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Shear strain related non-linear stochastic dynamic analysis of rock-fill dams

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ABSTRACT

The effect of the non-linear material behavior of a rock-fill dam subjected to random loads is investigated by the equivalent linear method that considers the non-linear variation of soil shear moduli and damping ratios as a function of shear strain. The Keban dam constructed in Elazığ, Turkey is chosen as a numerical example. The interaction of the rock-fill dam with the reservoir is neglected, but not the foundation rock. The properties of the dam materials were taken from the dam project and assumed to be isotropic in the analysis. A stationary and ergodicity assumption are made for stochastic dynamic analysis. The E-W component of the Erzincan earthquake recorded on March 13, 1992, Erzincan, Turkey is chosen as a ground motion since it occurred nearby the dam site. The component considered is applied to the dam in the horizontal direction. The non-linear stochastic responses of the Keban dam are compared to its linear stochastic and deterministic response.

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1. Introduction

Rock-fill dams are constructed for various purposes such as irrigation, energy production, flood control and recreation. A serious damage on these dams has been not recorded in the literature due to an earthquake ground motion. Accordingly, it can be said that rock-fill dams are highly resistant to seismic loads. The satisfactory seismic behaviour of these dams is due to the capacity of the rock-fill body.

Gazetas and Dakoulas (1992) have presented comprehensive reviews on theoretical methods for estimating the dynamic response and the performance of earth and rock-fill dams subjected to strong earthquake ground motions. Several factors such as liquefaction effects, non-linear material behaviour, and permanent deformations affect the dynamic response of earth and rock-fill dams during the earthquakes. Linear and non-linear earthquake responses of earth and rock-fill dams including these factors were carried out by many researchers (Seed, 1979; Sayed and Abdel-Ghaffar, 1992; Khoei et al., 2004).

The highly non-linear and hysteretic material behaviour may considerably affect the seismic response of rock-fill dams. Traditionally, an equivalent linear method is used to determine the non-linear response of rock-fill dams to earthquakes. Seed and Idriss (1969) used the equivalent linear method to incorporate the observed strain-dependent non-linear behaviour of soils. Vrymoed (1981) studied the dynamic analysis of a dam by using the equivalent linear method. Mejia et al. (1982) used the same method for the three-dimensional dynamic analysis of earth dams. These studies were performed by using the deterministic methods.

Because of the randomness of earthquake ground motions, the researchers have started to use the random vibration theory for estimating the dynamic response of embankment dams. Singh and Khatua (1978) reported probabilistic techniques in assessing the seismic stability of earth dams. Gazetas et al. (1982) developed a new random vibration procedure to estimate the statistics of the non-linear hysteretic response of earth dams modelled as inhomogeneous shear slices and excited by strong motions consisting of vertical shear waves.

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In recent years, the stochastic seismic responses of earth dams have also been investigated by only a limited number of researchers (Mellah et al., 2000; Chen and Harichandran, 2001; Hacıefendioğlu, 2006). However, it can be seen from the literature review that a few works on stochastic response of rock-fill dams to earthquake ground motion have been studied. Therefore, the objective of this study is to determine the non-linear seismic response of the Keban dam, which is a rock-fill dam, to random ground motions using the finite element method.

2. Simple Method for Non-linear Response

In this study, equivalent linear method is used in an iterative way for stochastic analysis in which non-linear material behavior is considered. Strain properties of the materials in each finite element are defined by a shear modulus and an equivalent damping ratio which depends on the shear strain. By considering low-strain (10^{-4}%), initial values of the shear modulus and the damping ratio are taken into consideration for each element. With given values of the shear modulus and the damping ratio, a linear elastic analysis is performed to determine the stochastic dynamic response. An effective strain, which is usually considered as the maximum value for the stochastic analysis, is computed in each finite element. It is noted that, in establishing the effective strain, it is not necessary to resort to arbitrary scaling of the computed strain values as in the deterministic methods where a strain reduction factor on the computed value of strain is applied. That is, the effective strain is used as the mean value of the random process describing the maximum value of the strain. Then the moduli and the damping ratios are selected for the computed effective strain and used for the next iteration (Gazetas et al., 1982). This procedure is repeated until the differences of the moduli and the damping ratios between two iterations are very small. The response value obtained at the last iteration is considered as the true nonlinear response.

The maximum dynamic shear modulus of cohesionless materials, G_{max}, is computed by using the following expression (Seed and Idriss, 1970)

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

where σ_m is mean stress. Values of $(K_2)_{max}$ determined by laboratory tests have been found to vary from 150 to about 250 for compacted gravels and rock-fill. Experimental data from the literature on the shear strain dependent moduli and damping for rock-fill materials are depicted in Fig. 1(a) (Seed et al., 1986). The shear modulus values for saturated cohesive soils have been found to vary with the undrained shear strength level as

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

where $s_u = c + \sigma_m \tan\phi$ is the undrained shear strength, c is the cohesion factor and ϕ is the angle of internal friction. The variations of the shear modulus and the damping ratios with shear strain for clay material is presented in Fig. 1(b) (Sun et al., 1988; Idriss, 1990). Finally, the variations of the shear modulus and the damping ratios with shear strain for rock material is shown in Fig. 1(c) (Schnabel et al., 1972).

3. Formulation of Stochastic Analysis

An acceleration-time history of ground motion recorded at one point is used as seismic input in the deterministic method. In the stochastic method, however, recorded ground motions appropriate to the site are characterized by statistically. Since the ground motion caused by seismic disturbance is random, the best way to characterize the random excitation statistically is to employ a power density function and autocorrelation function. So, the stochastic parameters describing the seismic output can be determined from the power spectral density function of the seismic input.

In this study, a stationary assumption where the statistical parameters are independent of time is made for stochastic analysis. Besides, the ergodicity assumption is made to use only one earthquake record.

If a single ground acceleration record is used for the input, the cross power spectral density function, $S_{ij}(\omega)$, can be determined by using the equation of motion of the system by the following equation (Dumanoğlu and Severn, 1990).

$$S_{ij}(\omega) = S_{in}(\omega) \sum_{r=1}^{N} \sum_{s=1}^{N} \psi_{ir}\psi_{js} H_{ir}(\omega) H_{js}^{*}(\omega) , \tag{3}$$

where ω is the frequency; $H(\omega)$ is the frequency response function; $S_{in}(\omega)$ is the power spectral density function of the ground motion; N is the number of modes which are considered to contribute to the response; ψ_{ir} is the contribution of the rth mode to $U_j(t)$ displacement and * denotes the complex conjugate. The expected maximum value (μ) is the mean value of all maximum values and can be expressed as

$$\mu = p\sqrt{\lambda_0} , \tag{4}$$

where p is the peak factor and λ_0 is the initial spectral moment (Der Kiureghian, 1980).

4. Numerical Example

In this study, the Keban dam constructed in Elazığ, Turkey is chosen as a numerical example to investigate the non-linear stochastic response of rock-fill dams by the finite element method. The finite element mesh of the dam is shown in Fig. 2.

The Keban dam is 163 m high from riverbed. The crest has a length of 1097 m. The main purpose of the dam is to regulate river flow and supply energy. In the finite element mesh of the dam, there are 326 nodes and 286 quadrilateral elements. The dam is treated as a plane strain problem. The interaction of the rock-fill dams with the reservoir has generally neglected (Priscu et al., 1985). Therefore, the interaction with the reservoir is accordingly ignored, but not the foundation rock.

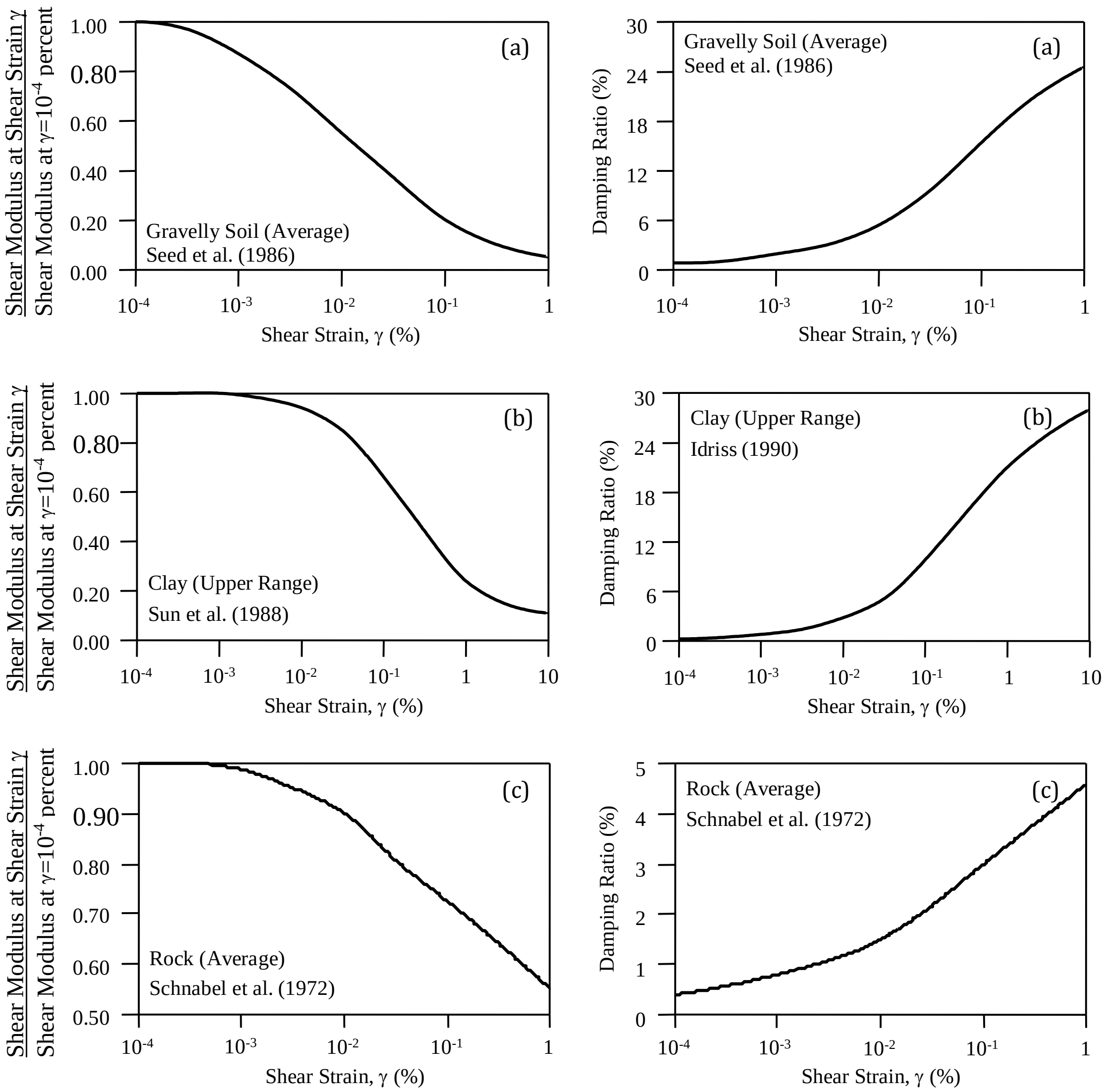

Fig. 1. The variation of shear modulus and damping ratios for (a) gravelly soil, (b) clay material and (c) rock material, respectively.

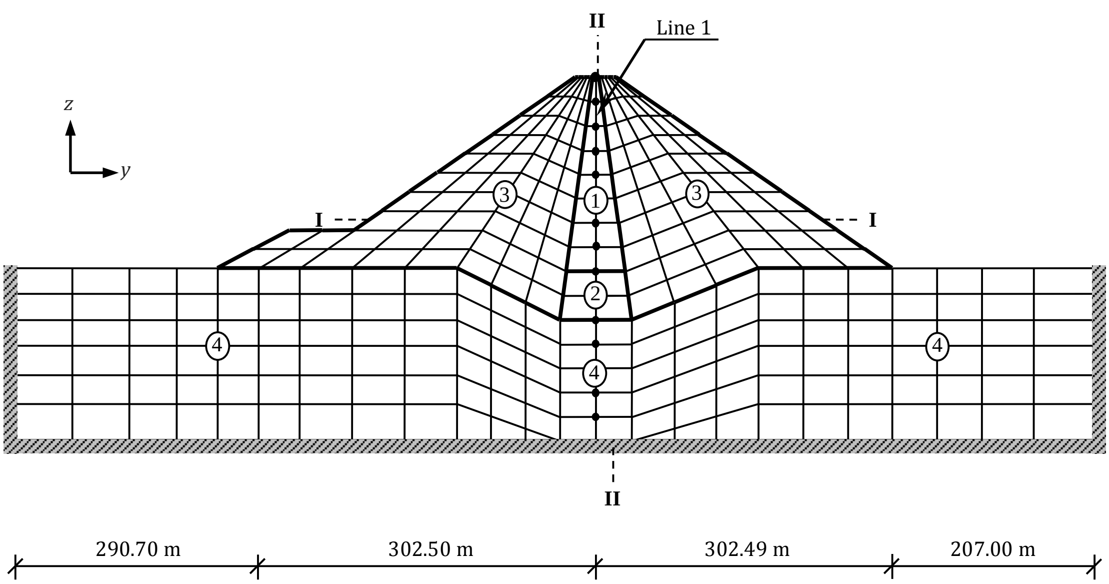

Fig. 2. Finite element mesh of the Keban dam.

Materials in the dam section can be grouped in three main categories: the impervious clay core (material number 1) flanked by transition filters, a concrete core (material number 2) at the bottom of the dam and the compacted rock-fill (material number 3) placed at various lifts. The properties of these materials taken from the dam project are as follows: For the impervious clay core, mass density ρ=2089.70 kg/m^3, and Poisson's ratio ν=0.45; for the concrete core, mass density ρ=2446.48 kg/m^3, and Poisson's ratio ν=0.15. The elasticity modulus, mass density and Poisson's ratio of the foundation rock are taken as 1.379×10^{10} N/m^2, 2689.09 kg/m^3, and 0.24, respectively. The cohesion constant is 15 kN/m^2 and the angle of friction is equal to 20^o for the saturated clay core. $(K_2)_{max}$ factor is given as 170 at small-strains for the dynamic modulus coefficient of the gravel material. 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 non-linear stochastic response analysis of the rock-fill dam.

The E-W component of the Erzincan earthquake recorded on March 13, 1992, Erzincan, Turkey is chosen as ground motion since it occurred nearby the dam site. The component is applied to the dam in the upstream-downstream direction. The power spectral density (PSD) function of the Erzincan earthquake is determined with the Fourier transforms of the autocorrelation function. Fig. 3 shows the E-W component of the Erzincan Earthquake and its power spectral density function. The calculated intensity parameter value is S_0=0.00593 m^2/s^3. Filter parameter values proposed by Der Kiureghian and Neuenhofer (1991) are utilized as ω_g=10.0 rad/s, ξ_g=0.4, ω_f=1.0 rad/s, and ξ_f=0.4.

In this paper, the dynamic response of the Keban dam subjected to the Erzincan earthquake is also obtained by the deterministic method. The linear and non-linear results obtained from the stochastic analysis and the linear results obtained from the deterministic analysis are compared to each other. The dynamic responses of the Keban dam are calculated for a time interval of 0.00225 sec.

In clay core, the initial shear moduli and shear moduli obtained from the non-linear analysis are shown in Fig. 4. As shown in Fig. 4, the shear moduli obtained from the non-linear analysis are smaller than the initial shear modulus. Fig. 4 shows also the variation of shear strain with the height of the dam. It is seen from Fig. 4 that the shear strain values obtained from the non-linear analysis inrease with the height of the dam as well as the initial shear strain used for the linear analysis (value (%)=10^{-4}).

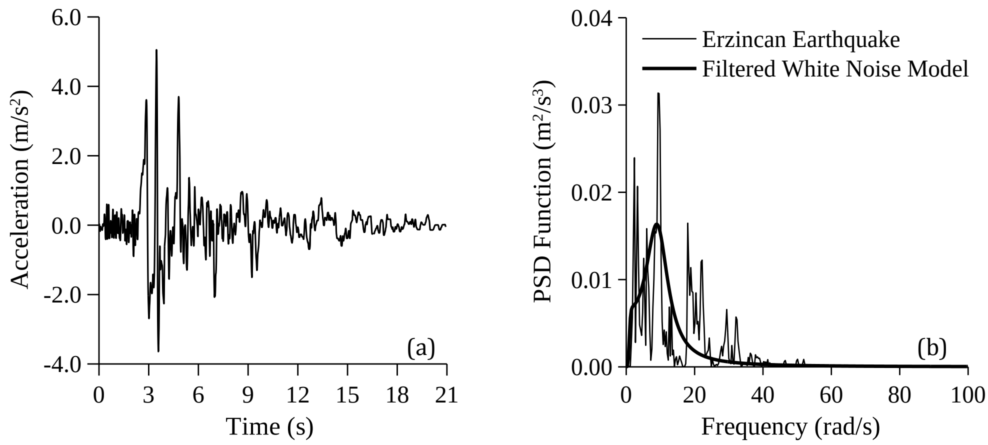

Fig. 3. a) The E-W component of the Erzincan earthquake recorded on March 13, 1992, Erzincan, Turkey and b) its power spectral density (PSD) function.

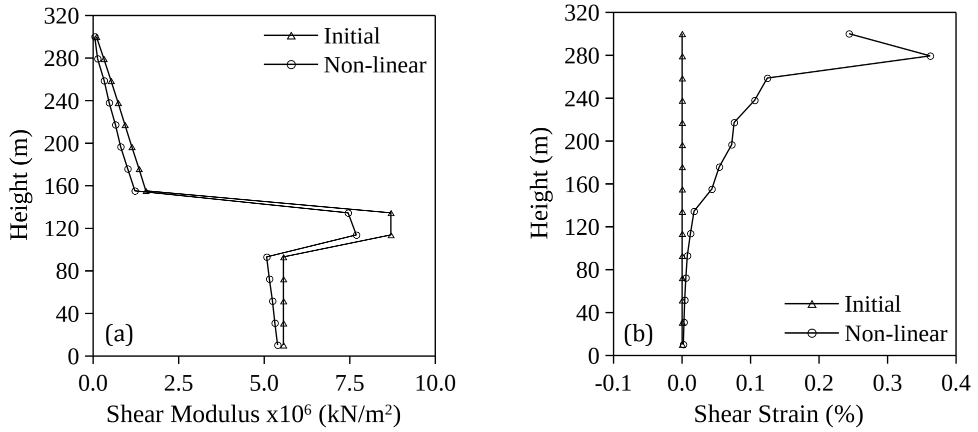

Fig. 4. a) Initial shear moduli versus shear moduli and b) initial shear strains versus shear strains obtained from the non-linear analysis.

4.1. Displacements

Mean of maximum values of displacements are calculated from the stochastic dynamic analyses while the absolute maximum values of displacements are obtained from deterministic dynamic analysis. Horizontal displacements at the marked nodes on line 1 (nodes along the core of the dam in Fig. 2) obtained from the deterministic analysis (for linear material behavior) and the stochastic analyses (for linear and non-linear material behaviors) of the Keban dam are plotted in Fig. 5. It is seen from Fig. 5 that the expected maximum values of horizontal displacements obtained from the stochastic analyses are smaller than the absolute maximum horizontal displacements obtained from the deterministic analysis. In addition, the non-linear displacements are smaller than linear displacements for stochastic analysis.

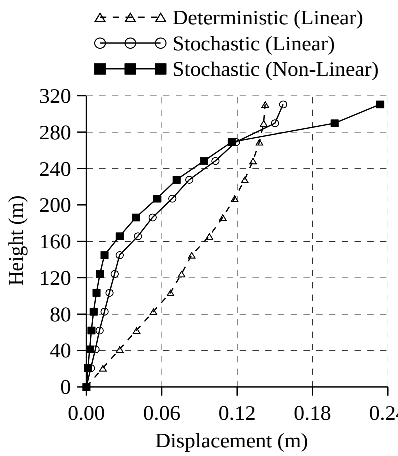

Fig. 5. Horizontal displacements at the marked nodes on line 1.

4.2. Stresses

The stress components, which are obtained from the stochastic and deterministic dynamic analyses, are also compared with each other. The stress values are calculated at the middle points of the elements. Horizontal, vertical and shear stress components on sections I-I are compared in Figs. 6-8 while those on section II-II are compared in Fig. 9. It can be seen from Figs. 6-9 that the expected maximum values of all stress components for stochastic analyses (for linear and non-linear material behaviors) are smaller than the absolute maximum stresses for the deterministic analysis (for linear material behavior).

5. Conclusions

Non-linear material behavior of the Keban dam, which is a rock-fill dam, subjected to random loads is investigated by the equivalent linear method which considers the non-linear variation of soil shear moduli and damping ratios as a function of shear strain.

It is observed that for the non-linear displacement and stress responses obtained from the stochastic analysis are smaller than the linear responses obtained from the stochastic and deterministic analyses. In addition, all displacement and stress results obtained from deterministic analysis are greater than the mean of maximum values obtained from stochastic analyses.

Because the mean of maximum values obtained from stochastic analyses is calculated by averaging all the maximum response values, it should be expected that the absolute maximum values obtained from deterministic analysis would be greater than the mean of maximum values.

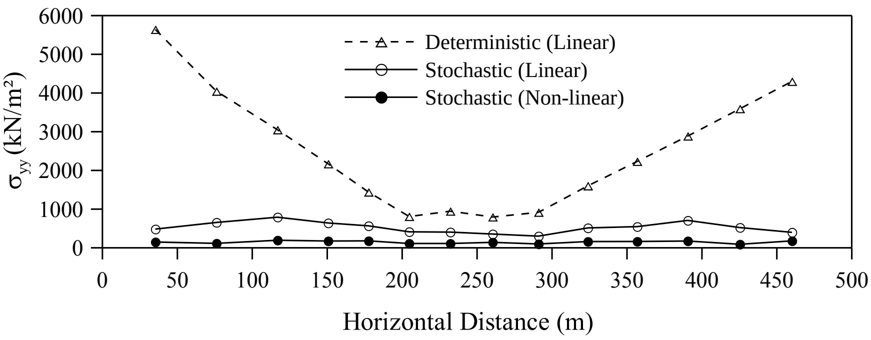

Fig. 6. Horizontal stresses (σ_{yy}) on section I-I of the Keban dam.

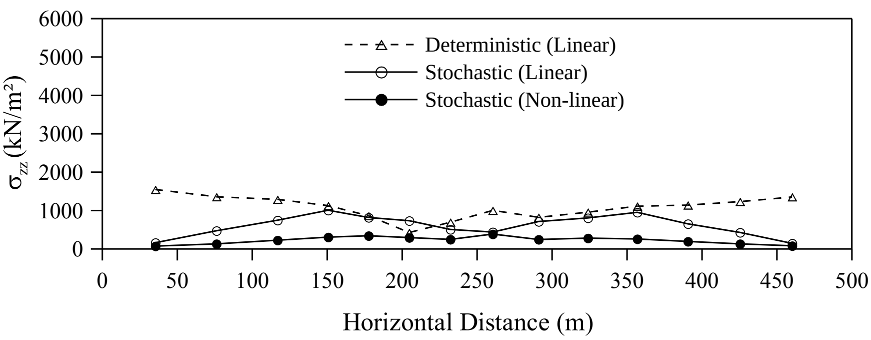

Fig. 7. Vertical stresses (σ_{zz}) on section I-I of the Keban dam.

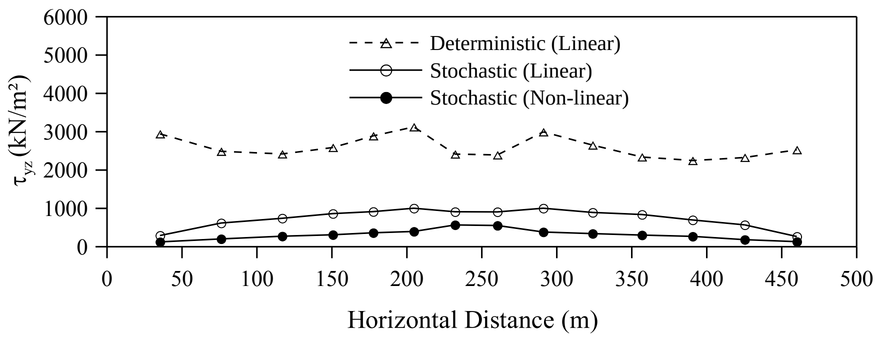

Fig. 8. Shear stresses (τ_{yz}) on section I-I of the Keban dam.

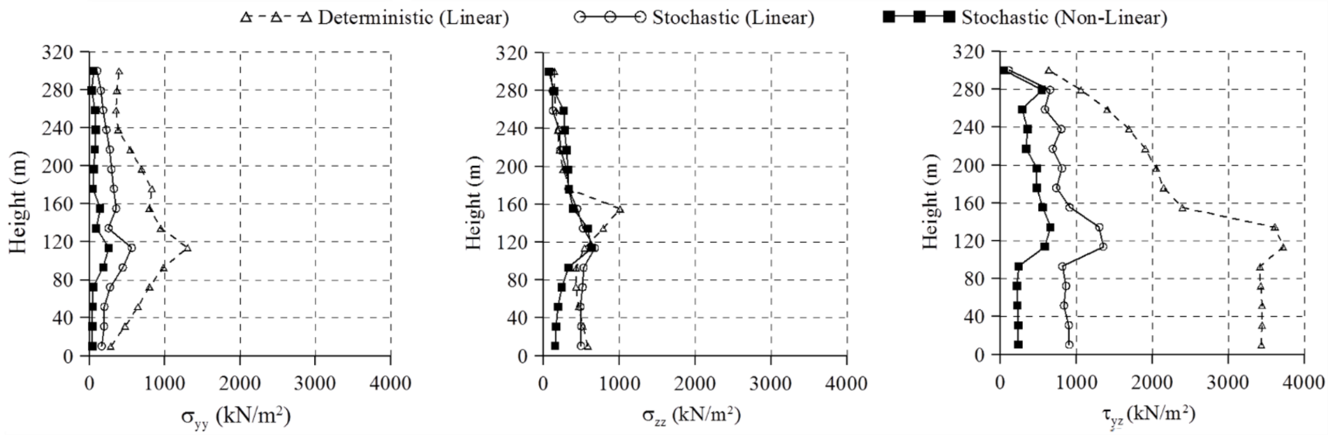

Fig. 9. Horizontal (σ_{yy}), vertical (σ_{zz}) and shear stresses (τ_{yz}) on section II-II of the Keban dam.

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