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Influence of wall flexibility on dynamic response of cantilever retaining walls

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Abstract. A seismic evaluation is made of the response to horizontal ground shaking of cantilever retaining walls using the finite element model in three dimensional space whose verification is provided analytically through the modal analysis technique in case of the assumptions of fixed base, complete bonding behavior at the wall-soil interface, and elastic behavior of soil. Thanks to the versatility of the finite element model, the retained medium is then idealized as a uniform, elastoplastic stratum of constant thickness and semi-infinite extent in the horizontal direction considering debonding behavior at the interface in order to perform comprehensive soil-structure interaction (SSI) analyses. The parameters varied include the flexibility of the wall, the properties of the soil medium, and the characteristics of the ground motion. Two different finite element models corresponding with flexible and rigid wall configurations are studied for six different soil types under the effects of two different ground motions. The response quantities examined incorporate the lateral displacements of the wall relative to the moving base and the stresses in the wall in all directions. The results show that the wall flexibility and soil properties have a major effect on seismic behavior of cantilever retaining walls and should be considered in design criteria of cantilever walls. Furthermore, the results of the numerical investigations are expected to be useful for the better understanding and the optimization of seismic design of this particular type of retaining structure.

Keywords: wall flexibility; soil-structure interaction; backfill-wall interaction; dynamic analysis

1. Introduction

Cantilever retaining walls are critical geotechnical engineering structures which have become widespread during the recent decades following the introduction of reinforced concrete construction techniques, especially in connection with the protection of transportation facilities and/or residential areas. For this type of retaining wall, structural weight is not predominant as equilibrium depends mainly on backfill actions and the resistance of foundation soil (Kloukinas *et al.* 2012). Dynamic loads have come to be forefront of attention due to a number of events that affected retaining structures all over the world, clearly indicating that this issue is important for purposes of structural design (Ambrosini and Luccioni 2009). The widespread damage to retaining structures due to earthquakes may have a substantial impact on the economy of the region in terms of both direct and indirect losses. While direct losses result from structural and nonstructural

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infrastructure (Sezen and Whittaker 2006). Therefore, in order to consider the economics of design and also mitigation of damage due to strong earthquakes, knowing the behavior and seismic design of retaining walls is of great importance (Mojallal and Ghanbari 2012). Plenty of research has been carried out concerning the dynamic behavior of retaining walls, and a number of methods of varying degrees of accuracy, efficiency and sophistication have been developed for its evaluation. In spite of multitude of studies that have been performed over the years, the dynamic response of cantilever retaining walls is still not well understood. There is, in particular, a paucity of conclusive information that may be used in design applications (Veletsos and Younan 1997, Younan and Veletsos 2000). Evidence of earthquake-induced damages to retaining structures is widely documented in the literature and still stimulates the interest for solutions capable to embody the effect of seismic loading (Caltabiano *et al.* 2012).

Seismic analysis and design trends of retaining walls reflected in the technical literature can be divided broadly into three main categories based upon the approach and the theory used by the various researchers. These are limit state analyses, elastic analyses, and elasto-plastic and nonlinear analyses. Most of the investigations carried out within these three categories in the past are well known and summarized in various studies (Nazarian and Hadjian 1979, Veletsos and Younan 1994, Theodorakopoulos *et al.* 2001, Gazetas *et al.* 2004; Psarropoulos *et al.* 2005, Madabhushi and Zeng 2007, Giarlelis and Mylonakis 2011, Cakir 2013), and need not be repeated herein.

Recent major advances have been made in state-of-practice procedures for characterizing the seismic behavior of retaining walls. Researchers have developed a variety of analytical and numerical models to predict the dynamic behavior of retaining walls or performed various types of experiments to study the mechanisms behind the development of seismic earth pressures on retaining structures. Some of the analytical, numerical, and experimental works carried out in recent years related to retaining walls will be briefly summarized in the ensuing to reflect the current state-of-technology.

Younan and Veletsos (2000) formulated a method of analysis with which the response to horizontal ground shaking of flexible walls retaining a uniform, linear viscoelastic stratum may be evaluated reliably, and they emphasized that magnitudes of the wall displacements and pressures are quite sensitive to the wall flexibility. Psarropoulos *et al.* (2005) presented finite element model to study seismic earth pressures developed on rigid or flexible retaining walls, and showed that numerical results are in good agreement with available analytical solutions. Nakamura (2006) carried out centrifuge tests on retaining walls and concluded that Mononobe-Okabe theory does not express the real seismic behavior of the retaining wall/backfill system. Mylonakis *et al.* (2007), Evangelista *et al.* (2010) and di Santolo and Evangelista (2011) proposed stress plasticity solutions for evaluating earth pressure coefficients, and made some comparisons with established numerical solutions. Huang *et al.* (2009) used a pseudostatic-based multiwedge method in conjunction with Newmark's sliding block theory to perform displacement analyses on a conventional gravity-type and a cantilever-type model retaining wall. Al Atik and Sitar (2010) conducted two sets of dynamic centrifuge experiments and two-dimensional nonlinear finite element analyses to evaluate the magnitude and distribution of seismically induced lateral earth pressures on cantilever walls with dry medium dense sand backfill. Callisto and Soccodato (2010) studied the seismic behavior of embedded cantilevered retaining walls in a coarse-grained soil by means of a number of numerical analyses, using a nonlinear hysteretic model coupled with a Mohr-Coulomb failure criterion, and they maintained that seismic performance is mostly related to the strength of the soil-wall system. Giarlelis and Mylonakis (2011) investigated the dynamic response of rigid and flexible walls retaining dry cohesionless soil in light of experimental results and analytical elastodynamic and limit analysis solutions, and stated that wall flexibility, which is not taken into account in classical design approaches, should be considered to establish the point of application of seismic thrust on the wall. Shukla and Bathurst (2012) presented an analytical expression for the dynamic active thrust from cohesive soil backfills on rigid retaining walls based on the pseudo-static approach considering tension cracks in the backfill, and stated that obtained equations are useful for the calculation of destabilizing earth forces. Kloukinas *et al.* (2012) performed a series of shaking table tests on scaled models of cantilever retaining walls to explore the dynamic behavior, and concluded that the results were in good agreement with the theoretical models used for analysis. Cakir (2013) proposed a seismic analysis procedure based on the finite element method for evaluation of the effects of earthquake frequency content on the behavior of cantilever retaining walls, and stated that the earthquake frequency content may be one of the most important parameters to be considered in seismic analysis.

The traditional approach for the analysis of cantilever walls is based on the well-known limit equilibrium Mononobe-Okabe type solutions under plane strain conditions, and attention is generally given to the determination of seismic earth pressures. However, notwithstanding the theoretical significance and practical appeal of the Mononobe-Okabe solution, its formulations can be criticized (Mylonakis *et al.* 2007). Furthermore, the latest results and observations clearly run counter to the currently prevailing seismic earth pressure theories and design recommendations (Al Atik and Sitar 2010). Many modern codes, including the Eurocodes (Eurocode-8 2003) and the Italian Building Code (NTC 2008), do not explicitly refer to cantilever walls. The current Greek Seismic Code (EAK 2000 2003), Turkish Earthquake Code (TEC 2007) and Indian Standard Code (IS-1893 2002) address the retaining walls adopting pseudo-static analysis although it does not consider the SSI effects. On the other hand, if we investigate the numerical studies on retaining walls, we can see that almost all of them were carried out based on the two-dimensional modeling. It may be possible to obtain close approximations to the system frequencies, by properly selecting the two-dimensional model. However, two-dimensional modelling of a three-dimensional case can not be recommended for actual engineering applications and two-dimensional models should not be used to solve three-dimensional SSI problems, as clearly emphasized by Luco and Hadjian (1974) and Wolf and Song (2002). Moreover, very few studies concentrated on the effects of SSI and wall flexibility even though the roles of them are of paramount importance, and this also implies that the issue of seismic behavior remains little explored. The above observations provided the initial motivation for the herein-reported work.

In this study, a comprehensive investigation of dynamic behavior of cantilever retaining walls is carried out using the finite element model in three-dimensional space. Two main purposes have been selected for this paper. One of them is to present details of the problem and finite element model of backfill-cantilever wall system under investigation, and to verify the validity of finite element model under fixed-base and elastic soil assumptions through the proposed analytical model. The other is to further investigate the seismic behavior of the cantilever walls considering the effects of SSI and wall flexibility. This study has led to some findings which are presented with the aid of two different wall configurations (flexible and rigid walls) that are analyzed under time history excitations incorporating SSI. One of the major advantages of this study is in considering backfill-structure interaction, subsoil-structure interaction, wall flexibility, elastoplastic behavior of soil, properties of soil and effect of earthquake.

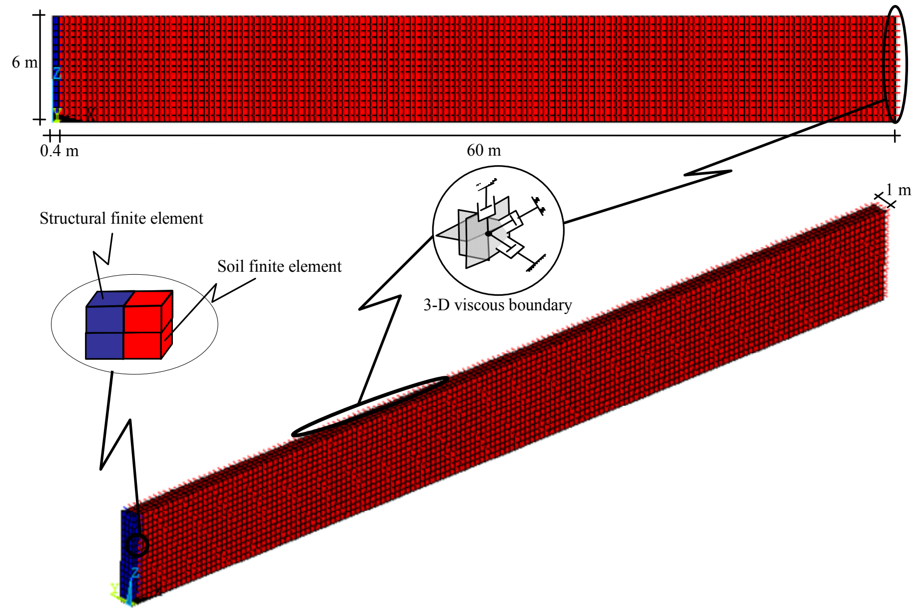

Fig. 1 Finite element model of backfill-cantilever wall system bonded to a rigid base

2. Finite element implementation

At any rate, the key issue in structural and geotechnical design continues to be the choice of an appropriate model, that is able to reproduce faithfully the actual conditions of a earth retaining structure. Concerning the model choice, it seems that a large part of the engineering community has followed a path towards the use of finite element models (Carpinteri *et al.* 2012). In this connection, the proposed finite element model for the problem under fixed-base assumption is depicted in Fig. 1. The problem under investigation consists of a uniform layer of material, that is free at its upper surface, is connected to a rigid base and is retained along one of its vertical boundaries by a uniform cantilever wall that is considered to be fixed at the base and to be free at the top. The heights of the wall and soil stratum are considered to be the same. Furthermore, dry-cohesionless soil is considered in the modelling. Since two-dimensional modelling of a three-dimensional case can not be recommended for actual engineering applications due to potentially dangerous reasons remarked by Luco and Hadjian (1974) and Wolf and Song (2002), three-dimensional modelling of interaction system for a cantilever wall length of 1m is adopted in this study. It should be noted here that the finite element modelling and analyses were carried out by using general purpose structural analysis program, ANSYS (ANSYS 2006).

The cantilever wall itself is discretized by 3-D solid elements (SOLID 65) defined by eight nodes having three translational degrees of freedom in each node in the finite element procedure. The discretization of the soil stratum is made by 3-D structural solid elements (SOLID 185) defined by eight nodes having three degrees of freedom at each node: translations in the nodal x, y, z directions. Regarding the backfill-wall interface, although the option of debonding was available in ANSYS, the assumption of complete bonding -made by both in the study of Veletsos and Younan (1994) and in the analytical model proposed by the author in this study- was also adopted to permit a comparative study at this stage. However, after an analytical verification of the numerical solution, the debonding behavior between the wall and the soil is also considered by using special interface elements when dynamic analysis of the interaction system are carried out in Section 5.

When modeling a dynamic problem including SSI, particular attention must be given to the soil boundary conditions. As the backfill and subsoil are modeled by a finite element grid which will be truncated by artificial grid boundaries, there is a need for using absorbing boundaries in order to minimize the errors related to these artificial grid boundaries and to simulate the radiation of energy away from the structure. The general approach of treating the problem is to divide the infinite medium into the near field (truncated layer), which includes the irregularity as well as the non-homogeneity of the soil adjacent to the structure, and the far field, which is simplified as an isotropic homogeneous elastic medium (Wolf and Song 1996). The finite element methods, being powerful in most engineering applications of normal size, are somewhat restrictive in the geotechnical area due to the large physical dimensions. Even using powerful computers, the modelling is usually very demanding and the lengthy and time consuming procedure of handling all the data can always be a source for errors. As an alternative to modelling very large soil volumes and to limit the model to a reasonable size, special artificial and/or transmitting boundaries must be introduced in the finite element analysis of dynamic SSI problems. This not only avoids unrealistic wave reflections against the artificial boundaries introduced in the mathematical model but also provides the consideration of radiation effects, and thus, the results are not distorted. Several artificial boundaries have been proposed in frequency and time domains in the case of solids. Lysmer and Kuhlemeyer (1969), Kuhlemeyer and Lysmer (1973) suggest applying viscous tractions that must absorb reflected energy along the artificial boundary. This technique is widely used because it is easy to implement and gives satisfactory results for dilatational and shear waves.

In this study, the viscous boundary model, which was successfully employed in the finite element models of liquid tanks performed by Livaoglu and Dogangun (2007), Livaoglu *et al.* (2011), Cakir and Livaoglu (2012), is used in three dimensions to consider radiational effect of the seismic waves through the soil medium. The detailed formulation can be found in the works done by Wilson (2002), Lysmer and Kuhlemeyer (1969). To represent the behavior of the semi-infinite backfill medium, the critical minimum distance from the face of the wall is taken as 10H, a value which is believed to approximate adequately the behavior of the semi-infinite layer (Veletsos and Younan 1994, Psarropoulos *et al.* 2005). In this context, the dashpots were also placed 10H away from the wall in three dimensions to improve the accuracy of the simulation, where H is the height of the cantilever wall.

3. Analytical formulation

The use of analytical models seems to set a reverse trend. Simplified analytical procedures relying on carefully chosen approximations, in fact, can not be renounced to understand the complex behaviour of structures. A simplified model can offer a number of potential advantages: data preparation and analysis is definitely faster; it has low computational requirements, and the modelling procedures are likely to be simpler and more transparent, thus less prone to be a potential source of errors; the accuracy is sufficient for the preliminary design stage. Most importantly, the use of this kind of procedures offers a clear picture of the structural behaviour, allowing to gain insight into the key structural parameters governing the behaviour (Carpinteri *et*

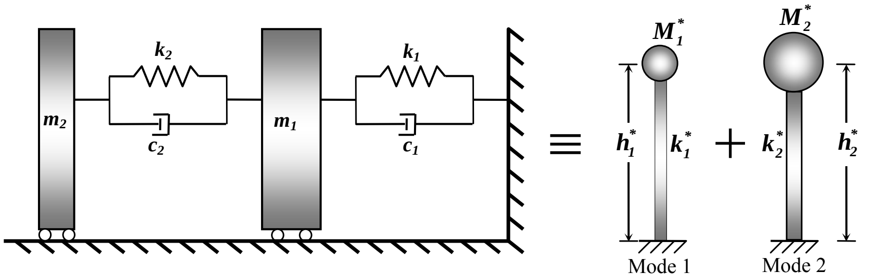

Fig. 2 Proposed simplified analytical model

al. 2012). Furthermore, these models can be used to check the results of rigorous methods such as the boundary element procedure or finite element method. In this connection, a simplified model with constant parameters is introduced in order to demonstrate the accuracy of the finite element model. The model is given in Fig. 2. For the calculation of the stiffness and mass values of backfill soil, the equations presented by Veletsos and Younan (1994) was adopted. Furthermore, the mass of the cantilever wall is taken into account, and the system is represented by a spring-dashpot-mass model with two degrees of freedom in this study while Veletsos and Younan had regarded the wall as massless. To obtain a simplified model and to permit a comparative study, the assumption of complete bonding is adopted, as clearly stated previously.

The coefficients of the springs, dashpots and masses can be determined for varying parameters such as the dimensions, physical and mechanical properties of both the soil and the wall. To define the modal characteristics of the system, the design parameters must be introduced primarily. The mass m_1 refers to soil mass and is equal to

$$m_1 = 0.543\psi_\sigma \rho H^2 \tag{1}$$

where

$$\psi_\sigma = \frac{\psi_0^{\;2}}{\psi_e}; \qquad \psi_0 = \sqrt{\frac{2}{1-\nu}}; \qquad \psi_e = \sqrt{\frac{2-\nu}{1-\nu}} \tag{2}$$

where ρ is the mass density for medium, H is the height of both the wall and the soil stratum, ν is the Poisson's ratio for soil and $\psi_\sigma, \psi_0, \psi_e$ are the functions of ν.

The spring stiffness k_1 for the model with constant parameters is

$$k_1 = m_1 \frac{\pi^2}{4H^2}\frac{G}{\rho} = 1.339\psi_\sigma G \tag{3}$$

where G is the shear modulus of elasticity of soil material.

The mass of the wall is represented by m_2, and the lateral stiffness of the wall, k_2, can easily be determined as $k_2=3EI/H^3$. The parameters of c_1 and c_2 are the damping values for backfill and structure, respectively. Considering the free-body diagrams of both the soil and the wall masses, and the dynamic equilibrium of masses by using D'Alembert's principle, from the Fig. 2, basic dynamic equations can be written in matrix form

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}\begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \begin{bmatrix} c_1+c_2 & -c_2 \\ -c_2 & c_2 \end{bmatrix}\begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix} + \begin{bmatrix} k_1+k_2 & -k_2 \\ -k_2 & k_2 \end{bmatrix}\begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = \begin{Bmatrix} P_1(t) \\ P_2(t) \end{Bmatrix} \tag{4}$$

where (u_1, u_2), ($\dot{u}_1$, $\dot{u}_2$), ($\ddot{u}_1$, $\ddot{u}_2$) are the displacements, velocities and accelerations of masses m_1, m_2, respectively, and $P_1(t)$ and $P_2(t)$ are the external forces. It is worth noting that since the natural frequencies of the system in the modal analysis are determined by using undamped free vibration equation of motions, any data on both the damping matrix and the external forces are not given herein. However, these data will be included in Section 5, where the seismic analysis of the interaction system is performed by means of the finite element model.

The obtained equations can be solved by employing the modal analysis technique. To this end, firstly, the modal properties such as effective modal masses (M_1^*, M_2^*), heights (h_1^*, h_2^*) and stiffnesses (k_1^*, k_2^*) must be determined (see Fig. 2). These modal properties can be estimated by using Eqs. (5) and (6) (Chopra 2007).

$$M_n^* = \Gamma_n L_n^h = \frac{\left(L_n^h\right)^2}{M_n}; \qquad h_n^* = \frac{L_n^\theta}{L_n^h}; \qquad k_n^* = \omega_n^2 M_n^* \tag{5}$$

where

$$M_n = \phi_n^T m \phi_n = \sum_{j=1}^{N} m_j \phi_{jn}^2; \qquad \Gamma_n = \frac{L_n^h}{M_n}; \qquad L_n^h = \sum_{j=1}^{N} m_j \phi_{jn}; \qquad L_n^\theta = \sum_{j=1}^{N} h_j m_j \phi_{jn} \tag{6}$$

where N, ϕ_n and ω_n^2 are the total mode number, the n^{th} mode vector and its eigenvalue, respectively.

4. Modal analysis and model verification

The modal analysis of the systems is done to show the effectiveness of the finite element model in this section. In the numerical example, a 6 m-high cantilever retaining wall with a constant thickness of 0.4 m is considered. As stated before, the critical minimum distance from the face of the wall is taken as 10H=60 m. The Young's modulus, Poisson's ratio and unit weight of the wall are 28000 MPa, 0.2 and 25 kN/m^3, respectively. The Young's Modulus, Poisson's ratio and the unit weight of the soil are taken to be 50 MPa, 0.3 and 18 kN/m^3, respectively. The modal analysis results obtained from the simplified analytical model are summarized in Fig. 3. As can be seen in Fig. 3, the mode frequencies are computed as 2.85 and 4.47 Hz. It should be noted here that the first and second modes represent the modes of backfill and wall, respectively. 25% of the total effective mass is represented by the backfill mode, and 75% of it is represented by the structural mode.

The modal characteristics of the same system can be also determined through the proposed finite element model. Fig. 4 shows the mode shapes of the system. The first three vibration modes, which are capable of representing all system behavior based on the effective modal masses, are identified in this figure. The mode frequencies of 4.45, 4.89 and 5.61 Hz are determined from the finite element model.

A comparison between the analytical and numerical results is seen in Table 1. It should be stated here that only the comparison of the modes related to structure is done since this study is mainly concentrated on the cantilever wall behavior subjected to soil effects in accordance with the aim of the study. Accordingly, if a comparison is carried out for the first structural mode, we

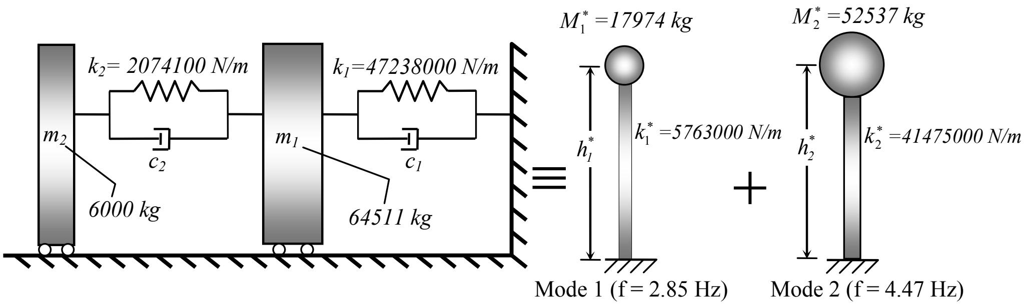

Fig. 3 Modal characteristics obtained from the analytical model

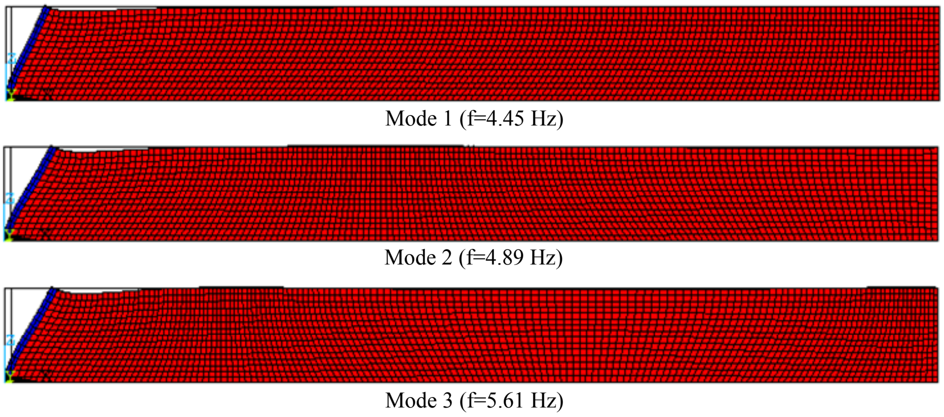

Fig. 4 Mode shapes and frequencies obtained from the finite element model

Table 1 Analytical and numerical results

Mode categories	Mode descriptions	Modal Frequencies (Hz)	
		Analytical Model	Finite Element Model
Backfill	Backfill mode	2.85	---
Structure	First mode	4.47	4.45
	Second mode	---	4.89
	Third mode	---	5.61

can find very good agreement between the numerical and analytical results. The mode frequency is computed as 4.47 Hz from the analytical model while the same quantity is calculated as 4.45 Hz from the finite element model. Actually, this exhibits successful estimation, and the analytical verification provides strong support for the finite element model for use in further investigations.

5. Dynamic analysis

After an analytical verification of the finite element simulation employing the spring-dashpot-mass model, the versatility of the finite element model permits the treatment of some more realistic situations. So the modelling was extended to consider the behavior of wall-soil interface,

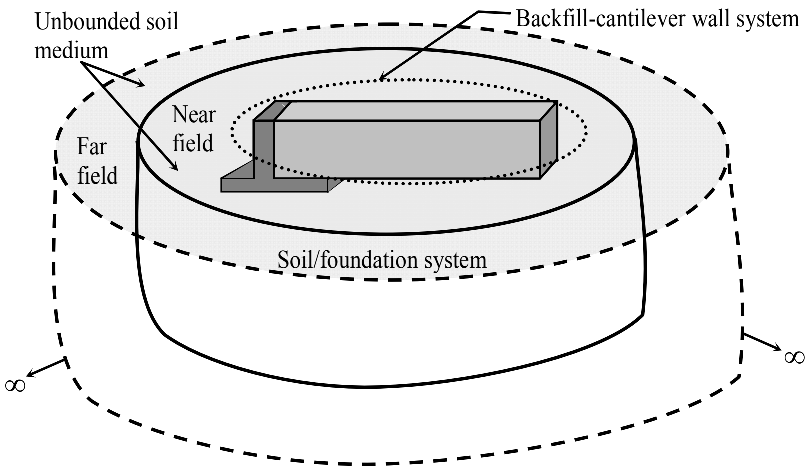

Fig. 5 Dynamic backfill-cantilever wall-soil/foundation interaction problem under investigation

elasto-plastic behavior of soil, and the wall flexibility and soil/foundation interaction effects.

Reasonable modelling of the wall-backfill interface requires using special interface elements between the wall and the adjacent soil to allow for separation. Hence, as a special interface element, nonlinear spring is used between the backfill and the wall allowing for the opening and closing of the gaps (i.e., debonding and bonding) to model backfill-wall interaction in this study. This is a unidirectional element with nonlinear generalized force-deflection capability that can be used in any analysis. The element has longitudinal or torsional capability in 1-D, 2-D, or 3-D applications. The longitudinal option is a uniaxial tension-compression element with up to three degrees of freedom at each node: translations in the nodal x, y, and z directions. The 1-D longitudinal option in the direction of normal to the wall is considered to simulate the behavior of backfill-cantilever wall interaction surface. The soil/foundation system is also modeled with 3-D structural solid elements (SOLID 185) defined by eight nodes with three translational degrees-of-freedom in each node, and the artificial viscous boundaries have been placed in three dimensions on the boundaries of soil/foundation medium. Accordingly, the problem depicted in Fig. 5 reveals a complex phenomenon that includes both the backfill and soil/foundation interaction effects. The proposed finite element model of the backfill-cantilever wall-soil/foundation system is also shown in Fig. 6. Furthermore, the idealization of complex behavior of soil is often necessary to develop simple mathematical constitutive laws for practical applications. Of course other smooth surfaces have been proposed but due to its simplicity, the Drucker-Prager model have gained popularity and are still used even for analysing challenging projects in spite of some limitations of it (Pöttler 1992). The relative simplicity of the Drucker-Prager material model reveals why this model is widely used. Therefore, elasto-plastic behavior of soil is described by the well-known Drucker-Prager yield criteria. The internal friction angle for the cohesionless soil is considered as ϕ=30° in the analyses.

A series of dynamic analyses with variation of parameters such as physical and mechanical properties of soil, wall thickness and ground motion were carried out employing the suggested finite element model. To evaluate the variation of the dynamic response of cantilever retaining walls supported on flexible foundation, six different foundation soil types were considered in the analyses, as shown in Table 2. Moreover, the Young’s Modulus, the Poisson’s ratio, the unit weight and the internal friction angle of cohesionless backfill soil were taken to be 30 MPa, 0.35, 18 kN/m^3 and 30°, respectively. Two different model configurations of cantilever wall associated with both flexible and rigid walls were also analyzed to evaluate the wall flexibility effect. The first one was named as flexible wall having a constant thickness of 0.4 m. The second one was named as rigid wall having a constant thickness of 0.8 m. In addition, in the nonlinear time history analyses, C-OLC360 component of the ground motion recorded at 1604 Oil City station during 1983 Coalinga, and HSP000 component of the ground motion recorded at 47524 Hollister-South & Pine during 1989 Loma Prieta earthquakes (PEER 2012) were used as excitations to exhibit the effect of earthquake. The horizontal peak ground accelerations for these records reach 0.37 g, as shown in Fig. 7. Furthermore, Rayleigh damping was taken into consideration in the seismic analyses. The damping values for both structure and soil were presumed to be 5%.

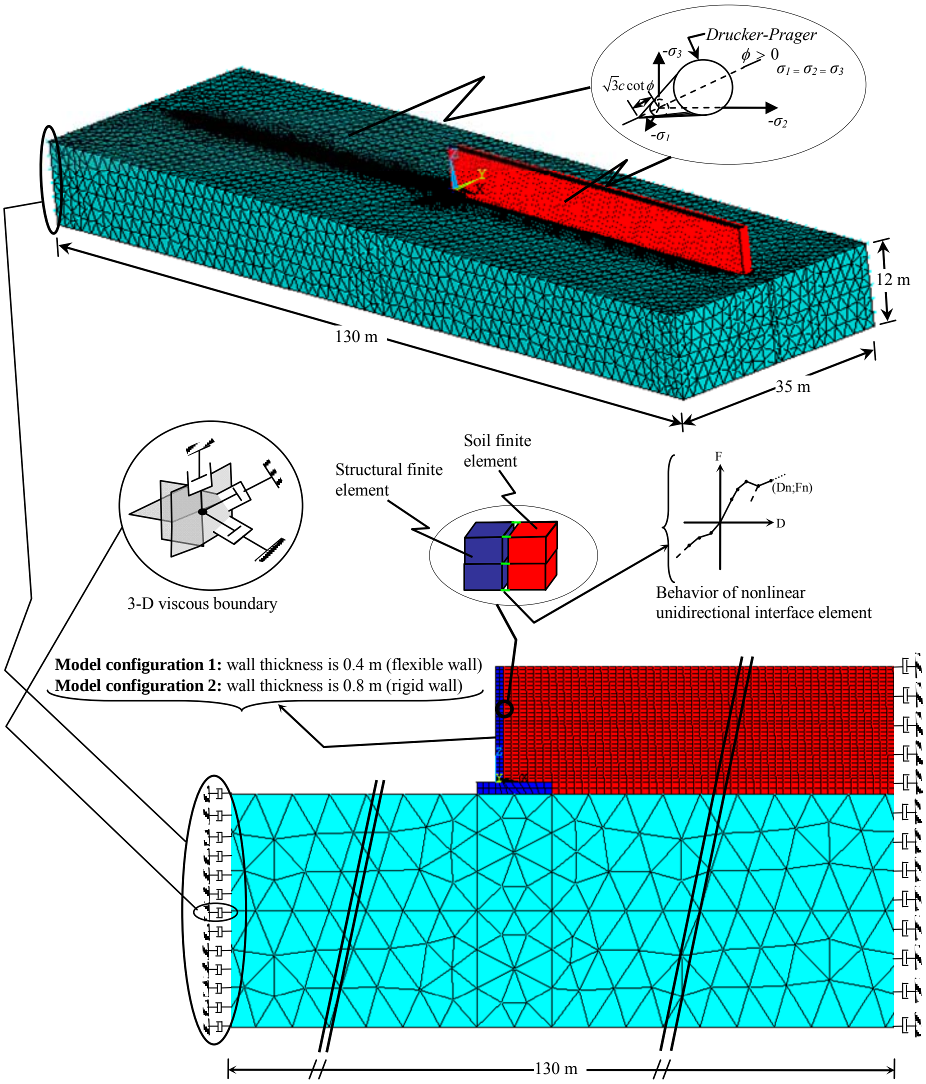

Fig. 6 Finite element model of backfill-cantilever wall-soil/foundation system considered in this study

Table 2 Properties of the considered soil types

Soil Types	E (kN/m^2)	G (kN/m^2)	υ	γ (kg/m^3)	v_s (m/s)	v_p (m/s)
S1	7000000	2692308	0.30	2000	1160.24	2170.61
S2	2000000	769231	0.30	2000	620.17	1160.24
S3	500000	185185	0.35	1900	312.20	649.89
S4	150000	55556	0.35	1900	171.00	355.96
S5	75000	26786	0.40	1800	121.99	298.81
S6	35000	12500	0.40	1800	83.33	204.12

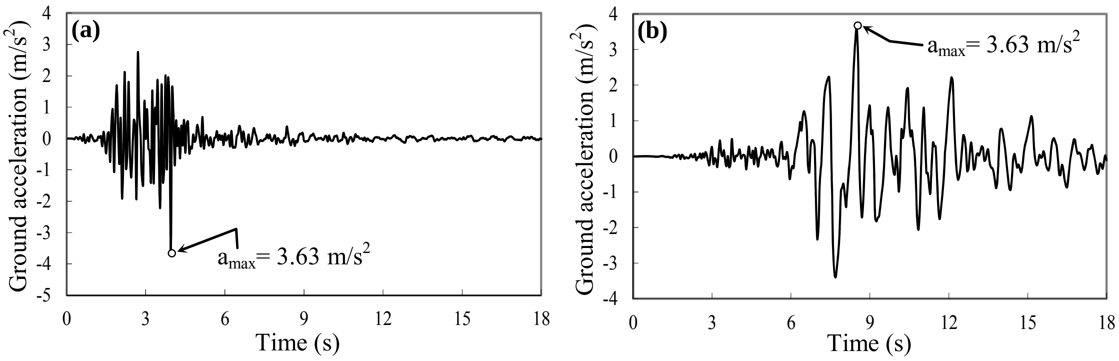

Fig. 7 Considered horizontal components of earthquake records: (a) 1983 Coalinga (b) 1989 Loma Prieta

Table 3 Dynamic analysis results for flexible wall under Coalinga earthquake

Maximum Responses	Soil Types											
	S1		S2		S3		S4		S5		S6	
	t(s)	Value	t(s)	Value	t(s)	Value	t(s)	Value	t(s)	Value	t(s)	Value
u_t (m)	3.9	-0.0002	3.9	0.0002	3.9	0.0014	3.95	0.0045	5.25	0.0078	5.4	0.0132
S_{zb} (MPa)	3.9	0.1728	3.9	-0.1668	2.8	-1.1687	2.8	-2.6230	2.8	-2.7286	2.0	-2.1852
S_{yb} (MPa)	3.9	0.0225	3.9	-0.0221	2.8	-0.1577	2.8	-0.3744	2.8	-0.4041	2.0	-0.3377
S_{xb} (MPa)	3.9	0.0516	3.9	-0.0500	2.8	-0.4176	2.8	-1.0780	2.8	-1.2065	2.0	-1.0772
S_{zf} (MPa)	3.9	-0.1760	3.9	0.1694	2.8	1.1863	2.8	2.6568	2.8	2.7587	2.0	2.2060
S_{yf} (MPa)	3.9	-0.0153	3.9	0.0154	2.8	0.1032	2.8	0.2330	2.8	0.2472	2.0	0.1975
S_{xf} (MPa)	3.9	-0.0258	3.9	0.0242	2.8	0.1410	2.8	0.2601	2.8	0.2455	2.0	0.1471

u_t : Maximum lateral top displacement of cantilever wall; S_{zb}, S_{yb} and S_{xb} : Stresses estimated on the back face (backfill side) of the cantilever wall in *z, y* and *x* directions, respectively; S_{zf}, S_{yf} and S_{xf} : Stresses estimated on the front face of the cantilever wall in *z, y* and *x* directions, respectively.

6. Results and discussions

Computational results, obtained by applying the proposed procedure, are presented in terms of the lateral displacements and stresses in three parts. In the first part, a detailed discussion on the effects of SSI on seismic behavior of cantilever wall is given. In the second part, the effects of different ground motions on dynamic behavior of cantilever wall subjected to the backfill and soil/foundation interactions are discussed. In the third part, the effects of wall flexibility on seismic response of cantilever wall are evaluated. Tables 3-6 summarize the peak responses and the

Table 4 Dynamic analysis results for rigid wall under Coalinga earthquake

Maximum Responses	Soil Types											
	S1		S2		S3		S4		S5		S6	
	t(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value
u_t (m)	3.9	-0.0002	3.9	0.0001	3.9	0.0011	3.95	0.0040	5.25	0.0072	5.35	0.0127
S_{zb} (MPa)	3.9	0.1967	3.9	-0.0660	2.8	-0.7673	2.8	-1.5682	2.8	-1.4926	2.0	-1.0225
S_{yb} (MPa)	3.9	0.0278	3.9	-0.0099	2.8	-0.1278	2.8	-0.2929	2.8	-0.2982	2.0	-0.2250
S_{xb} (MPa)	3.9	0.1099	3.9	-0.0385	2.8	-0.6264	2.8	-1.5347	2.8	-1.5961	2.0	-1.2484
S_{zf} (MPa)	3.9	-0.2015	3.9	0.0670	2.8	0.7545	2.8	1.4750	2.8	1.3674	2.0	0.8981
S_{yf} (MPa)	3.9	-0.0175	3.9	0.0060	2.8	0.0631	2.8	0.1155	2.8	0.1075	2.0	0.0677
S_{xf} (MPa)	3.9	-0.0603	3.9	0.0228	2.8	0.2366	2.8	0.3933	2.8	0.3410	2.0	0.1864

Table 5 Dynamic analysis results for flexible wall under Loma Prieta earthquake

Maximum Responses	Soil Types											
	S1		S2		S3		S4		S5		S6	
	t(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value
u_t (m)	8.2	0.0009	8.2	-0.0010	8.2	-0.0075	8.25	-0.0219	8.3	-0.0343	8.3	-0.0523
S_{zb} (MPa)	8.15	-0.5479	8.15	0.5592	7.9	3.9685	7.9	10.1317	7.9	13.1700	7.9	12.5971
S_{yb} (MPa)	7.85	-0.0676	8.15	0.0673	7.85	0.5199	7.85	1.3618	7.9	1.8017	7.9	1.8326
S_{xb} (MPa)	8.15	-0.1542	8.15	0.1727	7.9	1.4700	7.9	4.1134	7.9	5.6318	7.9	5.9426
S_{zf} (MPa)	8.15	0.5578	8.15	-0.5682	7.9	-4.0262	7.9	-10.2604	7.9	-13.3271	7.9	-12.7250
S_{yf} (MPa)	7.85	0.0479	7.85	-0.0472	7.85	-0.3355	7.85	-0.8235	7.9	-1.0731	7.9	-1.0664
S_{xf} (MPa)	7.85	0.0860	7.85	-0.0837	7.8	-0.4408	7.85	-0.8218	7.85	-0.9809	7.9	-0.8062

Table 6 Dynamic analysis results for rigid wall under Loma Prieta earthquake

Maximum Responses	Soil Types											
	S1		S2		S3		S4		S5		S6	
	t(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value	*t*(s)	Value
u_t (m)	7.9	0.0007	7.9	-0.0003	7.55	0.0042	7.55	0.0166	7.6	0.0285	8.3	-0.0482
S_{zb} (MPa)	7.85	-0.7770	7.85	0.2633	7.85	2.9445	7.85	6.4891	8.6	-7.4118	7.9	5.9347
S_{yb} (MPa)	7.85	-0.1083	7.85	0.0380	7.85	0.4916	7.85	1.1921	8.6	-1.3904	7.9	1.1994
S_{xb} (MPa)	7.9	-0.4313	7.9	0.1650	7.9	2.5869	7.85	6.5431	8.6	-7.8427	7.9	6.8650
S_{zf} (MPa)	7.85	0.8065	7.85	-0.2705	7.85	-2.8506	7.85	-6.0231	8.6	6.7651	7.9	-5.2628
S_{yf} (MPa)	8.55	-0.0735	8.55	0.0253	8.55	0.2327	8.55	0.4203	7.85	-0.4603	7.9	-0.3585
S_{xf} (MPa)	7.85	0.2666	8.55	0.1056	8.55	0.8763	8.55	1.3770	7.85	-1.3830	7.85	-0.9984

corresponding times calculated for varying the soil type, ground motion and wall thickness. The tables clearly indicate the effects of SSI, nature of earthquake, and wall flexibility as not only the magnitude of lateral displacements and stresses but also occurrence times changed significantly. These effects on seismic response of cantilever wall are illustrated, and their implications are comparatively discussed below. It should be noted here that since all results obtained from the analyses can not be illustrated, some comparisons were selected to describe the system behaviour.

6.1 Evaluation of SSI effects on the seismic behavior

The transient lateral displacements and stresses for flexible and rigid cantilever walls due to horizontal excitations are calculated by the proposed method. In this context, the height-wise variations of the lateral displacements of flexible and rigid walls for varying the foundation soil conditions under two different ground motions are exhibited in Figs. 8 and 9, respectively. It is worth noting here that these displacements represent the relative lateral displacements of the wall with respect to the ground. It is observed from these figures that as the soil stiffness decreases, the displacement response generally tends to increase for all conditions, and this reflects a significant SSI influence on the response.

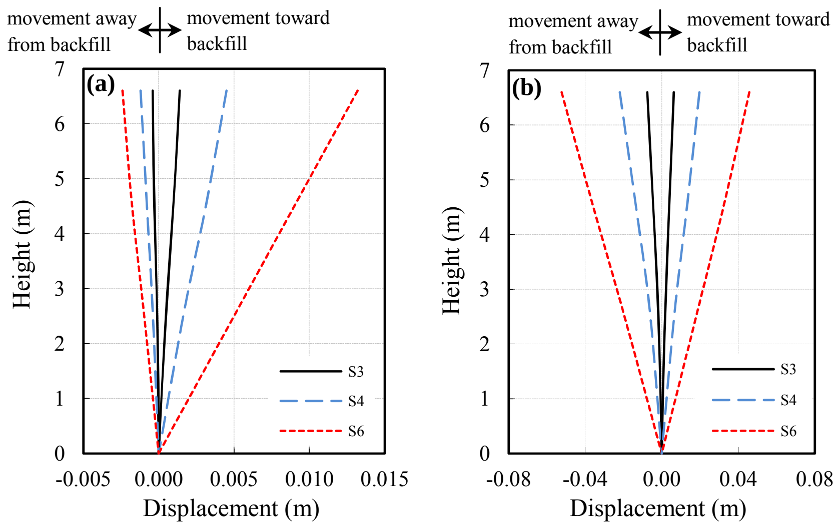

Fig. 8 Lateral displacements along the height of the flexible cantilever wall for (a) Coalinga (b) Loma Prieta earthquakes

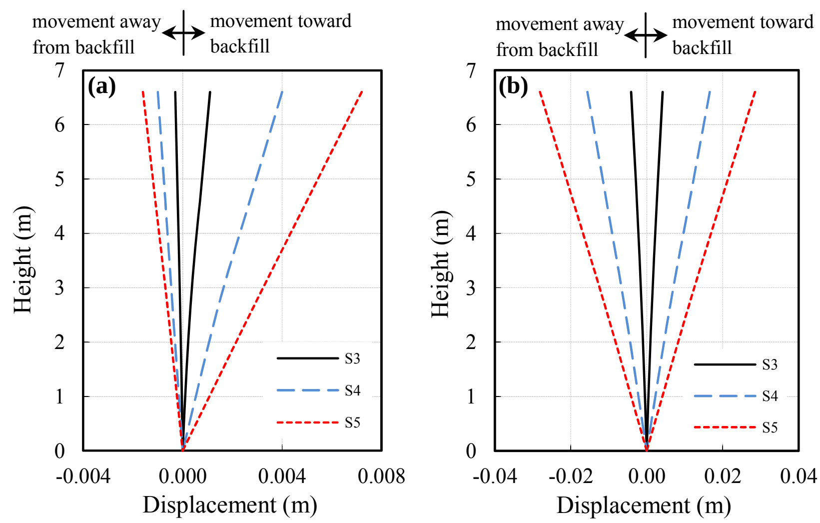

Fig. 9 Lateral displacements along the height of the rigid cantilever wall for (a) Coalinga (b) Loma Prieta earthquakes

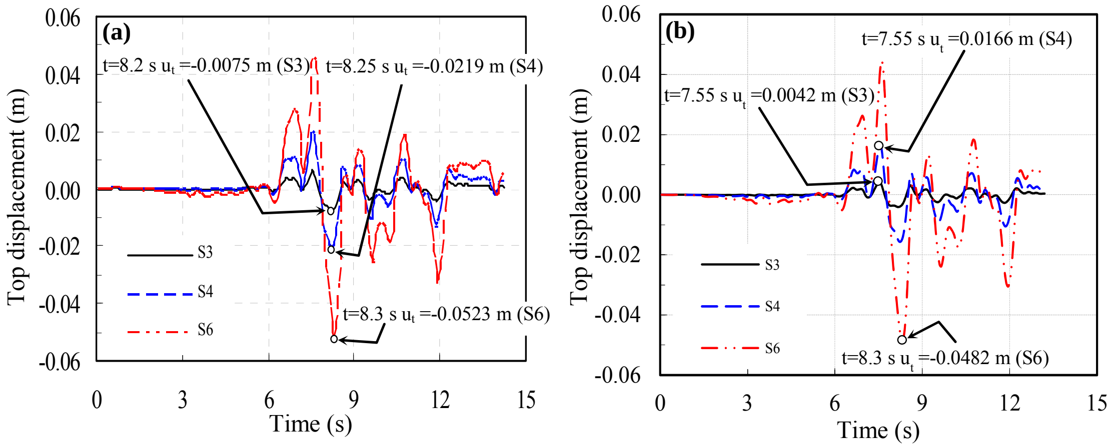

Fig. 10 Time histories of lateral top displacements under Loma Prieta earthquake for (a) flexible wall (b) rigid wall

The time history diagrams of lateral top displacements of both flexible and rigid walls under Loma Prieta earthquake are shown in Fig. 10 in order to clarify the changes of the lateral top displacement values due to flexible foundation conditions. It can be noted from Fig. 10 that the response amplification/reduction has occurred depending on the soil/foundation conditions. For example, while the maximum lateral displacement is estimated as 0.0075 m for S3 soil type, the same quantity is calculated as 0.0523 m for S6 soil type in case of flexible wall. Thus, it can be highlighted that SSI affects the wall behavior so that the dramatic increment in the displacement response is almost at a level of 597% between S3 and S6 soil types. A similar trend is observed for rigid wall as well, the maximum displacement responses due to the SSI are highly magnified, and the responses tend to increase with decreasing soil stiffness. For instance, the value of peak lateral displacement is 0.0042 m for S3 soil type, whereas the displacements are computed as 0.0166 m and 0.0482 m for S4 and S6 soil types, respectively. It is obvious that SSI leads to the dramatic increments in peak displacement responses for S4 and S6 soil types in comparison with S3 soil type, respectively. These variations reveal a significant SSI effect on the response, and confirm that the exclusion of the accurate soil properties may cause underestimation or overestimation of the displacement response, and this, in turn, fairly affects the design process due to the displacement sensitivity of cantilever retaining walls.

The computed stress responses and their variations in time at the back and front faces of the cantilever retaining wall can also be compared to introduce the SSI effects. Under the Coalinga and Loma Prieta earthquakes, the comparisons of stress time history responses in *z* direction at the back face of the flexible cantilever wall are shown in Fig. 11. As this figure depicts, the maximum stresses obtained at the critical sections of the wall change with varying soil conditions. For example, under the Coalinga earthquake, while the peak stress, as compression, has the value of 1.1687 MPa for S3 soil type, it is calculated as 2.7286 MPa for S5 soil type. This reflects a stress increment of about 133% between S3 and S5 soil types due to the variation of soil conditions. A similar trend can be observed for Loma Prieta earthquake as well, the maximum stress responses due to the SSI are highly magnified. For instance, the value of peak stress is 3.9685 MPa for S3 soil type, whereas the same quantity is calculated as 13.1700 MPa for S5 soil type, and a stress increment of nearly 232% takes place at the back face of the flexible wall. Furthermore, it is

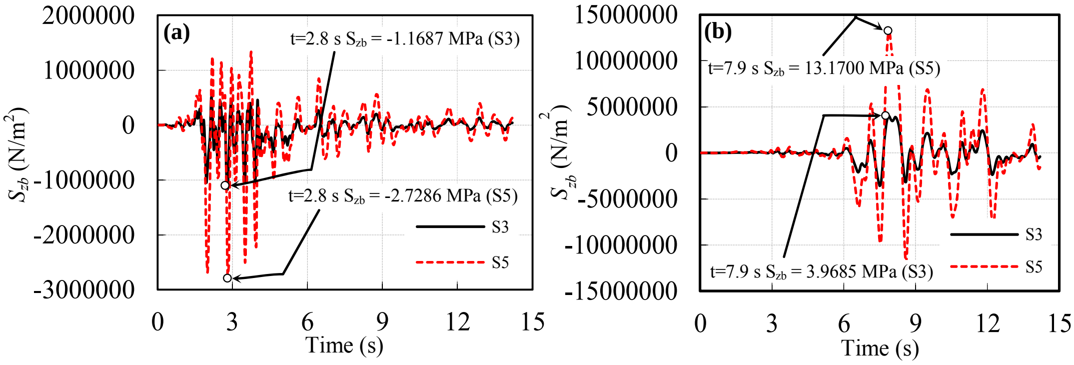

Fig. 11 Variation of stresses in time in *z* direction at the back face of the flexible cantilever wall for (a) Coalinga (b) Loma Prieta earthquakes

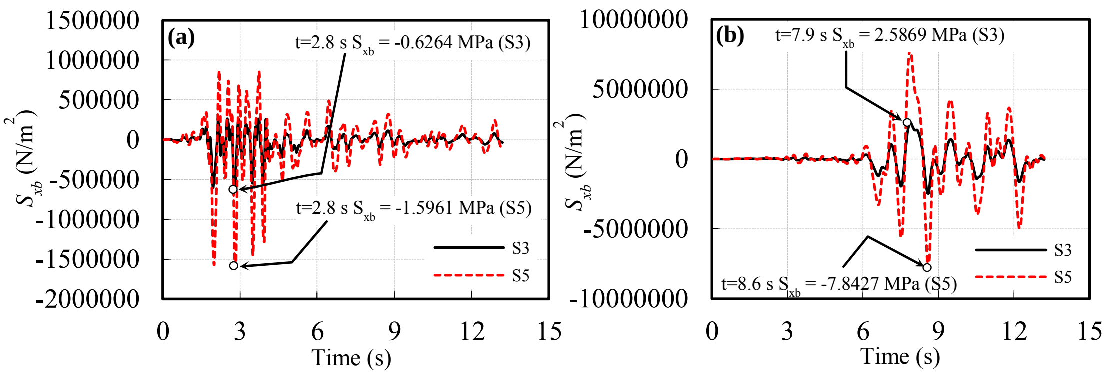

Fig. 12 Variation of stresses in time in x direction at the back face of the rigid cantilever wall for (a) Coalinga (b) Loma Prieta earthquakes

important to state here that the peak responses of stresses in the wall in *z* direction take place at the level of 0.2 m from the top of the foundation. If similar comparisons are made in *x* direction for rigid wall, as seen in Fig. 12, the same trend and SSI effects can be clearly observed. For example, the changing of soil type from S3 to S5 causes a stress increment of about 203% at the back face of the rigid wall for Loma Prieta earthquake. The most important point arising from these comparisons is that the variation of the soil properties notably affects the stress response of the system. This implies that the response amplification or reduction pattern due to the deformable foundation is highly dependent on the soil properties, and the time history diagrams describe different behaviors of the structure. Therefore, these evaluations should be considered as an alert that especially the mechanical properties of soil are of paramount importance, and thus should be measured with utmost care.

6.2 Evaluation of different ground motion effects on the seismic behavior

As mentioned before, to assess the effects of earthquake record on structural behavior, two different ground motions are used in the analyses. Accordingly, Tables 3-6 indicate that effect of earthquake characteristics is fairly significant on the structural response of the wall so that the peak

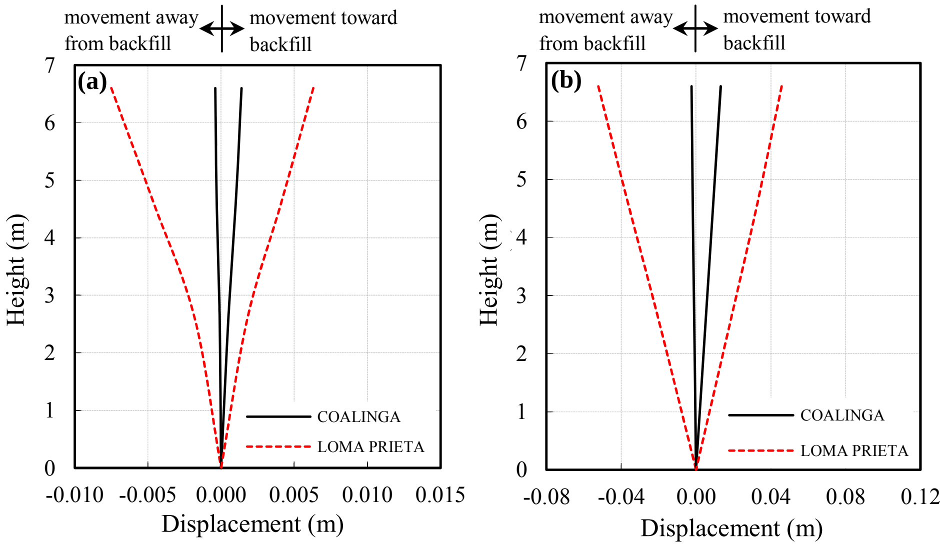

Fig. 13 Comparison of lateral displacements of the flexible wall under two different ground motions for (a) S3 (b) S6 soil types

responses are different from each other depending on the variation of the ground motion. Another sign of this influence is the differences in occurrence times of responses. Furthermore, maximum responses are smaller for Coalinga earthquake in comparison to the Loma Prieta earthquake, and this is valid for all soil types. At this point, this can be attributed to the magnitudes of the considered earthquakes since the Coalinga earthquake had a magnitude of 5.2 and the Loma Prieta earthquake had a magnitude of 6.9. The effects of earthquake on seismic response of the cantilever wall are illustrated, and their implications are also discussed comprehensively below.

A comparison among the height-wise variations of lateral displacements of flexible cantilever wall for S3 and S6 soil types is presented in Fig. 13. Similarly, a comparison for rigid cantilever wall for S4 and S5 soil types is given in Fig. 14. It should be stated once again that these displacements represent the lateral displacements of the wall related to the ground. Effects of nature of earthquake on the displacement response of the wall are clearly observed from these figures, and it is obvious that the structural response is highly dependent on the earthquake characteristics so that a considerable increase occurs in displacement response. The results show that the responses due to Loma Prieta earthquake are highly magnified, and the smaller responses are obtained under the Coalinga earthquake.

To clarify the changes of the lateral displacement due to different earthquake records, the deviations of the displacements in time are illustrated and compared for S4 and S5 soil types for rigid wall in Fig. 15. As can be seen in Fig. 15 for S4 soil type, while the maximum lateral top displacement is calculated as 0.0040 m at 3.95 s for Coalinga earthquake, the same quantity is computed as 0.0166 m at 7.55 s for Loma Prieta earthquake. Hence, it can be noted that input earthquake motion affects the system behavior so that the increment in the displacement response is almost at a level of 315%. If similar comparison is made for S5 soil type, a similar trend of an increase in the response can be clearly seen. For instance, the value of maximum lateral displacement is 0.0072 m for Coalinga earthquake, while the displacement is estimated as 0.0285 m for Loma Prieta earthquake. It is clear that the variation of the ground motion leads to the

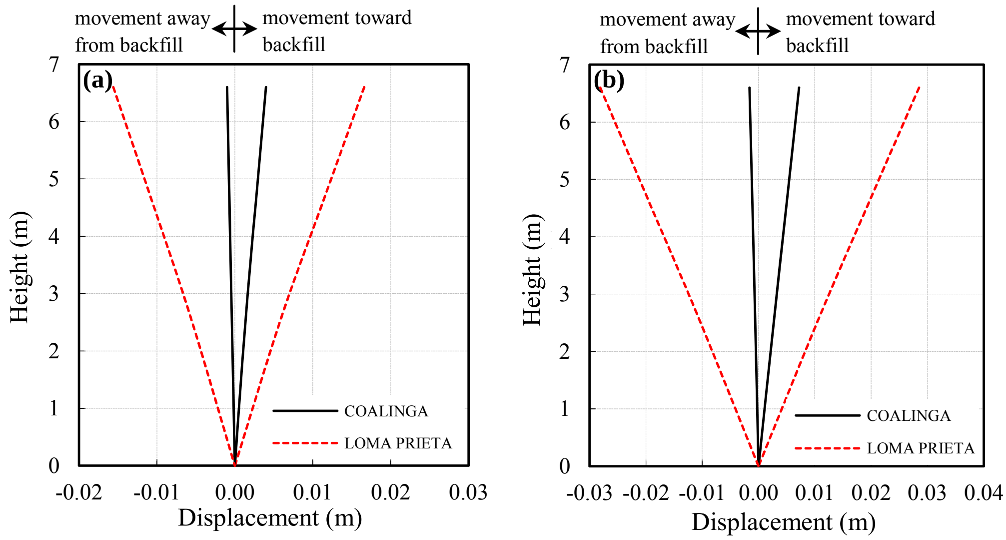

Fig. 14 Comparison of lateral displacements of the rigid wall under two different ground motions for (a) S4 (b) S5 soil types

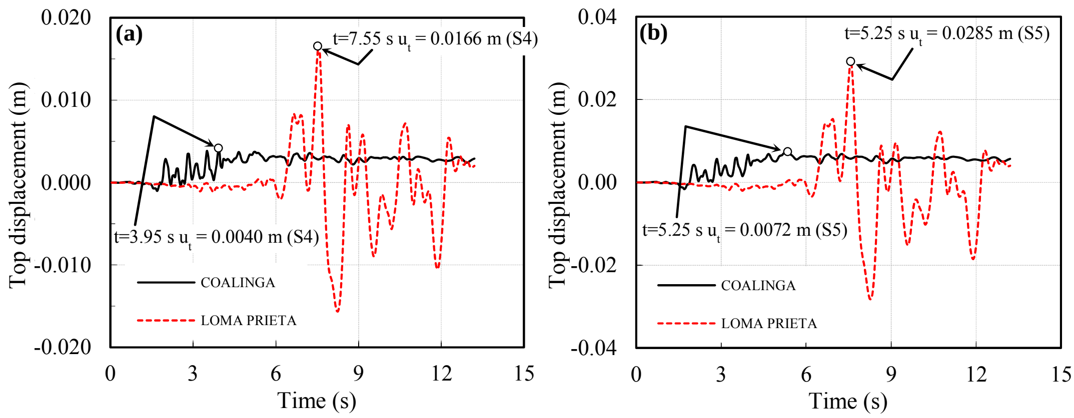

Fig. 15 Variation of lateral displacements of the rigid wall in time under two different ground motions for (a) S4 (b) S5 soil types

dramatic increment of about 296% in peak displacement response for Loma Prieta earthquake compared to the Coalinga record. It is found that the effect of nature of earthquake is quite significant on the displacement response, and may cause a considerable increase in time domain peak response values.

In addition to the lateral displacement response, stress response of the cantilever wall is investigated in this section. The time history diagrams of stress responses at the front face of the rigid cantilever retaining wall in *z* direction for S3 and S5 soil types are presented in Fig. 16, depending on the two different ground motions. As depicted in Fig. 16 for S3 soil type, while the maximum stress, as tension, has the value of 0.7545 MPa for Coalinga record, its value is 2.8506 MPa for Loma Prieta earthquake, as compression. This reflects an increase of about 278% in stress value due to the variation of the ground motion. The same tendency can be observed for S5 soil

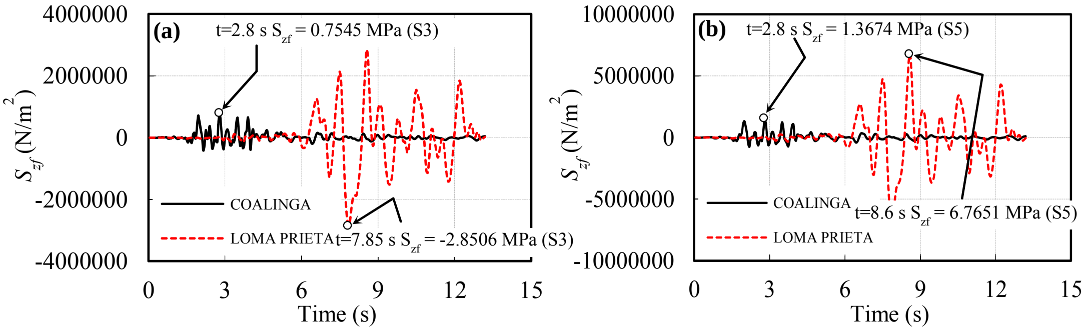

Fig. 16 Variation of stresses in *z* direction at the front face of the rigid wall under two different ground motions for (a) S3 (b) S5 soil types

type. For instance, the value of peak stress is 1.3674 MPa under the Coalinga record, whereas the same quantity is calculated as 6.7651 MPa under the Loma Prieta earthquake, and a dramatic stress increment of approximately 395% occurs.

It is clear that the response amplification or reduction pattern due to deformable foundation is highly dependent on the nature of the earthquake.

6.3 Evaluation of wall flexibility effects on the seismic behavior

Actually, the data presented so far clearly show the effects of wall flexibility on dynamic response of cantilever retaining walls. Accordingly, Tables 3-6 indicate that wall flexibility effect on the structural response of the wall is significant so that the peak responses are different from each other depending on the variation of wall thickness. The lateral displacements along the heights of the flexible and rigid cantilever walls under Loma Prieta earthquake for S2 and S3 soil types are displayed in Fig. 17. It is clearly observed from this figure that as the wall thickness decreases, the displacement response increases, and this reflects a significant wall flexibility influence on the response.

A comparison between flexible and rigid walls under Loma Prieta earthquake for S2 and S3 soil types based on the time history diagrams of lateral top displacements are shown in Fig. 18 in order to clarify the changes of the lateral top displacement values due to wall flexibility. It can be stated from Fig. 18 that the response amplification/reduction has taken place depending on the wall flexibility. For example, while the maximum lateral displacement is computed as 0.0003 m for rigid wall, the same quantity is calculated as 0.0010 m for flexible wall under S2 soil type. Hence, it can be maintained that the flexibility affects the wall behavior so that the increment in the displacement response is almost at a level of 234% between flexible and rigid walls. Another indication of the effect of wall flexibility is observed for S3 soil type as well, the maximum displacement response due to the wall flexibility is magnified. For instance, the value of peak lateral displacement is 0.0042 m for rigid wall, whereas the displacement is estimated as 0.0075 m for flexible wall. It is obvious that the wall flexibility leads to the dramatic increments in peak displacement response. Similar comparisons can be performed from Tables 3-6. All these variations reveal a significant wall flexibility effect on the response. In this connection, the inclusion of the effect of wall flexibility is requisite since it may considerably affect the design process due to the displacement sensitivity of cantilever retaining walls.

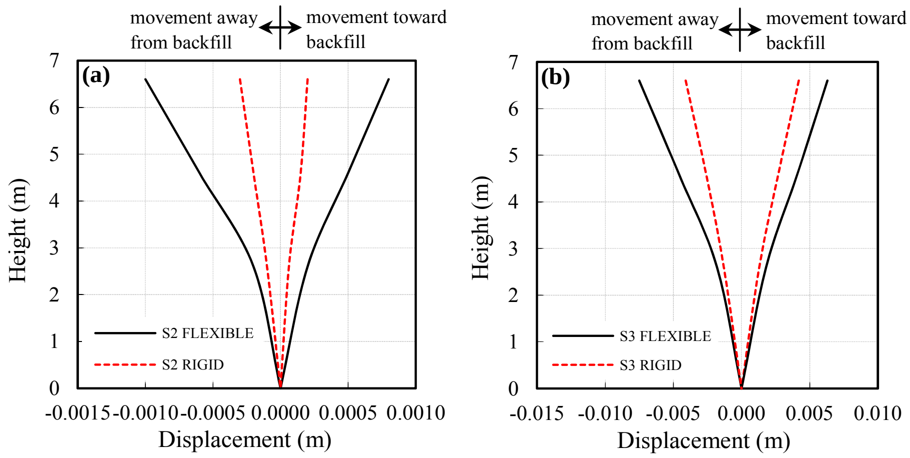

Fig. 17 Lateral displacements along the heights of the flexible and rigid cantilever walls under Loma Prieta earthquake for (a) S2 (b) S3 soil types

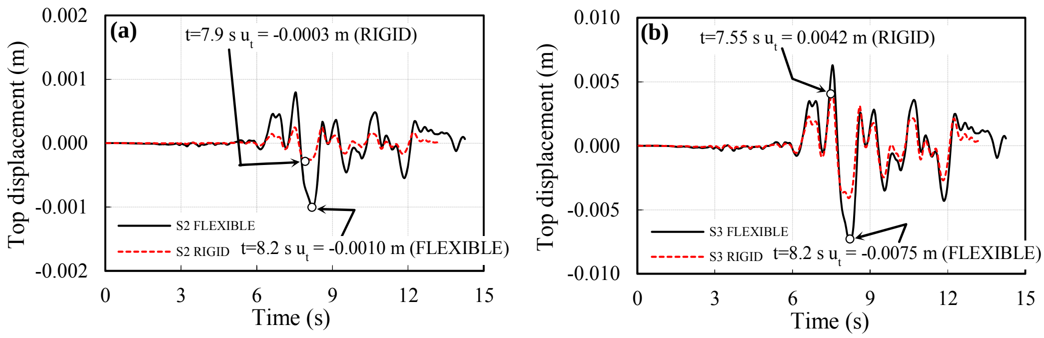

Fig. 18 Time histories of lateral top displacements of flexible and rigid walls under Loma Prieta earthquake for (a) S2 (b) S3 soil types

As might be anticipated, the flexibility of the wall also affects significantly the stress responses. The time history diagrams of stress responses can also be compared to exhibit the wall flexibility effects. Under Loma Prieta earthquake, the comparisons of stress time history responses in z direction at the back face of the flexible and rigid cantilever walls for S3 and S5 soil types are displayed in Fig. 19. As this figure depicts, the maximum stresses change with varying wall thickness. For example, for S3 soil type, while the peak stress has the value of 2.9445 MPa for rigid wall, it is calculated as 3.9685 MPa for flexible wall. This reflects a stress increment of about 35% between flexible and rigid walls due to the variation of wall thickness. The same tendency can also be observed for S5 soil type from Fig. 19. If similar comparisons are made in x direction from Fig. 20, it can also be seen that the results are as expected. For example, the changing of wall thickness from 0.4 m to 0.8 m causes a shear stress increment of 50% at the back face of the wall for S3 soil type under Coalinga earthquake due to wall rigidity. The same tendency may be clearly observed for S5 soil type as shown in Fig. 20. The results show that considering the wall flexibility/rigidity in SSI will amplify/reduce the dynamic response of the system. This implies that the response amplification or reduction pattern due to the deformable foundation is highly dependent on the wall flexibility, and the time history diagrams describe different behaviors of the

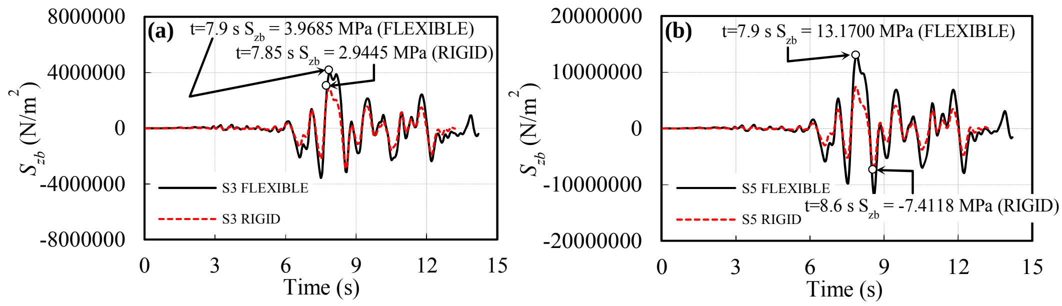

Fig. 19 Variation of stresses in time in *z* direction at the back face of flexible and rigid cantilever retaining walls under Loma Prieta earthquake for (a) S3 (b) S5 soil types

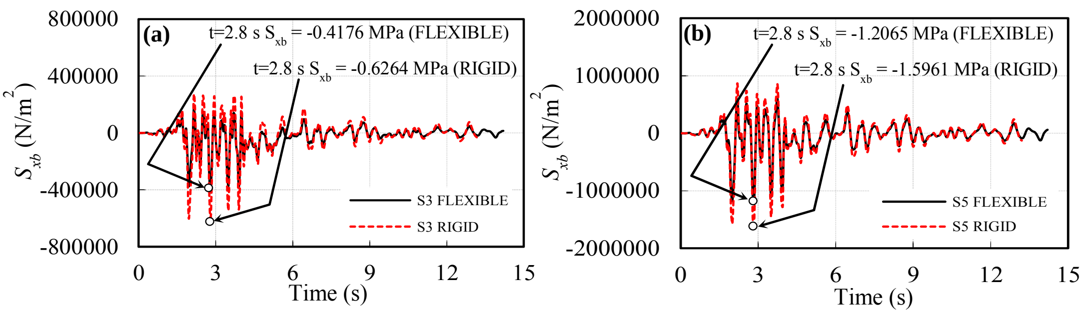

Fig. 20 Variation of stresses in time in x direction at the back face of flexible and rigid cantilever retaining walls under Coalinga earthquake for (a) S3 (b) S5 soil types

structure. Therefore, these assessments may be considered as an alert that the wall flexibility is of critical importance, and thus should be taken into account with utmost care in design process.

7. Conclusions

A series of dynamic analyses were conducted to determine seismic behavior of cantilever retaining walls by means of the finite element model developed in three dimensional space, considering the effects of SSI, ground motion and wall flexibility. Six different soil types, two different ground motions and two different wall thicknesses were taken into account in the analyses. It is concluded that the seismic response of cantilever retaining structures is a complex SSI problem, and the magnitudes of wall movements and stresses in the wall induced by horizontal ground shaking are quite sensitive to the response of the soil underlying the wall, the inertial and flexural responses of the wall itself, and the nature of the ground motions.

The dynamic response of backfill-cantilever wall-soil/foundation system was assessed by using the time histories of calculated lateral displacements of the wall and stresses in the wall. When the lateral displacements and stresses are scrutinized, it is highlighted that the response amplification or reduction pattern due to the deformable foundation under different ground motions is highly dependent on the wall flexibility/rigidity. Therefore, the exclusion of the effect of wall flexibility may cause underestimation or overestimation of the response, and this, in turn, may lead to unsafe seismic design of cantilever retaining walls. The responses of the wall also shed light on the importance of SSI for a proper design of walls since all responses change remarkably when the soil gets softer. Furthermore, it is found that effect of earthquake characteristics is significant on structural response, and may cause a considerable increase in time domain peak response values.

The comprehensive numerical solutions presented and their evaluation can provide not only insight into the effects and relative importance of the different factors that influence the response of the systems examined but also a framework for assessing the behavior of complex backfill-cantilever wall-soil/foundation interaction system. At this point, it should be stated that more analytical, numerical and experimental works and well documented case histories may be needed not only to further develop methods of analysis that are consistent with the actual dynamic behavior of these systems but also to generalize the results from the procedure presented here. However, it is hoped that this study will provide a contribution to the available knowledge database of the seismic analysis of cantilever walls.

References

Al Atik, L. and Sitar, N. (2010), "Seismic earth pressures on cantilever retaining structures", *J. Geotech. Geoenviron. Eng*., ASCE, **136**(10), 1324-1333.

Ambrosini, D. and Luccioni, M.B. (2009), "Reinforced concrete wall as protection against accidental explosions in the petrochemical industry", *Struct. Eng. Mech.*, **32**(2), 213-233.

ANSYS 10 (2006), ANSYS Inc., Canonsburg, PA.

Cakir, T. and Livaoglu, R. (2012), "Fast practical analytical model for analysis of backfill-rectangular tank-fluid interaction systems", *Soil Dyn. Earthq. Eng*., **37**, 24-37.

Cakir, T. (2013), "Evaluation of the effect of earthquake frequency content on seismic behavior of cantilever retaining wall including soil-structure interaction", *Soil Dyn. Earthq. Eng*., **45**, 96-111.

Callisto, L. and Soccodato, F.M. (2010), "Seismic design of flexible cantilevered retaining walls", *J. Geotech. Geoenviron. Eng*., ASCE, **136**(2), 344-354.

Caltabiano, S., Cascone, E. and Maugeri, M. (2012), "Static and seismic limit equilibrium analysis of sliding retaining walls under different surcharge conditions", *Soil Dyn. Earthq. Eng*., **37**, 38-55.

Carpinteri, A., Corrado, M., Lacidogna, G. and Cammarano, S. (2012), "Lateral load effects on tall shear wall structures of different height", *Struct. Eng. Mech.*, **41**(3), 313-337.

Chopra, A.K. (2007), *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, Prentice Hall, New Jersey.

di Santolo, A.S. and Evangelista, A. (2011), "Dynamic active earth pressure on cantilever retaining walls", *Comput. Geotech*., **38**, 1041-1051.

EAK 2000 (2003), Greek Seismic Code, Earthquake Planning and Protection Organization, Athens.

Eurocode 8 (2003), "Design of structures for earthquake resistance - part 5: Foundations, retaining structures and geotechnical aspects", Final draft, European Committee for Standardization, Brussels, Belgium.

Evangelista, A., di Santolo, A.S. and Simonelli, A.L. (2010), "Evaluation of pseudostatic active earth pressure coefficient of cantilever retaining walls", *Soil Dyn. Earthq. Eng*., **30**, 1119-1128.

Gazetas, G., Psarropoulos, P.N., Anastasopoulos, I. and Gerolymos, N. (2004), "Seismic behaviour of flexible retaining systems subjected to short-duration moderately strong excitation", *Soil Dyn. Earthq. Eng*., **24**, 537-550.

Giarlelis, C. and Mylonakis, G. (2011), "Interpretation of dynamic retaining wall model tests in light of elastic and plastic solutions" *Soil Dyn. Earthq. Eng.*, **31**, 16-24.

Huang, C.C., Wu, S.H. and Wu, H.J. (2009), "Seismic displacement criterion for soil retaining walls based on soil strength mobilization", *J. Geotech. Geoenviron. Eng.*, ASCE, **135**(1), 74-83.

IS-1893 (2002), "Indian standard criteria for earthquake for earthquake resistant design of structures", Part 1, General Provisions and Buildings.

Kloukinas, P., Penna, A., di Santolo, A.S., Bhattacharya, S., Dietz, M., Dihoru, L., Evangelista, A., Simonelli, A.L., Taylor, C.A. and Mylonakis, G. (2012), “Experimental investigation of dynamic behavior of cantilever retaining walls”, *Second International Conference on Performance-Based Design in Earthquake Geotechnical Engineering*, Taormina, Italy, May.

Kuhlemeyer, R.L. and Lysmer, J. (1973), “Finite element method accuracy for wave propagation problems”, *J. Soil Mech. Found. Div.*, ASCE, **99**, 421-427.

Livaoglu, R. and Dogangun, A. (2007), “Effect of foundation embedment on seismic behavior of elevated tanks considering fluid-structure-soil interaction”, *Soil Dyn. Earthq. Eng*., **27**, 855-863.

Livaoglu, R., Cakir, T., Dogangun, A. and Aytekin, M. (2011), “Effects of backfill on seismic behavior of rectangular tanks”, *Ocean Eng*., **38**, 1161-1173.

Luco, J.E. and Hadjian, A.H. (1974), “Two-dimensional approximations to the three-dimensional soil-structure interaction problem”, *Nucl. Eng. Des.*, **31**, 195-203.

Lysmer, J. and Kuhlemeyer, R.L. (1969), “Finite dynamic model for infinite media”, *J. Eng. Mech. Div.*, ASCE, **95**, 859-877.

Madabhushi, S.P.G. and Zeng, X. (2007), “Simulating seismic response of cantilever retaining walls”, *J. Geotech. Geoenviron. Eng*., ASCE, **133**, 539-549.

Mojallal, M. and Ghanbari, A. (2012), “Prediction of seismic displacements in gravity retaining walls based on limit analysis approach”, *Struct. Eng. Mech.*, **42**(2), 247-267.

Mylonakis, G., Kloukinas, P. and Papantonopoulos, C. (2007), “An alternative to the Mononobe-Okabe equations for seismic earth pressures”, *Soil Dyn. Earthq. Eng*., **27**, 957-969.

Nakamura, S. (2006), “Re-examination of Mononobe-Okabe theory of gravity retaining walls using centrifuge model tests”, *Soil. Found.*, **46**(2), 135-146.

Nazarian, H.N. and Hadjian, A.H. (1979), “Earthquake-induced lateral soil pressures on structures”, *J. Geotech. Eng. Div.*, ASCE, **105**(GT9), 1049-1066.

NTC (2008), Italian Building Code, DM 14 January, G.U., no. 29, February, no. 30.

PEER (2012), http://peer.berkeley.edu/svbin/GeneralSearch.

Pöttler, R. (1992), “Three-dimensional modelling of junctions at the Channel Tunnel project”, *Int. J. Numer. Anal. Meth. Geomech*., **16**, 683-695.

Psarropoulos, P.N., Klonaris, G. and Gazetas, G. (2005), “Seismic earth pressures on rigid and flexible retaining walls”, *Soil Dyn. Earthq. Eng*., **25**, 795-809.

Sezen, H. and Whittaker, A.S. (2006), “Seismic performance of industrial facilities affected by the 1999 Turkey earthquake”, *J. Perform. Constr. Facil.*, ASCE, **20**(1), 28-36.

Shukla, S.K. and Bathurst, R.J. (2012), “An analytical expression for the dynamic active thrust from c-φ soil backfill on retaining walls with wall friction and adhesion”, *Geomech. Eng.*, **4**(3), 209-218.

TEC (2007), “Specification for structures to be built in disaster areas”, Turkish Earthquake Code, Ministry of Public Works and Settlement Government of Republic of Turkey.

Theodorakopoulos, D.D., Chassiakos, A.P. and Beskos, D.E. (2001), “Dynamic pressures on rigid cantilever walls retaining poroelastic soil media. Part I. First method of solution”, *Soil Dyn. Earthq. Eng*., **21**, 315-338.

Veletsos, A.S. and Younan, A.H. (1994), “Dynamic soil pressures on rigid vertical walls”, *Earthq. Eng. Struct. Dyn*., **23**(3), 275-301.

Veletsos, A.S. and Younan, A.H. (1997), “Dynamic response of cantilever walls”, *J. Geotech. Eng.*, **123**(2), 161-172.

Wilson, E.L. (2002), *Three-Dimensional Static and Dynamic Analysis of Structures- A Physical Approach with Emphasis on Earthquake Engineering*, Computers and Structures, Inc., Berkeley, California, USA.

Wolf, J.P. and Song, C. (1996), “Finite element modelling of unbounded media”, *Proceedings of the 11th World Conference on Earthquake Engineering*, San Francisco.

Wolf, J.P. and Song, C. (2002), “Some cornerstones of dynamic soil-structure interaction”, *Eng. Struct.*, **24**, 13-28.

Younan, A.H. and Veletsos, A.S. (2000), “Dynamic response of flexible retaining walls”, *Earthquake Eng. Struct. Dyn*., **29**, 1815-1844.