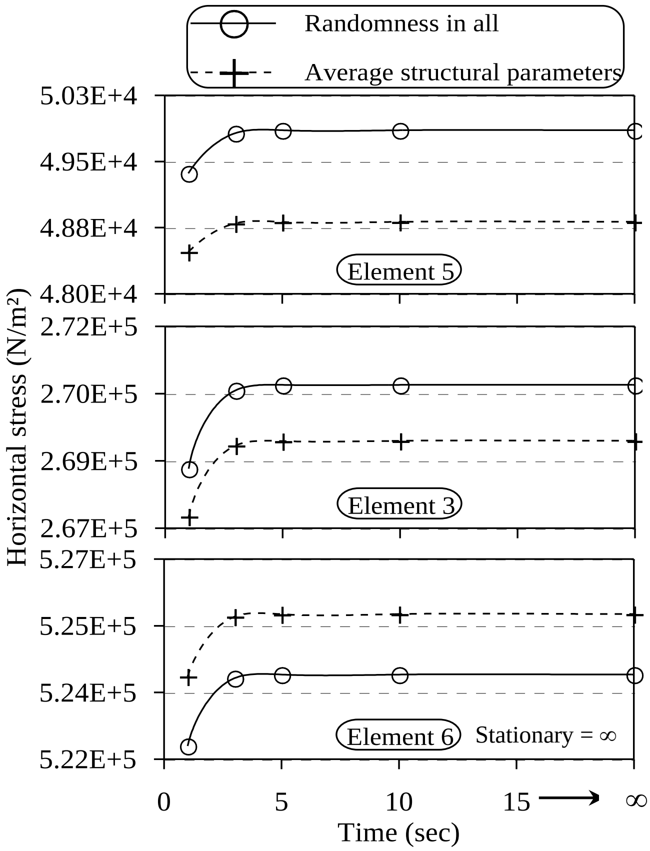


Fig. 9. Means of non-stationary horizontal stress at the center of elements E5, E3, and E6, respectively

To examine the effect of non-stationary excitation on stochastic response with average and randomness structural parameters (overall) of an asphaltic lining dam-foundation system, the analyses of the dam are calculated at times 1, 3, 5, 10, and 20 seconds and compared with the stationary responses for the spatially varying ground motion (general excitation case including the wave passage and incoherence effects).

The means of non-stationary horizontal displacements at the marked nodes (Fig. 3) are plotted in Fig.8. It is seen from the figures that the values due to the randomness of structural parameters are close to those of average structural parameters. In addition to this, these three dam stationary response levels are reached within approximately 5 sec for both average and randomness structural parameters. While at t = 1 sec the non-stationary displacements are smaller than the corresponding stationary ones, at t = 5 sec, the non-stationary displacements are very close to the stationary ones in both cases. At node N5, the displacement values of 98.57% of stationary response are achieved at the time of 1 sec of the transient response. At node N3, the displacement values 98.82% of stationary response is achieved at the time of 1 sec of the transient response for randomness structural parameters. Similarly, at node N6, the displacement values of 99.27% of stationary response are achieved at the time of 1 sec of the transient response.

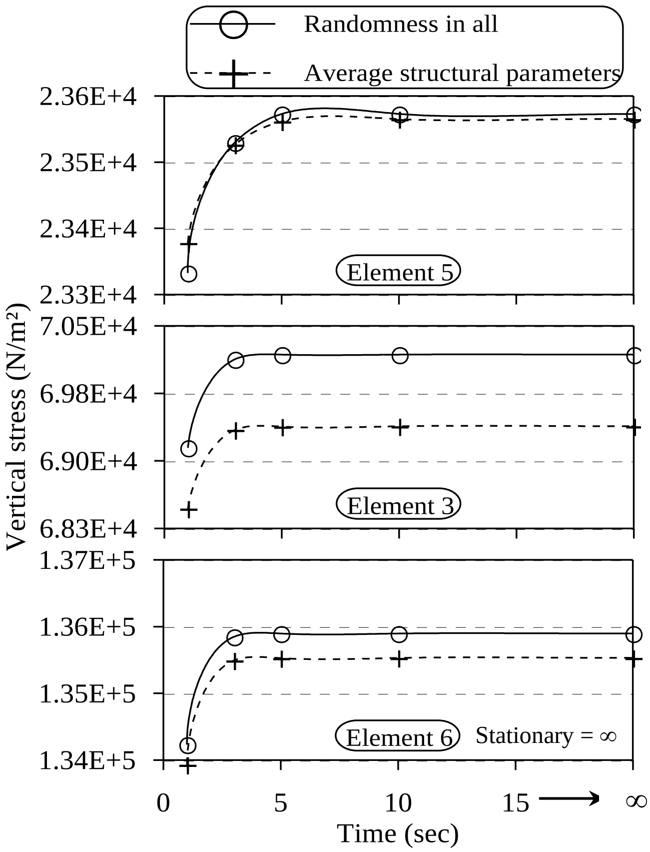


Fig. 10. Means of non-stationary vertical stress at the center of elements E5, E3, and E6, respectively

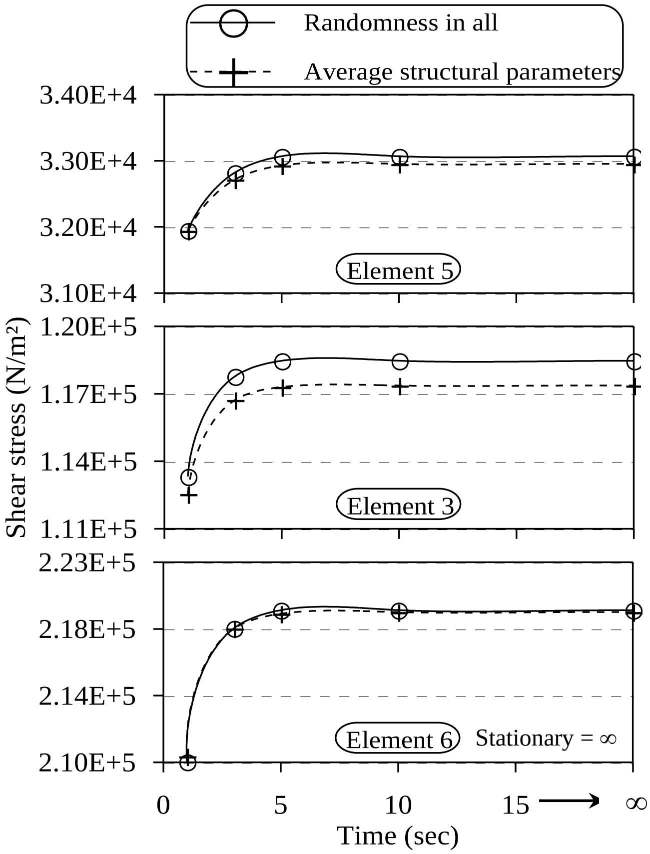


Fig. 11. Means of non-stationary shear stress at the center of elements E5, E3, and E6, respectively

The mean horizontal, vertical, and shear stress at the marked center of the elements (Fig.3) are shown in Figs. 9-11. It can be said from these figures that the stationary stress values obtained from the spatially varying ground motion are close to the non-stationary ones for both average and randomness structural parameters. In addition, it is seen from the stress graphs that the stresses obtained for both randomness and average structural parameters are very close to each other, except for Element 5 (E5) in vertical stresses and Element 5, 6 (E5, E6) in shear stresses. For randomness structural parameters, the stationary horizontal, vertical, and shear stress values at Element 5 (E5) are larger than as much as 0.98%, 1.29%, and 3.51% when compared to the non-stationary ones at the time of 1 sec, respectively. At Element 3 (E3), the stationary horizontal, vertical, and shear stress values are larger than as much as 0.70%, 1.50%, and 4.54% when compared to the non-stationary ones at the time of 1 sec, respectively. Lastly, at Element 6 (E6), the stationary horizontal, vertical, and shear stress values are larger than as much as 0.31%, 1.24%, and 4.55% when compared to the non-stationary ones at the time of 1 sec, respectively.

These figures indicate that for ground motion duration considered as t = 20.95 sec to compute means of stationary responses, close response values are obtained from stationary and non-stationary response analyses, especially for $t \geq$ 5 sec. For the selected example dam-foundation system, the assumption of stationary response is satisfactory for considered ground motion duration.

7. Conclusion

This paper presents the influence of the randomness associated with both the ground motion and structural parameters on the dynamic response of an asphaltic lining dam-foundation system. A Monte Carlo simulation based on the stochastic finite element method is used to investigate the uncertainty of structural parameters. This method, despite being computationally intensive, is found to be realistic. In this study, the spatially varying ground motion including the wave-passage and incoherence effects together is taken into account as stationary as well as non-stationary random seismic excitation.

Obtained results herein indicate that the uncertainty of structural parameters (shear modulus and mass density) does not show any significant influence on the stochastic response of the asphaltic dam-foundation system. However, in the dynamic analysis of asphaltic dam-foundation systems subjected to spatially varying ground motion, more sample structure models and more parameter variability need to be considered to generalize the effects of variation in structural parameters on these types of dams. This study is only important as it is a precursor to the future work of engineers.

In addition, in this study, both stationary and non-stationary responses are computed for the dam-foundation system with the uncertainty of structural parameters. When comparing the non-stationary responses obtained at various times with those of the stationary ones, it is observed that the stationary assumption is reasonable for the selected dam-foundation system. More extensive studies are needed to generalize such analyzes in such structures.

Declaration of conflicting interests

The author(s) declared no potential conflicts of interest with respect to the research, authorship, and/or publication of this article.

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