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Comparison of uniform and spatially varying ground motion effects on the stochastic response of fluid-structure interaction systems

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Abstract. The effects of the uniform and spatially varying ground motions on the stochastic response of fluid-structure interaction system during an earthquake are investigated by using the displacement based fluid finite elements in this paper. For this purpose, variable-number-nodes two-dimensional fluid finite elements based on the Lagrangian approach is programmed in FORTRAN language and incorporated into a general-purpose computer program SVEM, which is used for stochastic dynamic analysis of solid systems under spatially varying earthquake ground motion. The spatially varying earthquake ground motion model includes wave-passage, incoherence and site-response effects. The effect of the wave-passage is considered by using various wave velocities. The incoherence effect is examined by considering the Harichandran-Vanmarcke and Luco-Wong coherency models. Homogeneous medium and firm soil types are selected for considering the site-response effect where the foundation supports are constructed. A concrete gravity dam is selected for numerical example. The S16E component recorded at Pacoima dam during the San Fernando Earthquake in 1971 is used as a ground motion. Three different analysis cases are considered for spatially varying ground motion. Displacements, stresses and hydrodynamic pressures occurring on the upstream face of the dam are calculated for each case and compare with those of uniform ground motion. It is concluded that spatially varying earthquake ground motions have important effects on the stochastic response of fluid-structure interaction systems.

Keywords: spatially varying earthquake motion; stochastic analysis; Lagrangian approach; fluid finite element, fluid-structure interaction.

1. Introduction

The investigation of coupled fluid-structure systems subjected to dynamic loads is a research field of particular importance. The structure affects the behaviour of the fluid as well as the fluid affecting the behaviour of structure under a dynamic effect. So, hydrodynamic pressures in the fluid and additional loads in the structure due to hydrodynamic pressures occur (Hall and Chopra 1982, 1983).

Most fluid-solid interaction analyses are based on simplifying assumptions (e.g., inviscid flow)

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which allow one of two approaches (Olson and Bathe 1983): Firstly, displacements are the variables in the solid, pressures (or velocity potentials) are the variables in fluid according to Eulerian approach; the second type of approaches, displacements are the variables in both the fluid and solid according to Lagrangian approach. Since the variables in fluid and solid are different in Eulerian approach, a special-purpose computer program is required for the solution of the coupled systems. Because, existing formulations generally involve asymmetric matrices that render them difficult to incorporate in general finite element analysis programs. However, in the Lagrangian approach, compatibility and equilibrium are automatically satisfied at the nodes along the interfaces between the fluid and solid. This makes a Lagrangian displacement-based fluid finite element very applicable, which can be readily incorporated into a general-purpose computer program for structural analysis, because special interface equations are not required.

Many studies, loads due to earthquake forces, which are one of the most important actions in the design of structures are considered as deterministic. Since seismic waves are initiated by irregular slippage along faults followed by numerous random reflections, refractions, and attenuations within the complex ground formations through which they pass, seismic actions have essentially stochastic characteristics and they should be considered as random loads (Lin 1967). Dynamic analysis of large fluid-structure systems subjected to random loads has been performed in (Araujo and Awruch 1998, Bayraktar and Hançer 2005, Hançer and Bayraktar 2004, Bayraktar *et al.* 2005). However, earthquake ground motion which was considered uniform ground motion with infinite velocity was considered in these studies. Because of the complex nature of earth crust, the spatial variability of ground motions should be taken into account in the dynamic analysis over the base dimensions of large structures such as dams (Bilici *et al.* 2009, Hacıefendioğlu 2006, Chen and Harichandran 2001). Spatial variability of earthquake motions includes incoherence, wave-passage and site-response effects. As the incoherence effect results from reflections and refractions of seismic waves through the soil during their propagation, the wave passage effects results from the difference in the arrival times of waves at support points and the site response effect arises from the differences in local soil conditions at the supports of the structures (Dumanoğlu and Soyluk 2002).

The focus of the present paper is to compare uniform and spatially varying ground motion effects on the stochastic response of large fluid-structure interaction system during earthquakes by using the Lagrangian (displacement-based) fluid finite elements. For that reason, variable number-nodes two-dimensional fluid finite elements proposed by Wilson and Khalvati (1983) were programmed in FORTRAN language by the authors and incorporated into a computer program, SVEM (Dumanoğlu and Soyluk 2002), which is used for stochastic dynamic analysis of solid systems under spatially varying earthquake ground motion. The program SVEM (Dumanoğlu and Soyluk 2002) is modified for the stochastic dynamic analysis of fluid-structure systems subjected to spatially varying earthquake ground motions by using the Lagrangian approach and named as SVEMF. The program SVEMF is used in the stochastic dynamic analysis of the coupled system for the spatially varying earthquake motions.

2. Formulation

In this section, formulation of fluid systems based on the Lagrangian approach is obtained by using the finite element method and stochastic dynamic analysis formulation of fluid-structure interaction systems is given based on spatially varying earthquake motions.

2.1 Finite element formulation of fluid systems

The formulation of the fluid system based on Lagrangian approach is given according to (Wilson and Khalvati 1983, Calayır and Dumanoğlu 1993). Fluid is assumed to be linear-elastic, inviscid and irrational. For this fluid, the relation between pressure and volumetric strain is given by

$$P = \beta\varepsilon_v \tag{1}$$

where P is pressure, β is the bulk modulus of the fluid, and ε_v is the volumetric strain. For two-dimensional problems, ε_v can be expressed in terms of displacements as

$$\varepsilon_v = \frac{\partial u_{fy}}{\partial y} + \frac{\partial u_{fz}}{\partial z} \tag{2}$$

where u_{fy} and u_{fz} are the components of the displacement in the y and z directions, respectively.

To enforce the rotational constraint, the following rotation is defined as

$$w = \frac{1}{2}\left(\frac{\partial u_{fy}}{\partial z} - \frac{\partial u_{fz}}{\partial y}\right) \tag{3}$$

where w is rigid body rotation about the axis normal to the plane. The relation between the stress and stiffness associated with this rotation is given by

$$P_w = \alpha w \tag{4}$$

where P_w and α are the rotational stress and stiffness (constraint parameter), respectively.

The total strain energy of the fluid system using the finite element method can be expressed as follows

$$\Pi_e = \frac{1}{2}\mathbf{u}_f^T\mathbf{K}_f\mathbf{u}_f \tag{5}$$

where $\mathbf{K}_f$ and $\mathbf{u}_f$ are the stiffness matrix and the nodal displacement vector of the fluid system, respectively.

Using the finite element method, the free surface potential energy can be obtained as

$$\Pi_s = \frac{1}{2}\mathbf{u}_{fs}^T\mathbf{S}_f\mathbf{u}_{fs} \tag{6}$$

where $\mathbf{S}_f$ and $\mathbf{u}_{fs}$ are the free surface stiffness matrix and the free surface vertical displacement vector of the fluid system, respectively.

Using the finite element method, kinetic energy of the fluid can be written in the form

$$T = \frac{1}{2}\dot{\mathbf{u}}_f^T\mathbf{M}_f\dot{\mathbf{u}}_f \tag{7}$$

where $\mathbf{M}_f$ and $\dot{\mathbf{u}}_f$ are the mass matrix and the nodal velocity vector of the fluid system, respectively.

The direct application of Lagrange's equation (Clough and Penzien 1975) yields the following set of equations

$$\mathbf{M}_f\ddot{\mathbf{u}}_f + \mathbf{K}_f^*\mathbf{u}_f = \mathbf{F}_f \tag{8}$$

where $\mathbf{K}_f^*$ and $\mathbf{F}_f$ are the system stiffness matrix including the free surface stiffness and time-varying nodal forces vector for the fluid system, respectively.

The equations of motion for a fluid system, Eq. (8), have a similar form to that of the structure when the Lagrangian approach is used. But, it requires a different sensitivity to determine interface condition of the coupled system. At the interface of the fluid-structure system, only the displacements in the direction normal to the interface are assumed to be compatible in the structure as well as in the fluid. This condition is imposed by the constraint equations (Bathe 1982). Using the interface condition, the equations of motion of coupled systems subjected to ground motion including damping effects are given by

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = -\mathbf{M}\boldsymbol{\delta}\ddot{u}_g(t) \tag{9}$$

where **M**, **C**, **K** are $n \times n$, positive definite, mass, damping and stiffness matrices; $\mathbf{u}(t)$, $\dot{\mathbf{u}}(t)$ and $\ddot{\mathbf{u}}(t)$ are the vectors of displacement, velocity and acceleration of the coupled system, respectively. $\boldsymbol{\delta}$ is the direction vector that links the mass terms to the ground acceleration, $\ddot{u}_g(t)$. As is seen from Eq. (9), the equation of motion of a fluid-structure system based on the Lagrangian approach is the same form as that of the structural systems.

2.2 Stochastic(Random) response

Since the formulation of the random vibration theory for spatially varying ground motion is given previously by many researchers (Harichandran *et al.* 1996, Soyluk *et al.* 2004, Dumanoğlu and Soyluk 2003, Harichandran and Wang 1988), in this study only the required final equations will be considered briefly. The variance of the *i*th total response can be obtained as follows (Harichandran *et al.* 1996)

$$\sigma_{z_i}^2 = \sigma_{z_i}^{2^{qs}} + \sigma_{z_i}^{2^d} + 2Cov(z_i^{qs}, z_i^d) \tag{10}$$

where $\sigma_{z_i}^{2^{qs}}$ and $\sigma_{z_i}^{2^d}$ are the pseudo-static and dynamic variances, respectively, and $Cov(z_i^{qs}, z_i^d)$ is the covariance between the pseudo-static and dynamic responses. The three components on the right-hand side of Eq. (10) are given by

$$S_{z_i}^{qs}(\omega) = \frac{1}{\omega^4}\sum_{l=1}^{r}\sum_{m=1}^{r} A_{il}A_{im}S_{\ddot{u}_{gl}\ddot{u}_{gm}}(\omega) \tag{11}$$

$$S_{z_i}^{d}(\omega) = \sum_{j=1}^{n}\sum_{k=1}^{n}\sum_{l=1}^{n}\sum_{m=1}^{n} \Psi_{ij}\Psi_{ik}\Gamma_{lj}\Gamma_{mk}H_j(-\omega)H_k(\omega)S_{\ddot{u}_{gl}\ddot{u}_{gm}}(\omega) \tag{12}$$

$$Cov(z_i^{qs}, z_i^d) = \sum_{j=1}^{n}\sum_{l=1}^{r}\sum_{m=1}^{r} \psi_{ij}A_{il}\Gamma_{mj}\left(-\int_{-\infty}^{\infty}\frac{1}{\omega^2}H_j(\omega)S_{\ddot{u}_{gl}\ddot{u}_{gm}}(\omega)d\omega\right) \tag{13}$$

where ω is the circular frequency, 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 static displacements for unit displacements assigned to each support points, Ψ_{ij} is *i*th mode vector, Γ_{lj} is the modal participation factor, $S_{\ddot{u}_{gl}\ddot{u}_{gm}}(\omega)$ is the cross-spectral density function of accelerations between supports l and m; $H_k(\omega)$ is the frequency response function.

The most important parameter in stochastic analysis is mean of maximum value. The mean of maximum value depend on peak factor (p) which is function of the time of the motion and standard deviation of the total response (σ_z). It can be expressed as

$$\mu = p\sigma_z \tag{14}$$

2.3 Spatially varying earthquake ground motion

The spatially varying earthquake ground motion includes incoherence, wave-passage and site-response effects. The cross-spectral density function of the earthquake ground motion, between support points l and m is expressed as Harichandran and Wang (1988)

$$S_{\ddot{u}_{gl}\ddot{u}_{gm}}(\omega) = \gamma_{lm}(\omega)\sqrt{S_{\ddot{u}_{gl}\ddot{u}_{gl}}(\omega)S_{\ddot{u}_{gm}\ddot{u}_{gm}}(\omega)} \tag{15}$$

where $\gamma_{lm}(\omega)$ denotes the coherency function describing the variability of the ground acceleration processes for support degrees of freedom l and m as a function of frequency ω, $S_{\ddot{u}_{gl}\ddot{u}_{gl}}(\omega)$ and $S_{\ddot{u}_{gm}\ddot{u}_{gm}}(\omega)$ are the power spectral densities of the accelerations $\ddot{u}_{gl}$ and $\ddot{u}_{gm}$ at the support points l and m. In the case of homogeneous soil type ($S_{\ddot{u}_{gl}\ddot{u}_{gl}}(\omega) = S_{\ddot{u}_{gm}\ddot{u}_{gm}}(\omega) = S_{\ddot{u}_g}(\omega)$) Eq. (15) becomes

$$S_{\ddot{u}_{gl}\ddot{u}_{gm}}(\omega) = \gamma_{lm}(\omega)S_{\ddot{u}_g}(\omega) \tag{16}$$

The power spectral density function adopted in this paper is the well-known modified Kanai-Tajimi spectrum of earthquake ground motion expressed as the following form suggested by Clough and Penzien (1993)

$$S_{\ddot{u}_g}(\omega) = S_0\frac{\omega_g^4 + 4\xi_g^2\omega_g^2\omega^2}{(\omega_g^2 - \omega^2)^2 + 4\xi_g^2\omega_g^2\omega^2}\frac{\omega^4}{(\omega_f^2 - \omega^2)^2 + 4\xi_f^2\omega_f^2\omega^2} \tag{17}$$

where S_0 is the amplitude of the white-noise process; ω_g and ξ_g are the resonant frequency of and damping of the first filter, and ω_f and ξ_f are the resonant frequency of and damping of the second filter.

In this paper, S_0 is obtained for each soil type by equating the variance of the ground acceleration to the S16E component recorded at Pacoima dam during the San Fernando Earthquake in 1971. The calculated values of the intensity parameter for each soil type using STOCAL-II (Wung and Der Kiureghian 1989) are shown in Table 1. Homogeneous medium and firm soil types are used for the foundation supports and the filter parameters for these soil types which are proposed by Der Kiureghian and Neuenhofer (1992) are also utilized as shown in Table 1.

The S16E component recorded at Pacoima dam during the San Fernando earthquake is given in Fig. 1(a) and lasts for 13.5 s; its power spectral density function is given in Fig. 1(b); acceleration and displacement spectral density functions of the modified filtered white noise ground motion

Table 1 Filter and intensity parameter of white-noise process for different soil types

Soil type	w_g (rad/s)	ξ_g	w_f	ξ_f	S_0
Firm	15.0	0.6	1.5	0.6	0.04801
Medium	10.0	0.4	1.0	0.6	0.07134

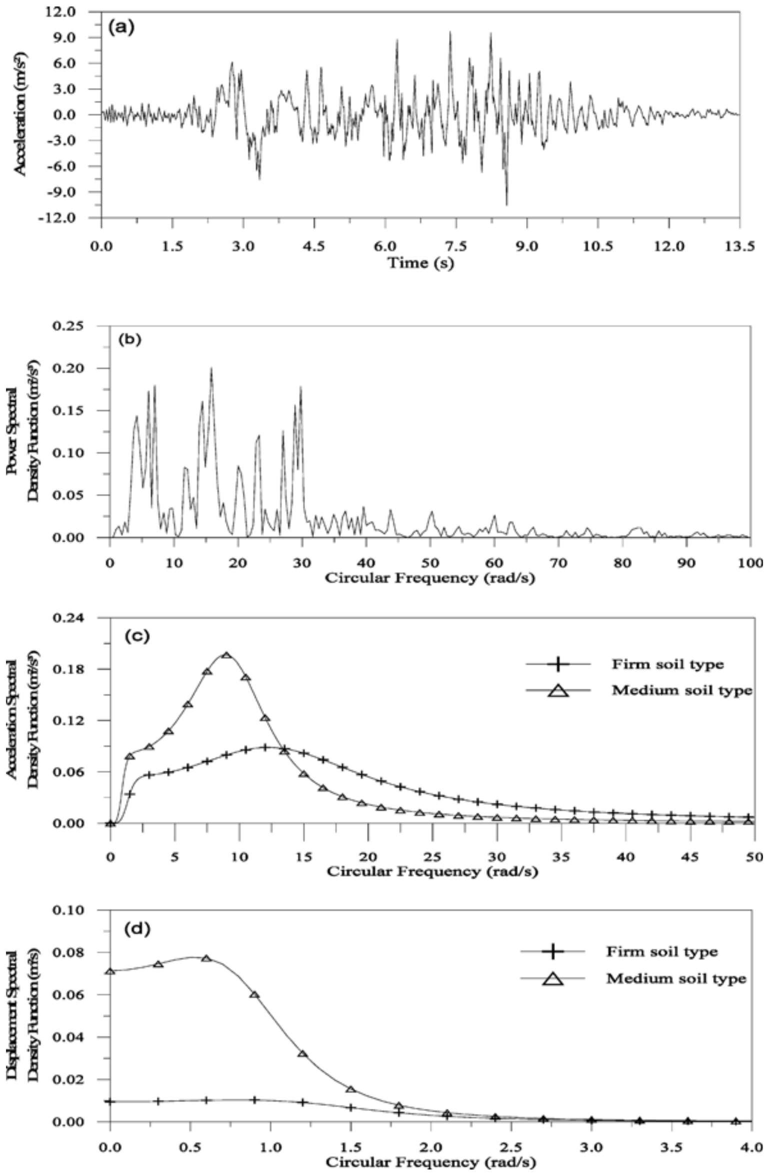

Fig. 1 The S16E component recorded at Pacoima dam during the San Fernando Earthquake in 1970(a), (b) power spectral density function, (c) acceleration and (d) displacement spectral density functions of the modified filtered white noise ground motion model for different soil types

model for different soil types are given in Figs. 1(c) and (d), respectively.

The coherency function is a key component in the characterization of spatially varying earthquake ground motion and describes the coherence between accelerations recorded at different spatial locations. The coherency function is dimensionless and complex valued and defined as (Santa-Cruz *et al.* 2000, Der Kiureghian 1996)

$$\gamma_{lm}(\omega) = |\gamma_{lm}(\omega)|^{i}\gamma_{lm}(\omega)^{w}\gamma_{lm}(\omega)^{s} = |\gamma_{lm}(\omega)|^{i}\exp[i(\theta_{lm}(\omega)^{w} + \theta_{lm}(\omega)^{s})] \tag{18}$$

where $|\gamma_{lm}(\omega)|^i$ characterizes the incoherence effect, $\gamma_{lm}(\omega)^w$ indicates the complex valued wave-passage effect and $\gamma_{lm}(\omega)^s$ denotes the complex valued site-response effect.

For the incoherence effect, resulting from reflections and refractions of waves through the soil during their propagation, several coherency models have been proposed by different investigators based on theoretical and empirical studies (Santa-Cruz *et al.* 2000). The Harichandran and Vanmarcke (1986) model is usually used in this paper, however the Luco and Wong (1986) coherency models are considered in analysis for comparing incoherence function effects. The model proposed by Harichandran and Vanmarcke (1986) is defined as

$$|\gamma_{lm}(\omega)|^i = Ae^{\frac{-2d_{lm}}{\alpha\theta(\omega)}(1-A+\alpha A)} + (1-A)e^{\frac{-2d_{lm}}{\theta(\omega)}(1-A+\alpha A)} \tag{19}$$

$$\theta(\omega) = k\left[1+\left(\frac{\omega}{2\pi f_0}\right)^b\right]^{-\frac{1}{2}} \tag{20}$$

where d_{lm} is distance between support points l and m; A, α, k, f_0, and b are modal parameters, the values are $A = 0.636$, $\alpha = 0.0186$, $k = 31200$, $f_0 = 1.51$ Hz, and $b = 2.95$ (Harichandran and Vanmarcke 1986). These are also used by some researchers (Zerva 1999, Soyluk 2004).

The wave-passage effect due to the difference in the arrival times of waves at support points is defined as Soyluk (2004)

$$\theta_{lm}(\omega)^w = -\frac{\omega d_{lm}^L}{v_{app}} \tag{21}$$

v_{app} is the apparent wave velocity and d_{lm}^L is the projection of d_{lm} on the ground surface along the direction of propagation of seismic waves. The apparent wave velocities employed in this study are selected as 1000 m/s, 1500 m/s, 2000 m/s and infinite wave velocities. Infinite velocity corresponds to uniform ground motion model.

The site-response effect due to the differences in the local soil conditions is obtained as Soyluk (2004)

$$\theta_{lm}(\omega)^s = \tan^{-1}\frac{\mathrm{Im}[H_l(\omega)H_m(-\omega)]}{\mathrm{Re}[H_l(\omega)H_m(-\omega)]} \tag{22}$$

and $H_l(\omega)$ is the local soil frequency response function representing the filtration through soil layers. For the soil frequency response function, a model which idealizes the soil layers as a single degree of freedom oscillator with frequency ω_l and damping ratio ξ_l is used as shown below (Soyluk 2004)

$$H_l(\omega) = \frac{\omega_l^2 + 2i\xi_l\omega_l\omega}{\omega_l^2 - \omega^2 + 2i\xi_l\omega_l\omega} \tag{23}$$

3. Application

3.1 Mathematical model of the Sarıyar Dam

The aim of this paper is to investigate the stochastic dynamic responses of fluid-structure interaction systems to spatially varying earthquake motions based on the Lagrangian approach. For that reason, Sarıyar concrete gravity dam constructed on Sakarya River, which is located 120 km in the northwest of Ankara, Turkey, is selected as an application for fluid-structure interaction systems. The dimensions of the dam are given in Fig. 2(a), and the finite element model of the dam-reservoir-foundation interaction system that is used in the stochastic solutions is given in

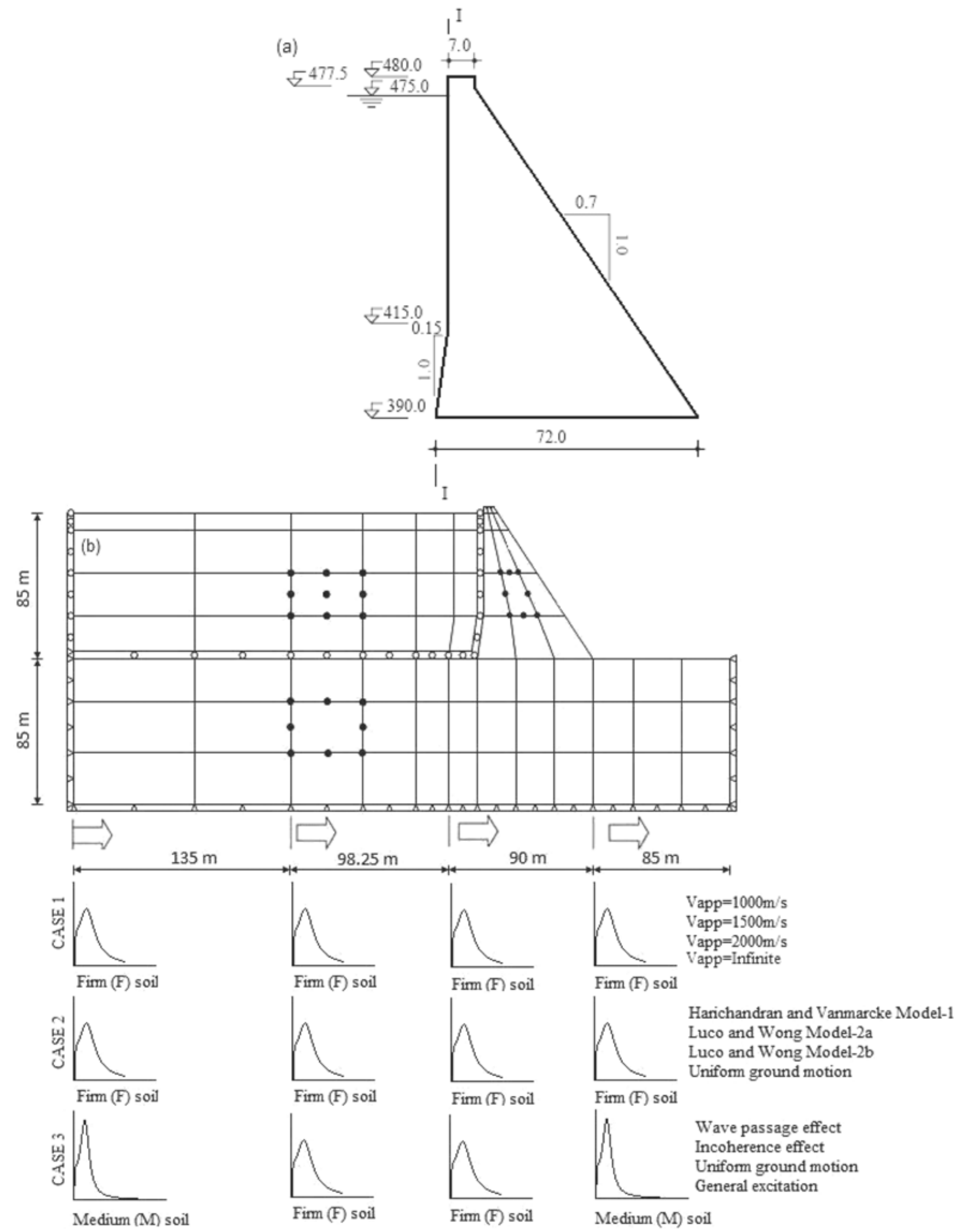

Fig. 2 (a) The dimensions of the Sarıyar concrete gravity dam, (b) the finite element model of the dam-reservoir-foundation interaction system including analysis cases

Fig. 2(b). There are two unknown displacements at each nodal point in the dam, reservoir and foundation model. The dam, the reservoir and the foundation are represented by 15 eight-noded, 24 nine-noded and 36 eight-noded isoparametric quadrilateral solid, fluid and solid finite elements, respectively. Short and axially almost rigid 19 truss elements are used to provide the interface condition. At the fluid-structure interface, truss elements of length and stiffness are chosen as 0.001 m and 1×10^{12} N/m, respectively. Reservoir height is 85 m, the length in the upstream direction is taken to be as much as three times the reservoir height. It is assumed that the reservoir has constant depth.

Element matrices are computed using the Gauss numerical integration technique (Bathe 1982). Integration orders of 2×2 for fluid and 3×3 for solid systems, respectively, are considered in calculations. Plane strain conditions are taken into account in the analyses. The dam material is assumed to be linear elastic, homogeneous and isotropic. For the dam, the elasticity modulus, the unit weight and Poisson's ratio are chosen as 33×10^9 N/m^2, 24×10^3 N/m^3 and 0.20, respectively. For the foundation, these values are 25×10^9 N/m^2, 26×10^3 N/m^3 and 0.30, respectively. For the reservoir, the bulk modulus and the mass density are taken as 207×10^7 N/m^2 and 10 kN/m^3 respectively. A damping ratio of 5% is adopted for the dam-reservoir-foundation system. The value of the rotational constraint parameter is taken as 100 times of the bulk modulus (Wilson and Khalvati 1983). The first 20 modes are taken into account in this study (Calayır and Dumanoğlu 1993).

The S16E component recorded at Pacoima dam during the San Fernando Earthquake in 1971 is applied the dam-reservoir-foundation interaction system in the horizontal direction and the base of the coupled system is divided into four regions as shown in Fig. 2(b). The lengths of the 1st, 2nd, 3rd and 4th regions vary between 0 ± 135 m, 135 ± 213 m, 213 ± 299.25 and 299.25 ± 408.25 m, respectively. 3 apparent wave velocities and 2 soil types are considered for the foundation in the analysis.

Stochastic analyses of the dam-reservoir-foundation interaction system are performed for spatially varying earthquake ground motion by taking into account the incoherence, the wave-passage, and the site response effects. For this purpose, three different analysis cases are considered as follows:

Case 1. All the supports are assumed to be founded on firm (F) soil (FFFF). This case corresponds to the homogeneous soil type. It is only considered wave-passage effect with various wave velocities between support excitation ($\gamma_{lm}(\omega)^i = 1, \gamma_{lm}(\omega)^w \neq 1, \gamma_{lm}(\omega)^s = 1$). The wave velocities are chosen for firm soil 1000 m/s, 1500 m/s, 2000 m/s and infinite velocity.

Case 2. All the supports are assumed to be founded on firm (F) soil (FFFF). This case corresponds to the homogeneous soil type. It is considered only loss of coherency effect ($\gamma_{lm}(\omega)^i \neq 1,\ \gamma_{lm}(\omega)^w = 1, \gamma_{lm}(\omega)^s = 1$). Harichandran and Vanmarcke (1986), Luco and Wong (1986) coherency models are considered in analysis. Harichandran and Vanmarcke coherency model is named Model-1; Luco and Wong coherency model is called Model-2. In this paper, for the Luco and Wong model the rate of α/v_s is chosen 2×10^{-4} s/m for low coherency effect, 1×10^{-3} s/m for high coherency effect. Low and high coherency effect are indicated Model-2a and Model-2b for the Luco and Wong model.

Case 3. While the side supports (1st and 4th regions) are assumed to be founded on medium (M) soil, the mid supports (2nd and 3rd regions) are assumed to be founded on firm (F) soil (MFFM). It is considered the incoherence, the wave passage and the site-response effects ($\gamma_{lm}(\omega)^i \neq 1$, $\gamma_{lm}(\omega)^w \neq 1$, $\gamma_{lm}(\omega)^s \neq 1$) in case 3. The Harichandran and Vanmarcke model was used as incoherency model. The wave velocity was chosen as 1000 m/s. This case is named as general excitation case in this study.

All of above situations were compared uniform ground motion model. Uniform ground motion model is considered by ignoring wave-passage and loss of coherency effects between support excitations in the homogeneous firm (F) soil condition ($\gamma_{lm}(\omega)^i = 1, \gamma_{lm}(\omega)^w = 1, \gamma_{lm}(\omega)^s = 1$). It is assumed that the earthquake waves propagate with infinite velocity in the uniform ground motion model.

3.2 Mean of maximum values of response components for Case 1

3.2.1 Displacements

Mean of maximum quasi-static, dynamic and total horizontal displacements on the upstream face of the dam including foundation are calculated for four different wave velocities defined above and compared in Fig. 3. Of all cases are compared with each other, the displacements obtained for infinite wave velocity case has the biggest values. Infinite velocity case corresponds to uniform ground motion. It can be seen from Fig. 3 that the changing of the wave velocities affects displacements considerably. Through these results, it is clearly seen that the quasi-static, dynamic

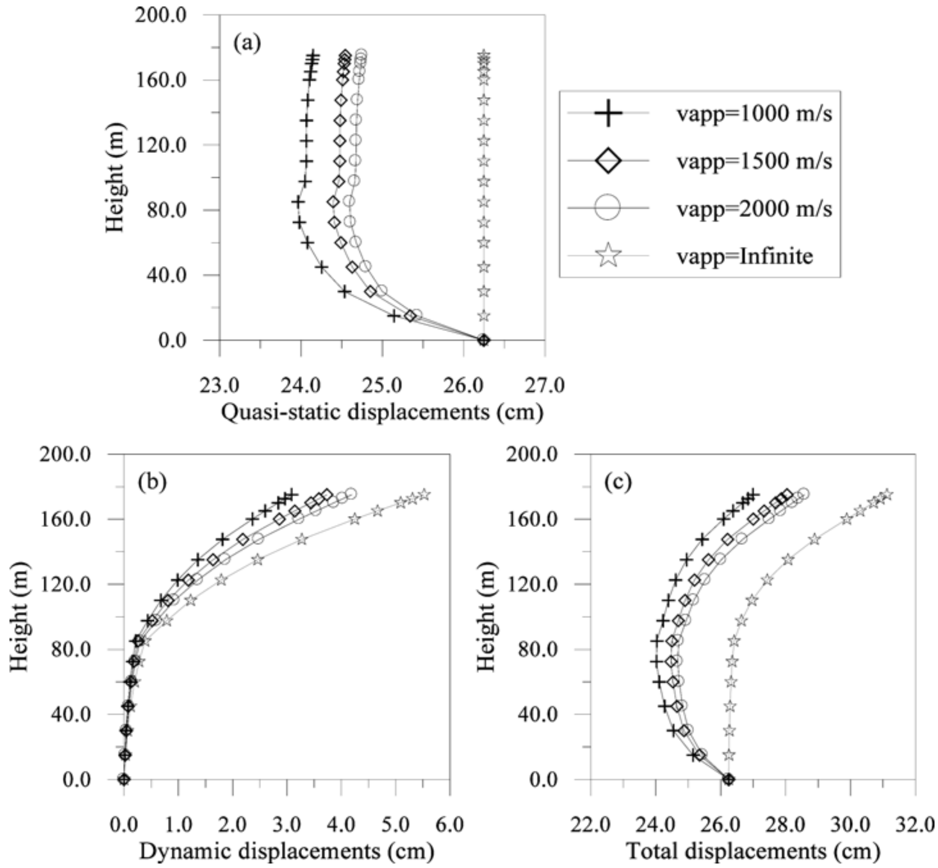

Fig. 3 Horizontal displacements along the upstream of the dam (a) Quasi-static displacements, (b) Dynamic displacements, (c) Total displacements

and total displacements increases significantly with increasing wave velocities.

3.2.2 Stresses and hydrodynamic pressures

The stress components and hydrodynamic pressures are also obtained for four different wave velocities. A vertical section (Section I-I) is taken into account for comparison of the stress components, which are horizontal, vertical and shear stresses. Stress values are obtained at Gauss points of the solid elements near the interface of the reservoir-dam interaction. Section I-I is selected along the upstream face of the dam including foundation shown in Fig. 2(a). The quasi-static, dynamic and total horizontal, vertical and shear stresses throughout the height of the dam are plotted in Figs. 4-6. While the mean of maximum quasi-static and total stress values calculated for wave velocity of 1000 m/s are the biggest, the mean of maximum dynamic stress responses obtained from wave velocity of 1000 m/s are the smallest. The total maximum horizontal stress occurred at the bottom of the dam and these values are bigger 5.60 times for 1000 m/s, 5.30 times for 1500 m/s and 5.20 times for 2000 m/s than the values obtained from infinite wave velocity. Total maximum vertical stress occurred at $h = 110.00$ m from foundation base line ($h = 25.00$ m from bottom of the dam). The increasing ratios according to infinite wave velocity are 0.90, 0.95 and 1.00 for 1000 m/s, 1500 m/s and 2000 m/s wave velocities, respectively. The same maximum values for total shear stress at this point are 6.00, 5.80 and 5.60, respectively.

It can be seen from Figs. 4-6 that the changing of the wave velocities also affect stresses considerably. The stress components calculated from the various wave velocities are generally bigger than those at the uniform ground motion. It is thought that this situation especially comes

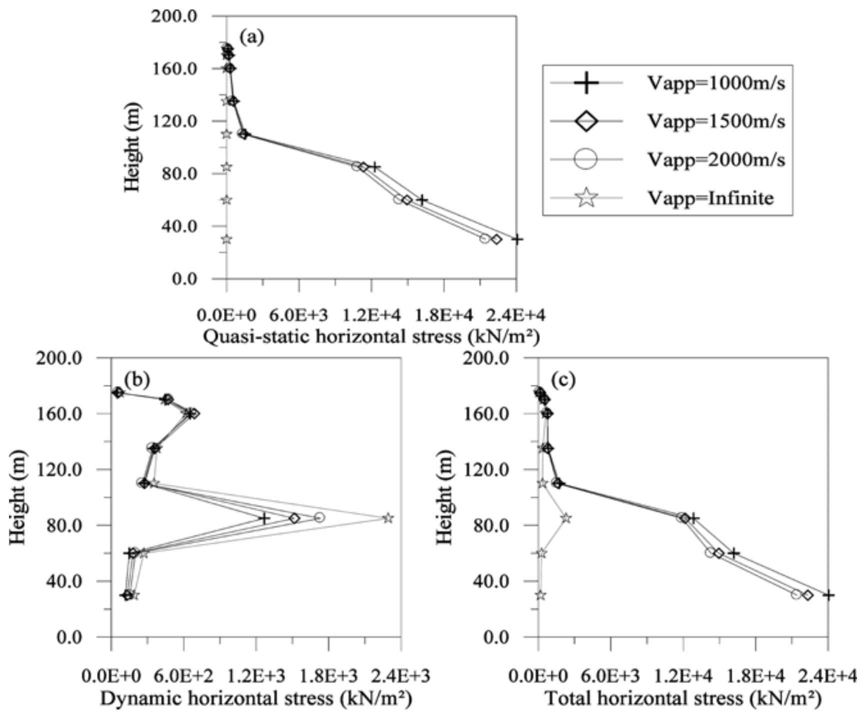

Fig. 4 Horizontal stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

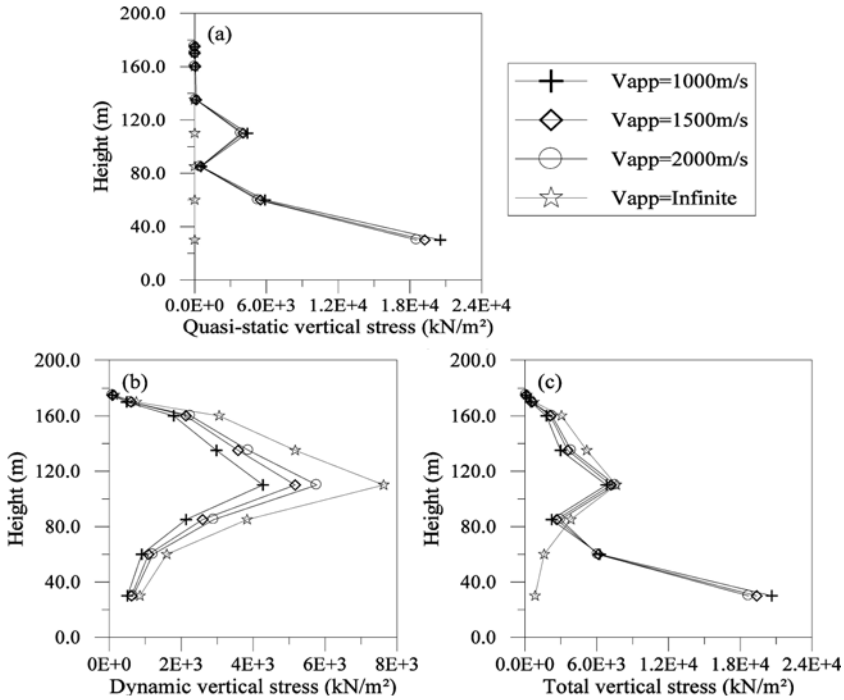

Fig. 5 Vertical stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

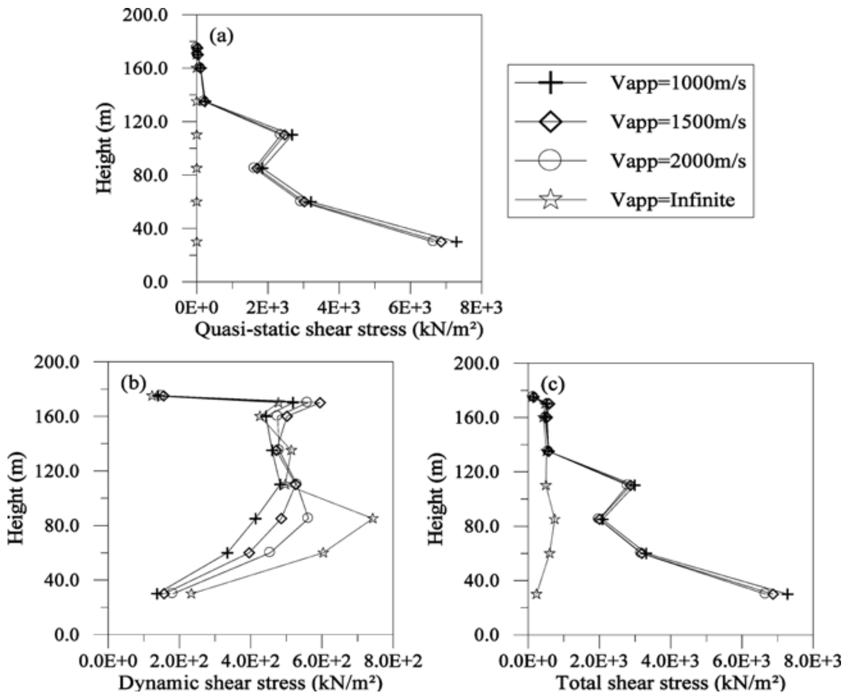

Fig. 6 Shear stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

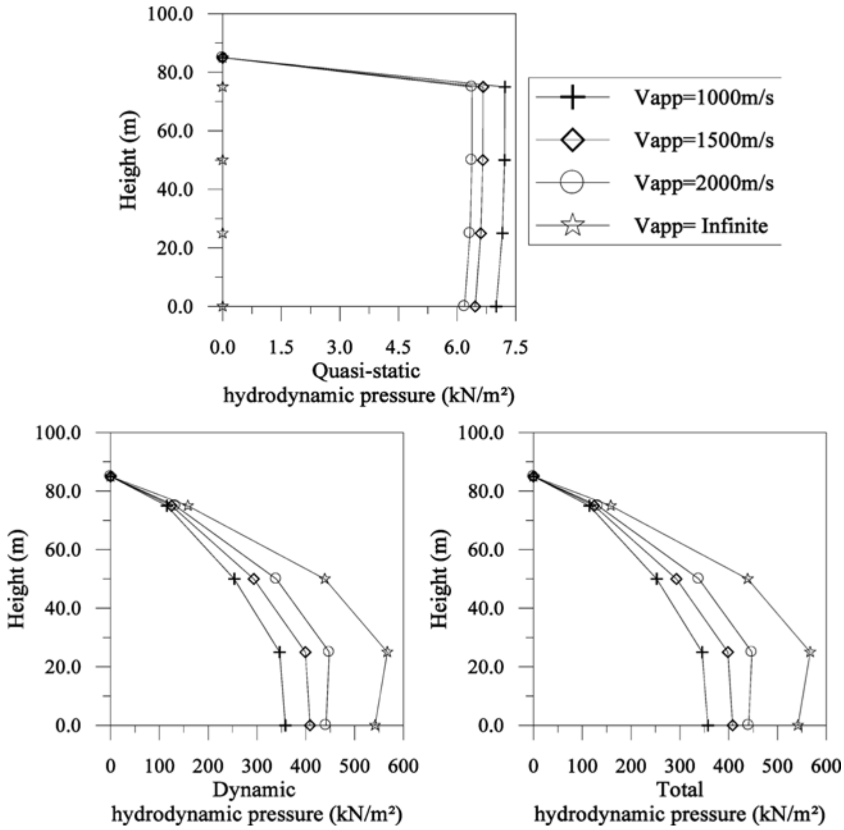

Fig. 7 Hydrodynamic pressure components along the upstream face of the dam (a) For quasi-static response, (b) For dynamic response, (c) For total response

from the quasi-static displacements.

Hydrodynamic pressures occurring on the upstream face of the dam are calculated and compared with 1000 m/s, 1500 m/s, 2000 m/s and infinite velocities. The changing hydrodynamic pressures on the upstream of the dam are plotted in Fig. 7. Hydrodynamic pressures values are obtained at Gauss points of the fluid elements near the interface of the reservoir-dam interaction. The total maximum hydrodynamic pressure occurred at $h = 25.00$ m from bottom of the dam. These values are bigger 0.60 times for 1000 m/s, 0.70 times for 1500 m/s and 0.80 times for 2000 m/s than the values obtained for infinite wave velocity, respectively. It is seen from Fig. 7 that the mean of maximum values of hydrodynamic pressures obtained using the uniform ground motion are bigger than those of the spatially varying earthquake ground motion. This is due to quasi-static displacements. Because, the effects of the quasi-static displacements on the hydrodynamic pressures are very small as shown in Fig. 7(a). Besides, it is known that spatially varying earthquake ground motion decreases the dynamic displacements (Bayraktar and Dumanoğlu 1998, Bayraktar *et al.* 1996). This situation is observed on the total hydrodynamic pressures envelopes clearly as shown in Fig. 7.

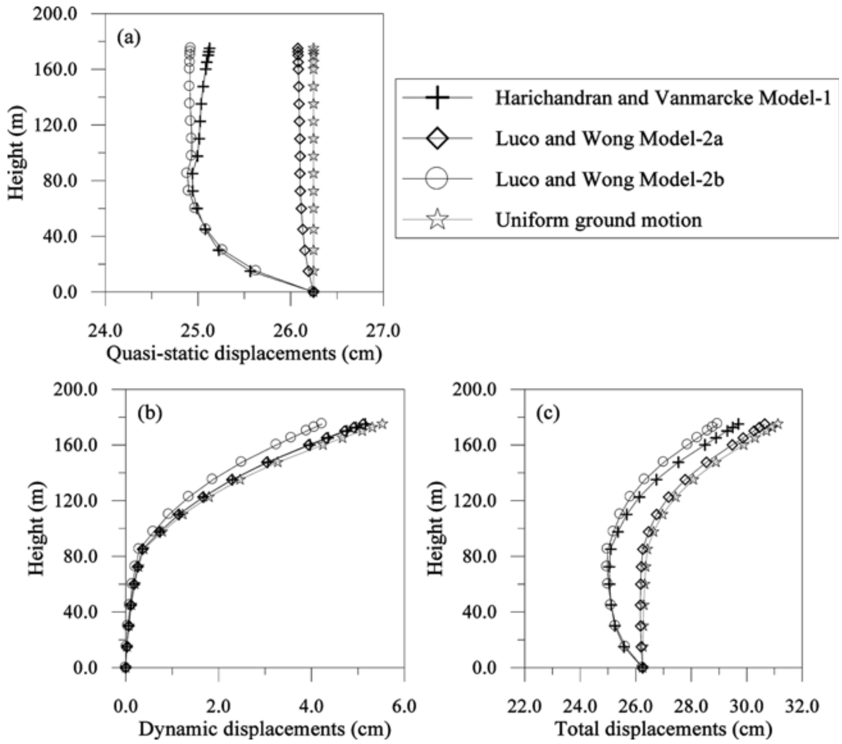

Fig. 8 Horizontal displacements along the upstream of the dam (a) Quasi-static displacements, (b) Dynamic displacements, (c) Total displacements

3.3 Mean of maximum values of response components for Case 2

3.3.1 Displacements

Mean of maximum quasi-static, dynamic and total horizontal displacements on the upstream face of the dam are calculated for three different incoherence function defined above and uniform ground motion and compared in Fig. 8. Of all cases are compared with each other, the displacements obtained for uniform ground motion case has the biggest values. It can be seen from Fig. 8 that the changing of the incoherence function affects displacements considerably. Through these results, it is clearly seen that the different incoherence models alters significantly results of quasi-static, dynamic and total displacements. The results were obtained by using Luco and Wong Model-2a and uniform ground motion are close to each other. In addition to Harichandran and Vanmarcke Model-1, Luco and Wong Model-2b gives very similar results.

3.3.2 Stresses and hydrodynamic pressures

The stress components and hydrodynamic pressures are also obtained from three different incoherence functions and uniform ground motion. The quasi-static, dynamic and total horizontal, vertical and shear stresses throughout the height of the dam vertical section are plotted in Figs. 9-11. The mean of maximum values of quasi-static and total response occur in Model-1 for all stress components. It can be seen from Figs. 9-11 that the changing of incoherence functions affect stresses considerably. The results indicated for total response that Model-1 and Model-2b; and

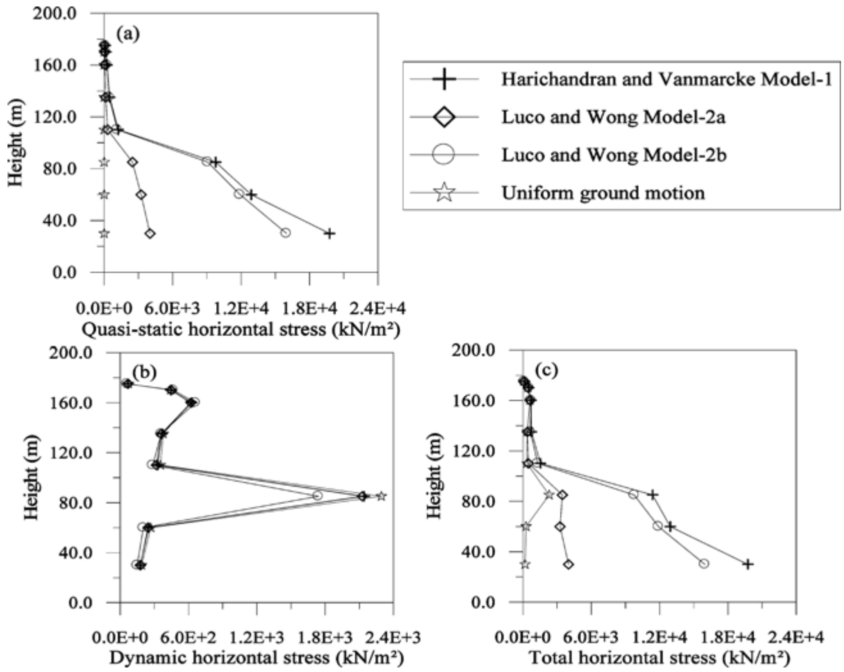

Fig. 9 Horizontal stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

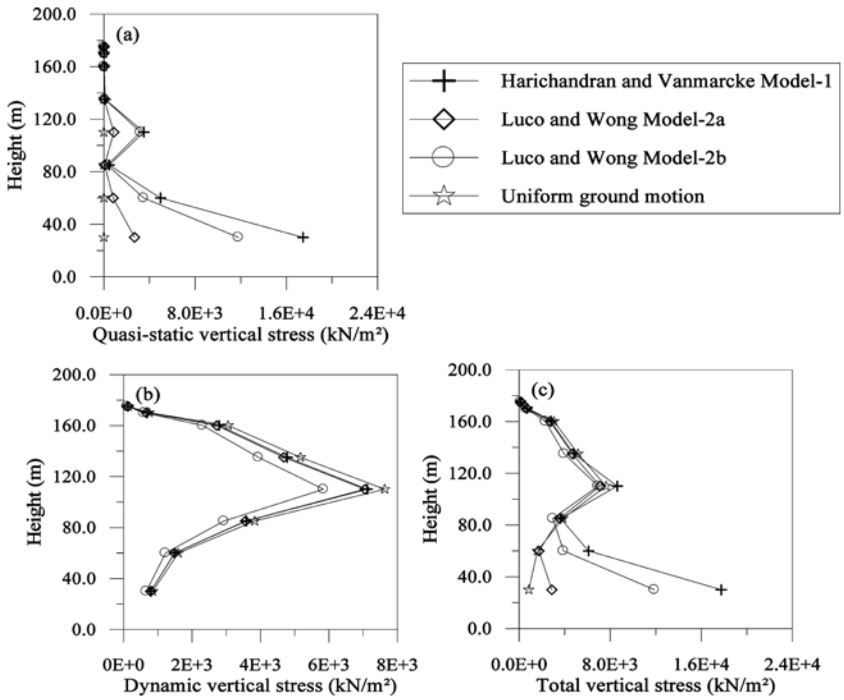

Fig. 10 Vertical stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

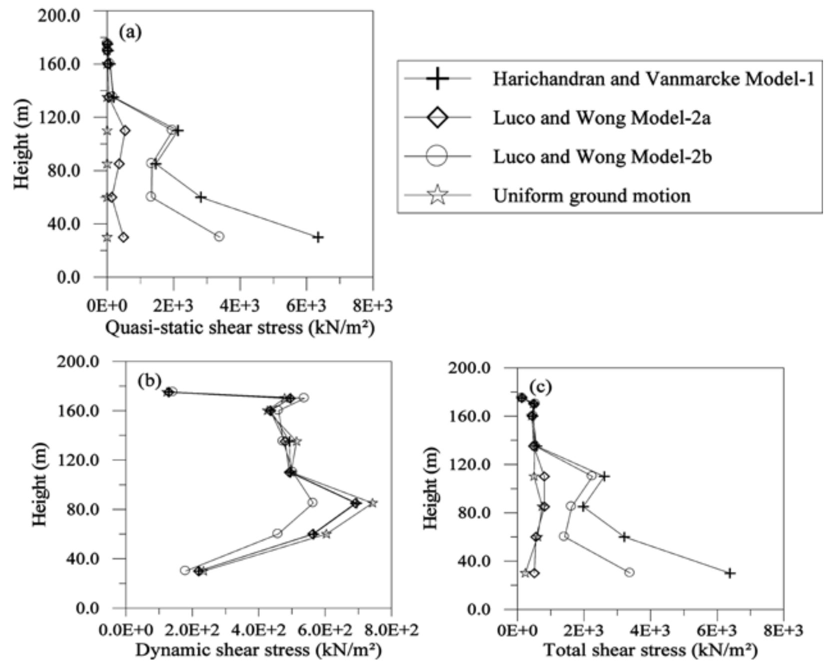

Fig. 11 Shear stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

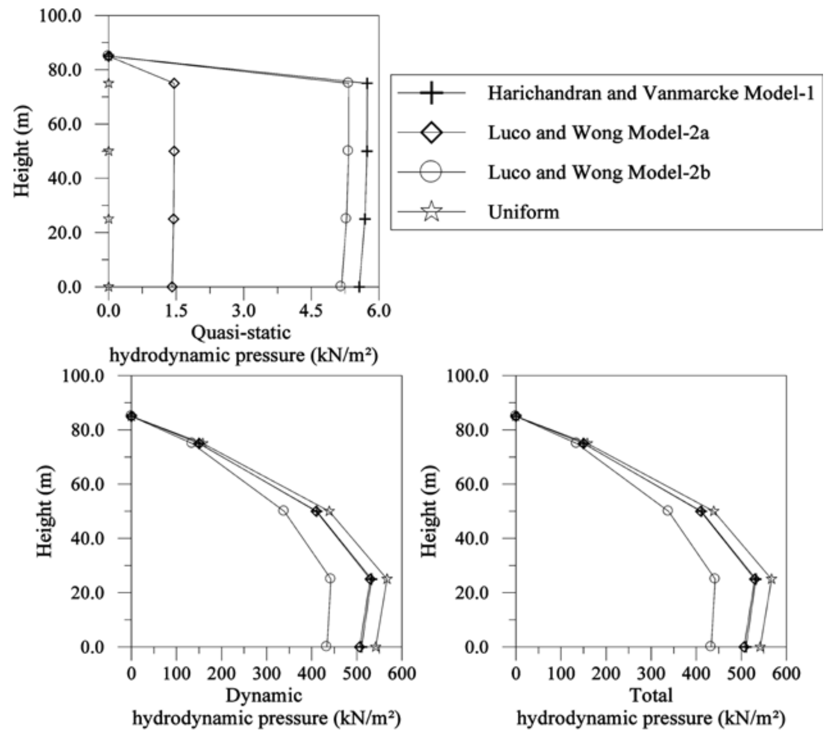

Fig. 12 Hydrodynamic pressure components along the upstream face of the dam (a) For quasi-static response, (b) For dynamic response, (c) For total response

Model-2a and uniform ground motion gives close results.

Hydrodynamic pressures occurring on the upstream face of the dam are calculated and compared with different incoherence models and uniform ground motion. The changing hydrodynamic pressures on the upstream of the dam are plotted in Fig. 12. It is seen from Fig. 12 that the mean of maximum values of hydrodynamic pressures obtained using the uniform ground motion are generally bigger than those of the different incoherence models.

3.4 Mean of maximum values of response components for Case 3

3.4.1 Displacements

Mean of maximum quasi-static, dynamic and total horizontal displacements on the upstream face of the dam are calculated for general excitation (Case 3) and compared with the results obtained from wave passage, incoherence and uniform ground motion in Fig. 13. It can be seen from Fig. 13 that the changing of the components affects displacements considerably and the displacements obtained for general excitation case (Case 3) has the biggest values. This maximum displacement value of general excitation is 1.56 times of uniform ground motion. These ratios are 1.64 and 1.80 for incoherence and wave passage effects, respectively.

3.4.2 Stresses and hydrodynamic pressures

The stress components and hydrodynamic pressures are also obtained from four different

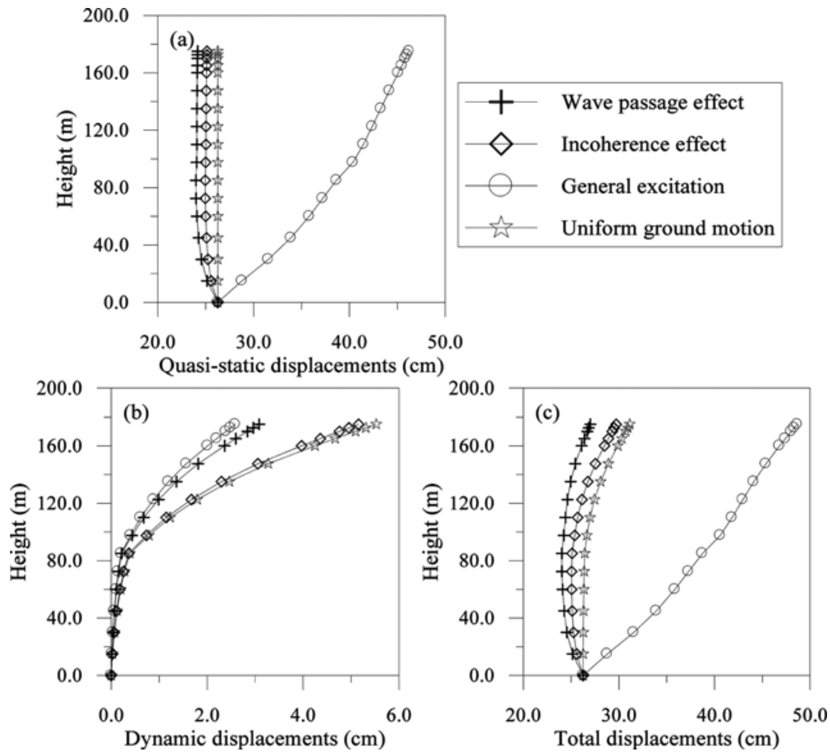

Fig. 13 Horizontal displacements along the upstream of the dam (a) Quasi-static displacements, (b) Dynamic displacements, (c) Total displacements

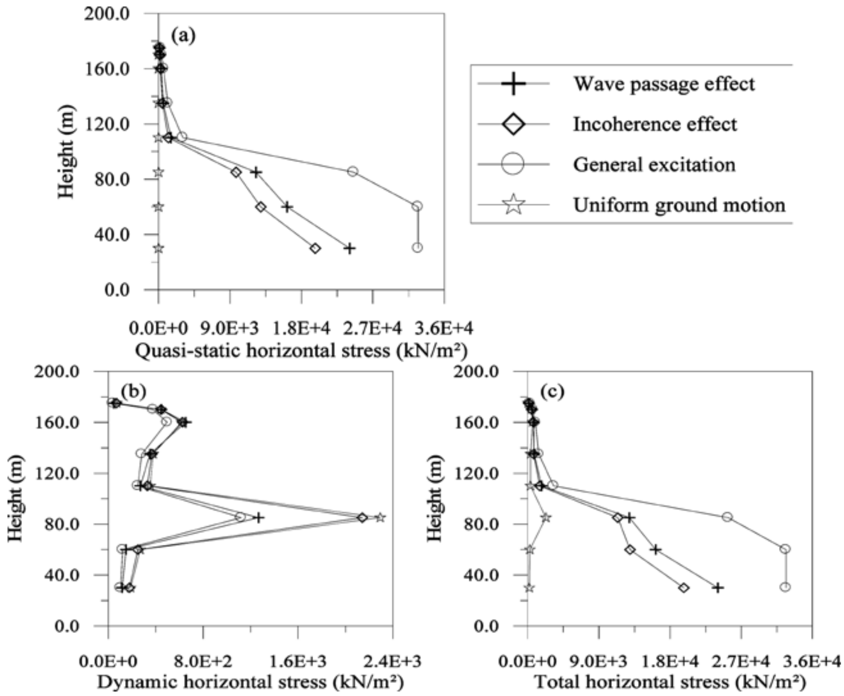

Fig. 14 Horizontal stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

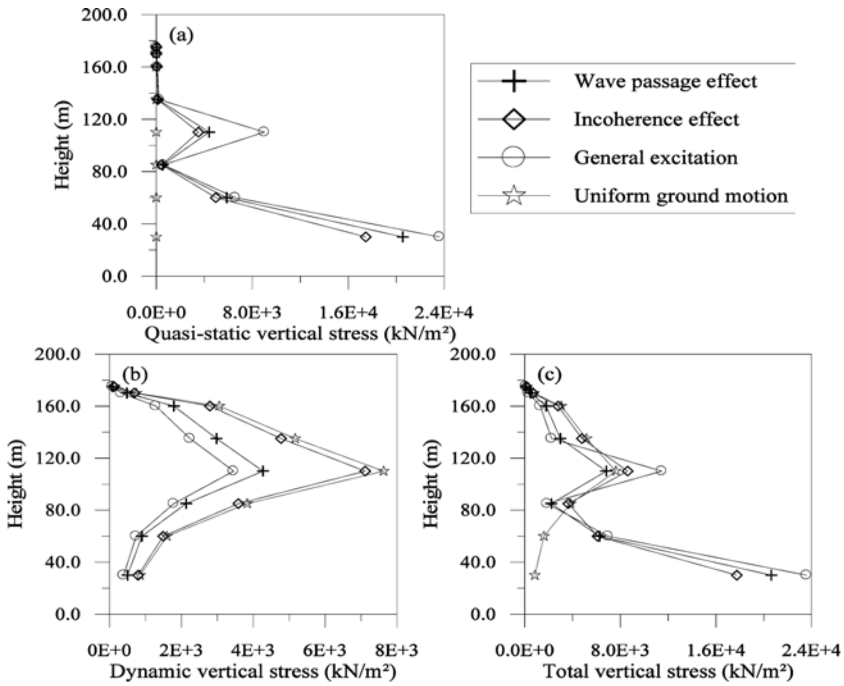

Fig. 15 Vertical stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

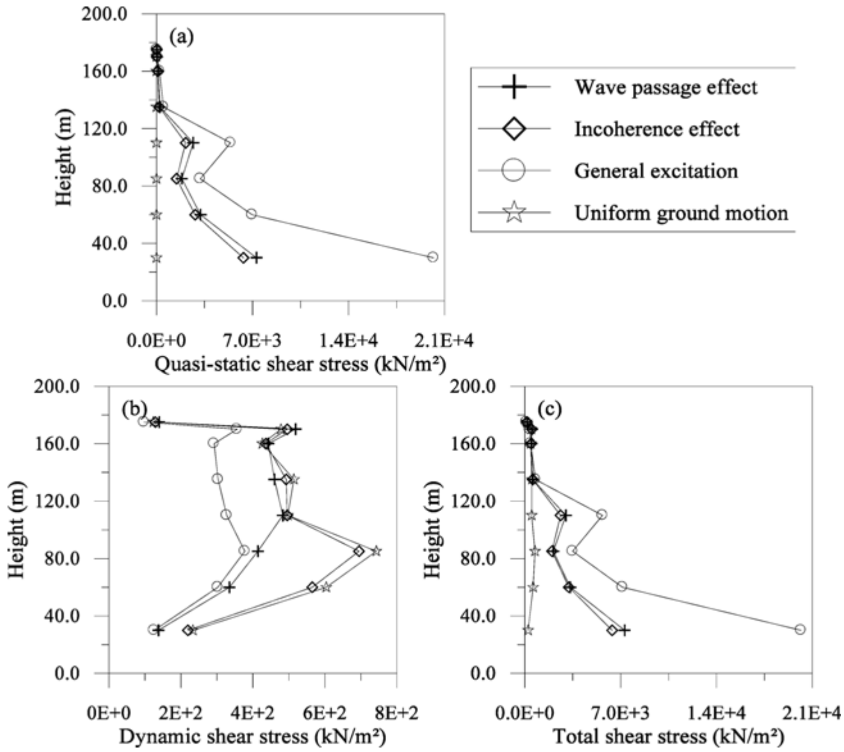

Fig. 16 Shear stress components on section I-I (a) For quasi-static response, (b) For dynamic response, (c) For total response

situations. Section I-I is taken into account for comparison of the stress components, which are horizontal, vertical and shear stresses. The quasi-static, dynamic and total horizontal, vertical and shear stresses throughout the height of the dam-foundation system are plotted in Figs. 14-16. The mean of maximum response values calculated from general excitation are the biggest for quasi-static and total response stress components. In addition to, the response values for the dynamic response obtained from uniform ground motion is the largest. It can be seen from Figs. 14-16 that the changing of parameters which taken into accounts affect stresses considerably. The maximum total horizontal stress occurred at thevbottom of dam upstream face. The maximum value obtained for general excitation is bigger 1.97 times than wave passage effect, 2.23 times than incoherence effect and 11.05 times than uniform ground motion. These ratios are, respectively, 1.68, 1.33 and 1.50 for maximum vertical shear stress. The maximum total shear stress occurred at $h = 25.00$ of the dam upstream face. The maximum shear stress appeared for general excitation. The maximum value of general excitation is approximately bigger 11.00 times than the maximum value of uniform ground motion. These ratios are 1.90 and 2.18 for wave passage effect and incoherence effect according to general excitation.

The changing hydrodynamic pressures on the upstream of the dam are plotted in Fig. 17. The mean of maximum values of hydrodynamic pressures obtained using the uniform ground motion are bigger than those of the other cases. This situation is observed on the hydrodynamic pressures envelopes clearly as shown in Fig. 17. This maximum value is bigger 1.66 times than general excitation, 1.51 times than wave passage effect and 1.06 times than incoherence effect.

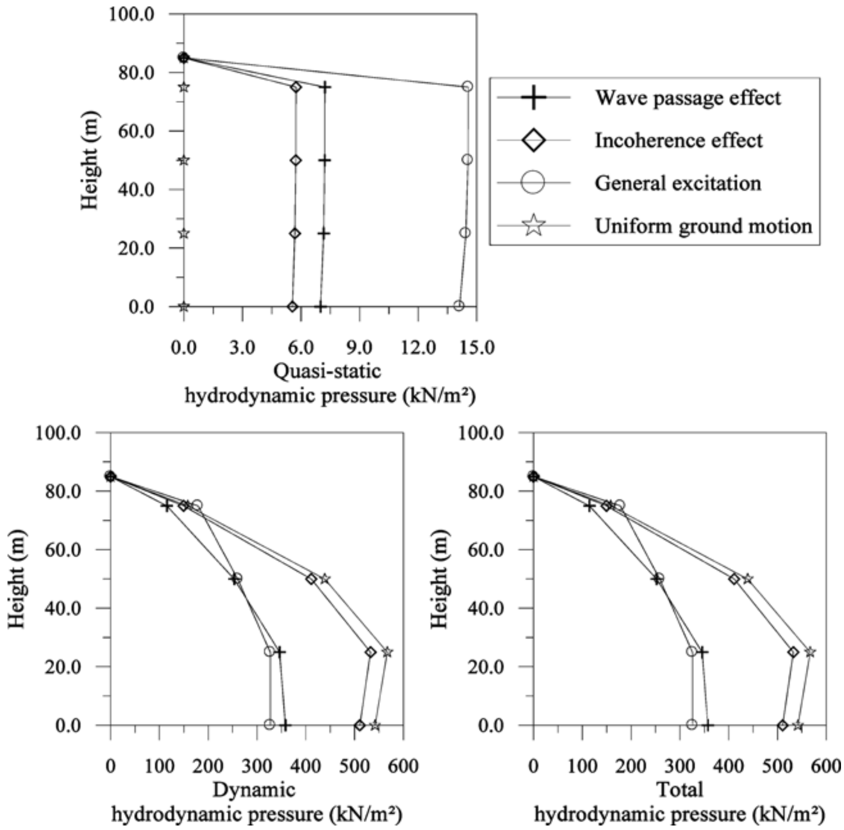

Fig. 17 Hydrodynamic pressure components along the upstream face of the dam (a) For quasi-static response, (b) For dynamic response, (c) For total response

4. Conclusions

In this paper, the stochastic dynamic analysis of fluid-structure interaction systems subjected to spatially varying earthquake motion is performed by using the Lagrangian (displacement-based) fluid and solid finite elements. The developed program SVEMF is used in the stochastic dynamic analysis of the coupled system. Mean of maximum values of displacements, stresses and hydrodynamic pressures are compared with each other for special cases at the support points. The conclusions drawn from this study can be written as:

a. The mean of maximum displacements and hydrodynamic pressures obtained from only wave passages effects (Case 1) increase significantly with increasing wave velocities. The stresses decrease with increasing wave velocities.
b. The displacements and hydrodynamic pressures obtained from Luco and Wong Model-2a and uniform ground motion are fairly close to each other and bigger than those of Harichandran and Vanmarcke-Model-1 and Luco and Wong Model-2b. Stress values obtained for Harichandran and Vanmarcke Model-1 and Luco and Wong Model-2a are close to each other and bigger than those of Luco-Wong Model-2b and uniform ground motion.
c. While mean of maximum quasi-static and total displacements and stresses obtained from

general excitation are the biggest, the dynamic response values calculated from uniform ground motion are the largest.

d. The total response values for displacements and stresses are generally dominated by quasi-static component. The dynamic component has negligible contribution on the displacements and stresses. However, the total response values of the hydrodynamic pressures are dominated by the dynamic component.

e. To be more realistic in calculating the coupled response, the spatially varying earthquake ground motion including wave-passage, incoherence and site-response effects should be incorporated in the analysis.

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