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Assessment of effect of material properties on seismic response of a cantilever wall

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Abstract. Cantilever retaining wall movements generally depend on the intensity and duration of ground motion, the response of the soil underlying the wall, the response of the backfill, the structural rigidity, and soil-structure interaction (SSI). This paper investigates the effect of material properties on seismic response of backfill-cantilever retaining wall-soil/foundation interaction system considering SSI. The material properties varied include the modulus of elasticity, Poisson's ratio, and mass density of the wall material. A series of nonlinear time history analyses with variation of material properties of the cantilever retaining wall are carried out by using the suggested finite element model (FEM). The backfill and foundation soil are modelled as an elastoplastic medium obeying the Drucker-Prager yield criterion, and the backfill-wall interface behavior is taken into consideration by using interface elements between the wall and soil to allow for de-bonding. The viscous boundary model is used in three dimensions to consider radiational effect of the seismic waves through the soil medium. In the seismic analyses, North-South component of the ground motion recorded during August 17, 1999 Kocaeli Earthquake in Yarimca station is used. Dynamic equations of motions are solved by using Newmark's direct step-by-step integration method. The response quantities 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 while the modulus of elasticity has a considerable effect on seismic behavior of cantilever retaining wall, the Poisson's ratio and mass density of the wall material have negligible effects on seismic response.

Keywords: modulus of elasticity; Poisson's ratio; mass density; cantilever retaining wall; SSI

1. Introduction

Earth retaining structures, such as retaining walls, bridge abutments, quay walls, anchored bulkheads, braced excavations, and mechanically stabilized walls, are used throughout seismically active areas. They frequently represent key elements of ports and harbors, transportation systems, lifelines, and other constructed facilities (Kramer 1996). Experience from past earthquakes has demonstrated that cantilever retaining walls subjected to dynamic loads have often been severely damaged although they were thought to have been properly designed against seismic motions. Many factors influence the seismic response of retaining walls. Several of the more important factors include the intensity and duration of strong ground motion, relative soil-structure displacements, dynamic properties of local soil deposits, structural rigidity, backfill and foundation soil properties, and SSI. The term "SSI" is an interdisciplinary field, and can include the

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interaction of all types of structures with the soil that they are constructed in or upon. Foundations for structures such as buildings, bridges, silos, and storage tanks, form the most common type of problem considered, while arch or rectangular culverts and retaining walls of all types form a large part of interaction analysis (Small 2001).

It has generally been recognized that the interaction between soil and structure can indeed affect the response of the structures, especially for structures on relatively flexible soil. The inclusion of the SSI effects is particularly important in the seismic analyses of structures located in active seismic zones. Therefore, accurate representation of the SSI effects is a crucial part of the seismic analysis (Livaoglu and Dogangun 2006). In general, a number of different sophisticated mathematical techniques and elaborate computer codes are available for assessing SSI effects (Veletsos 1984, Wolf 1985, Youssef 1998, Livaoglu 2008, 2013, Durmus and Livaoglu 2015, Khazaei *et al.* 2017). The SSI effect is considered important, for example, when the motion of the structural base is significantly different from the motion of the ground in the absence of the structure, the latter motion being usually referred to as the free-field ground motion (Tsai *et al.* 1974). From the standpoint of the structure, the SSI effects alter the structural response by modifying the free-field ground motion into the actual foundation motion (Wu 1997). The significance of SSI on the structural response calculations depends on the stiffness, mass and damping characteristics of the structure relative to those of the foundation. The flexibility of the supporting medium has a two-fold effect: (1) It increases the number of degrees of freedom of the system and lowers its effective stiffness; and (2) it makes it possible for part of the vibrational energy of the structure to be dissipated in the supporting medium by radiation of waves and by hysteretic action in the soil itself (Veletsos *et al*. 1988). Two mechanisms of interaction take place between the structure, foundation, and soil:

• Inertial interaction: Inertia developed in the structure due to its own vibrations gives rise to base shear and moment, which in turn cause displacements of the foundation relative to the free-field.

• Kinematic interaction: The presence of stiff foundation elements on or in soil cause foundation motions to deviate from free-field motions as a result of ground motion incoherence, wave inclination, or foundation embedment (Stewart *et al.* 1999).

The total SSI is given by the sum of the inertial and kinematic interaction effects.

The analysis of the seismic response of SSI systems has been well studied in the past. There are generally two major methods for analyzing SSI (Wolf 1988): the direct method and the substructure method. Accumulating experience indicates that while the direct method is a conceptually easier way to model the entire soil-structure system in a single step, the substructure method is computationally more efficient (Li *et al*. 2008). The direct method of analysis proceeds by applying a consistent free-field ground motion to the boundaries of a discrete model and computing the response of the combined soil-structure system. Hence, a direct method determines the response of the soil and structure simultaneously. In practice, the structural model used in a direct method of analysis represents only an overall dynamic response of the structure. A second stage structural analysis must be performed to obtain the detailed structural response, using the results of SSI analysis as input. Implementation of a direct method requires solution of the free-field ground motion problem, analysis of the coupled soil-structure system, and a detailed second-stage analysis of structural response (Jaya and Prasad 2002). On the other hand, in the substructure method, the soil-structure system is divided into two parts: one part is the generalized structure including a portion of adjacent soil with an irregular boundary, which can behave nonlinearly; the other part is the semi-infinite, unbounded linear soil medium. The generalized soil-structure interface separates the two parts (Li *et al*. 2008). In this study, the direct method of analysis is adopted for SSI evaluation.

There are mainly three categories of methods for design and seismic analysis of retaining walls: (1) analytic limit-state analysis methods where the wall can displace and/or rotate sufficiently at its base to induce a limit or failure state in the backfill, (2) analytic linear elastic or viscoelastic methods where the wall remains fixed at its base and the backfill soil is considered to respond in a linear elastic or viscoelastic manner, (3) numerical methods of solution, mainly finite element methods under the assumption of linear elastic or nonlinear elastoplastic soil behavior (Nazarian and Hadjian 1979, Veletsos and Younan 1994, Vrettos *et al.* 2016). An extensive list of papers for each one of the above three categories can be found in Nazarian and Hadjian 1979, Veletsos and Younan 1994, 1997, Theodorakopoulos *et al.* 2001, Gazetas *et al.* 2004, Psarropoulos *et al.* 2005, Giarlelis and Mylonakis 2011, Cakir 2013, 2014a, b, Vrettos *et al.* 2016), and the details need not be repeated herein. On the other hand, some of the analytical, numerical, and experimental studies performed in recent years related to backfill-wall systems will be briefly summarized below to reflect the current state-of-technology. The present paper belongs to the third category of methods to seismically analyze the cantilever retaining wall under consideration.

Al Atik and Sitar (2010) designed an experimental and analytical program to assess the magnitude and distribution of seismically induced lateral soil pressures on cantilever walls with dry medium dense sand backfill, and they showed that the finite element analysis is able to capture quite well the system responses observed in the centrifuge experiments. di Santolo and Evangelista (2011) reported the results of the analyses performed to evaluate active pressure in different soil behavior and displacements under seismic loads of two cantilever walls, and they stated that the theoretical and numerical simplified pseudo-static approaches give proximal results regardless of the different hypotheses on soil deformability. 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 noted that the measured results from the shaking table experiments compare far better with the limit-state solutions, whereas for the centrifuge tests the results seem to match better with the elastic solution. Shukla and Bathurst (2012) derived an improved explicit analytical expression in terms of seismic earth pressure coefficients and a tension crack factor for calculating the dynamic active thrust from a c-ϕ soil backfill acting at the back of a rigid retaining wall with uniform surcharge and wall-soil friction and adhesion, and they showed that the analytical expression gives the same results for simpler special cases previously reported in the literature. Cakir and Livaoglu (2013) performed finite element analysis and in-situ tests on determination of modal characteristics of backfill-wall-fluid system, and found close agreement between theory and experiment. Cakir (2013) evaluated effects of earthquake frequency content and SSI on seismic behavior of cantilever retaining walls, and concluded that the earthquake frequency content and SSI are among the most important parameters to be considered in seismic analysis. Cakir (2014a, b, 2016) investigated effects of wall flexibility and backfill interaction effects on dynamic response of cantilever retaining walls, and showed that these parameters are quite important for design. Wilson and Elgamal (2015) conducted shake table tests for determination dynamic lateral earth pressure, and indicated that limit equilibrium predictions using residual ϕ with c=0 significantly over-predicted all of the test results, and exclusion of the cohesion intercept results in substantial over-prediction of the measured lateral forces. Papagiannopoulos *et al*. (2015) examined the dynamic response of a water-saturated linear poroelastic soil layer over bedrock retained by a pair of rigid cantilever walls to a horizontal seismic excitation under plane strain conditions. They computed soil displacements and stresses, pore water pressure as well as wall pressure, base shear, and bending moment, and displayed their variations with frequency hysteretic damping, porosity, and permeability in pictorial form. Brandenberg *et al.* (2015) presented a new interpretation of theoretical and experimental test results in the frame work of kinematic SSI and wave propagation, and concluded that the proposed approach produces estimates of seismic earth pressures that are significantly more accurate than Mononobe-Okabe theory. Kloukinas *et al*. (2015) explored the earthquake response of cantilever retaining walls by means of theoretical analyses and shaking table testing considering different aspects of the problem, and showed that the results of the experimental investigations are in good agreement with the theoretical models and provide a better understanding on the complex mechanics of the problem. Vrettos *et al*. (2016) investigated the dynamic response of an elastic continuously nonhomogeneous soil layer over bedrock retained by a pair of rigid cantilever walls to a horizontal seismic motion and the associated seismic pressure acting on these walls analytically-numerically. They determined the seismic pressures, resultant horizontal forces, and bending moments acting on the walls, and evaluated the effect of soil non-homogeneity on the system response. In addition to the abovementioned studies, some researchers investigated the effect of the material properties of structures on seismic behavior. Sevim (2011) investigated the effect of material properties on the seismic performance of arch dam-reservoir-foundation interaction systems, and concluded that different material properties affect the seismic behavior of the dam. Sevim *et al*. (2014) examined the effects of the construction stages using time dependent material properties on the structural behavior of concrete arch dams, and stated that construction stage analysis using time dependent material strength variations and geometric variations has an important effect on the structural behavior of arch dams.

Literature investigation shows that although much research has been carried out concerning the dynamic behavior of retaining walls, the traditional approach in geotechnical practice is to work in terms of the dynamic soil pressure distributions. On the other hand, limited research has been carried out on the effects of material properties on seismic response of the cantilever walls considering SSI. Furthermore, numerical model studies have demonstrated that the retaining wall rigidity/movement can substantially impact the seismic behavior of retaining walls. As such, the main target of this study is to shed light on the effects of the variation of elasticity modulus, Poisson’s ratio, and mass density on dynamic response of a cantilever retaining wall subjected to backfill and subsoil interactions.

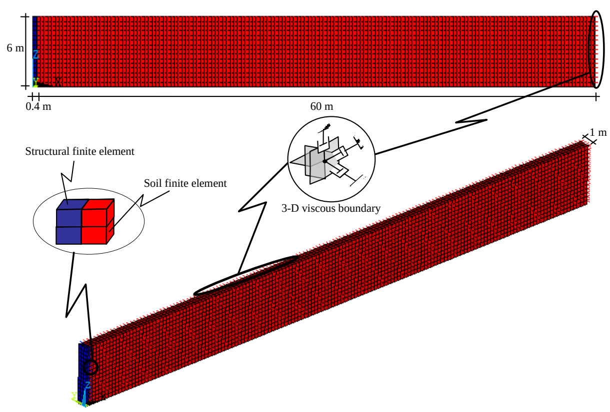

Fig. 1 FEM of backfill-cantilever wall system

2. FEM under fixed-base condition

The problem under investigation consists of a uniform layer of elastic material that is free at its upper surface, bonded to a non-deformable rigid base, and 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, and they are denoted by H. The properties of the soil stratum are defined by its mass density, shear modulus of elasticity, and Poisson's ratio. The properties of the structural wall are defined by its thickness, mass density, moment of inertia, Young's modulus, and Poisson's ratio. The proposed FEM for the problem under fixed-base assumption is depicted in Fig. 1. The computer program, ANSYS was used to model the system (ANSYS 2010).

The cantilever wall is discretized by 3-D solid elements (SOLID 65) defined by eight nodes having three translational degrees of freedom in each node. 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 de-bonding was available in ANSYS, the assumption of complete bonding -made by both in the study of Veletsos and Younan (1994) and in the simplified model proposed by the author in this study- was also adopted to permit a comparative study at this stage. However, following the analytical verification of the numerical solution, the de-bonding behavior between the wall and the soil is also considered by using special interface elements when dynamic analysis of the interaction system is carried out.

3. Simplified model and verification

The equivalent spring-dashpot-mass model can be constructed for practical use in many engineering problems. In this connection, a simplified model with constant parameters is introduced in order to demonstrate the accuracy of the FEM. The model is given in Fig. 2. To calculate the stiffness and mass values of backfill soil, the equations presented by Veletsos and Younan (1994) were 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 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.

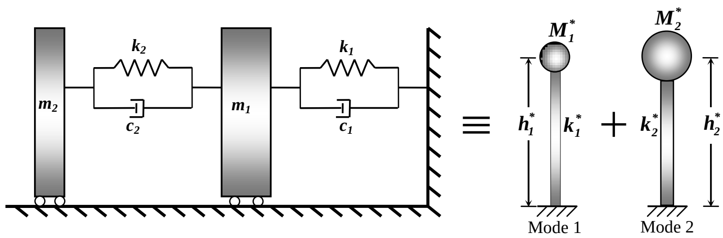

Fig. 2 Proposed simplified model

To define the modal characteristics of the system, the design parameters should be introduced primarily.

The mass m_1 refers to soil mass and is equal to;

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

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

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 ψ_σ, ψ_0, ψ_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.

The mass of the wall is represented by m_2, and the lateral stiffness of the wall, k_2, can 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 here. However, these data will be included in Section 5, where the dynamic analysis of the interaction system is performed by means of the FEM.

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) where N, ϕ_n and ω^2_n are the total mode number, the n^th^ mode vector and its eigenvalue, respectively.

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

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

The modal analysis was carried out through a computer program coded by the author.

To show the effectiveness of the FEM, the modal analysis of the systems is done. In the numerical example, a 6 m-high cantilever retaining wall with a constant thickness of 0.4 m is considered. 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 seen from 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 FEM. 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 FEM. The modal analysis was carried out assuming elastic material responses in this study. Furthermore, it is worth noting that the viscous boundaries and nonlinear properties considered in the FEM are for clear understanding of seismic response of the system during time history analysis.

A comparison between the analytical and numerical results can be done in this section. It should be noted here that only the comparison of the modes regarding the 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 can find very good agreement between the numerical and analytical results. The mode frequency is computed as 4.47 Hz from the analytical model whereas the same quantity is calculated as 4.45 Hz from the FEM. Actually, this reveals successful estimation, and the analytical verification provides strong support for the FEM for use in further investigations.

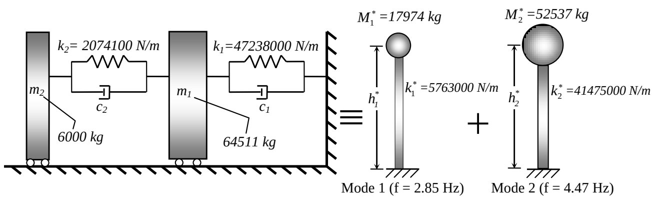

Fig. 3 Modal properties obtained from the simplified model

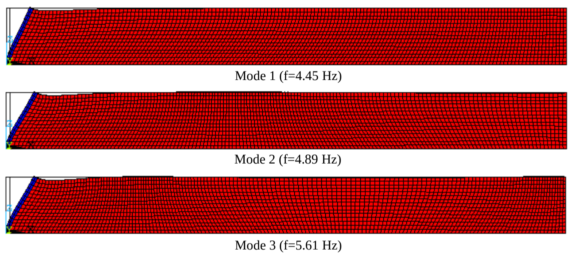

Fig. 4 Mode shapes and frequencies obtained from the FEM

4. FEM of the backfill-cantilever wall-soil/foundation system

After an analytical verification of the numerical solution, the versatility of the FEM permits the treatment of some more realistic situations that are not amenable to analytical solution. Therefore, the modelling was extended to account for the behavior of wall-soil interface, elastoplastic behavior of soil, and soil/foundation interaction effects. For solving the problem of backfill-cantilever wall-soil/foundation system, the general purpose structural analysis program ANSYS (2010) is also used. Numerical analysis of the cantilever retaining wall problem with backfill and subsoil interactions and subjected to earthquake loading is a complex problem. Fig. 5 shows the proposed FEM of the backfill-cantilever retaining wall-soil/foundation system, which contains different aspects of the model.

The heights of the wall and soil stratum are considered to be the same. The vertical stem height of the cantilever wall is H=6 m, the wall stem has a constant thickness of 0.4 m, the thickness of base slab is 0.6 m, and the base slab width is 4.0 m. The cantilever wall system is founded on a deformable soil layer of thickness 2H. In the finite element modelling, as it is mentioned before, the structural wall is modelled with 3-D reinforced concrete solid elements (SOLID65) defined by eight nodes having three translational degrees of freedom in each node. The SOLID65 is used for the 3-D modeling of solids with or without reinforcing bars. The solid is capable of cracking in tension and crushing in compression. The backfill and soil/foundation system are modelled with 3-D structural solid elements (SOLID185) with eight nodes having three degrees-of-freedom at each node: translations in the nodal x, y, and z directions. The SOLID185 has plasticity, hyperelasticity, stress stiffening, creep, large deflection, and large strain capabilities. It also has mixed formulation capability for simulating deformations of nearly incompressible elastoplastic materials, and fully incompressible hyperelastic materials. 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 (COMBIN39) is used between the backfill

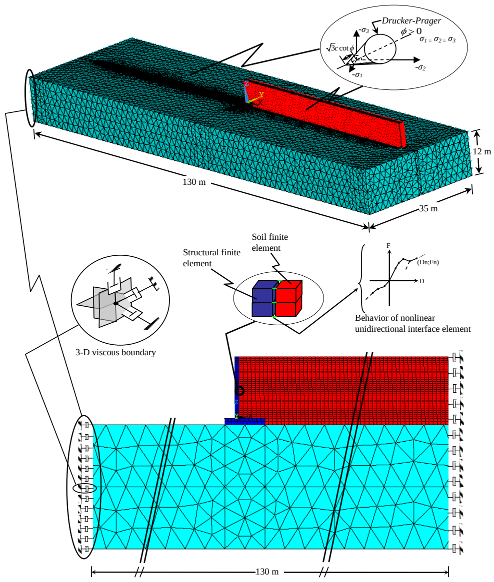

Fig. 5 FEM of the backfill-cantilever wall-soil/foundation interaction system

and the wall allowing for the opening and closing of the gaps (i.e., de-bonding and bonding) to model backfill-wall interaction in this study. COMBIN39 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.

In a dynamic SSI analysis, a bounded structure (which may be linear or nonlinear) consisting of the actual structure and an adjacent irregular soil if present, will interact with the unbounded (infinite or semi-infinite) soil which is assumed to be linear elastic. The most striking feature in an unbounded soil, which is never encountered in a bounded medium, is, in general, the radiation of energy towards infinity, leading to so-called radiation damping even in such a linear system (Wolf and Song 2002). There are traditionally two ways for implementation of the radiation condition: one way is to enforce the condition rigorously at the soil-structure interface by using the boundary element technique, and the other way is to impose a wave absorbing boundary condition on the outer boundary of a bounded domain (Li *et al*. 2008). In this study, the viscous boundary model (Lysmer and Kuhlemeyer 1969) is used in three dimensions to consider radiational effect of the seismic waves through the soil medium. 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 are 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. Moreover, the soil is modeled as an elastoplastic medium obeying the well-known Drucker-Prager yield criterion. It is well known that soil materials have a very complicated behavior. Idealizations are, therefore, often necessary in order to develop simple mathematical constitutive laws for practical applications. Several models can be found in the literature, most of them are complex and require many parameters, where some can be physically meaningless.

Therefore, practical engineers very often prefer relatively simple elastic-perfectly plastic material models (Schweiger 1994). The relative simplicity of the Drucker-Prager model, which can reflect some characteristics of soil behavior with only three parameters, explains why this model is widely used. Furthermore, it is possible to mention the other advantages of the Drucker-Prager model. For example; it can be matched with the Coulomb model, its validity is well established for many soils, by a proper choice of constants, computer codes are available for it, and it satisfies the uniqueness requirement etc. (Chen and Mizuno 1990). In the light of the explanations mentioned above, Drucker-Prager material model was used for soils in the FEM. Furthermore, elastic material properties were considered for cantilever wall. On the other hand, the designers for special design, and scientists for scientific purposes to the specific problem can use rigorous models for concrete wall. However, in this study, the main purpose has been selected to investigate parametrically the effects of the variation of constant material properties of wall on seismic response of cantilever wall considering SSI. For such a purpose, the author did not consider to concentrate on the effects of specific properties of concrete.

5. Time history analysis

Static analysis may be carried out to show material effects. However, one of the main targets of this study is also to investigate dynamic response of cantilever retaining walls considering SSI.

Table 1 Material properties of cantilever wall, foundation soil and backfill soil

Component	Elasticity Modulus (MPa)			Poisson's Ratio			Mass Density (kg/m^3)		
Cantilever wall	E1	E2	E3	P1	P2	P3	D1	D2	D3
	24000	28000	32000	0.16	0.20	0.24	2200	2400	2600
Foundation soil	150			0.30			1800		
Backfill soil	50			0.30			1800		

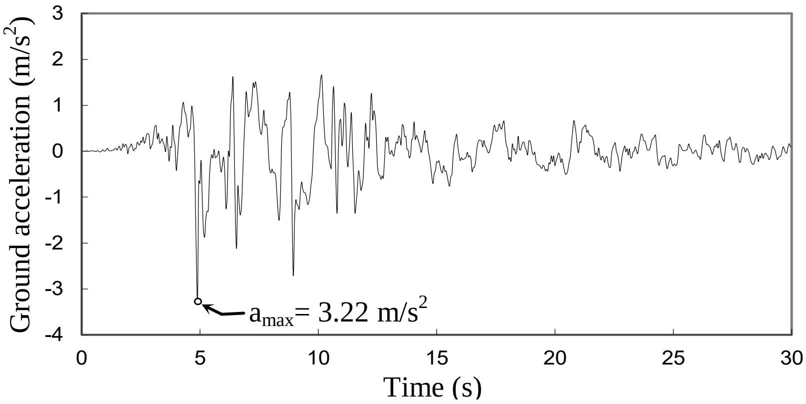

Fig. 6 Ground motion recorded at Yarimca Station during 1999 Kocaeli Earthquake

Therefore, time history analysis was performed in this study. A series of nonlinear time history analyses with variation of material properties of the cantilever retaining wall are carried out by using the suggested FEM. As shown in Table 1, the material properties of the cantilever wall are taken into account in three groups. In the first group, three elasticity modules are selected as E1, E2, and E3; Poisson's ratio and mass density are constant as 0.2 and 2400 kg/m^3, respectively. In the second group, three Poisson's ratios are selected as P1, P2, and P3; elasticity modulus and mass density are constant as 28000 MPa and 2400 kg/m^3, respectively. In the third group, three mass densities are considered as D1, D2, and D3; elasticity modulus and Poisson's ratio are constant as 28000 MPa and 0.2, respectively. Furthermore, as can be seen from Table 1, foundation and backfill soil properties are constant in the analyses. In the seismic analyses, North-South component of the ground motion recorded during August 17, 1999 Kocaeli Earthquake in Yarimca station is used (Fig. 6). In the FE procedure, Rayleigh damping is taken into consideration in the Newmark's direct step-by-step integration method. The damping values for both structure and soil are taken as 5%.

6. Results and discussions

Analysis results are presented in three major steps by calculating lateral displacements and stresses. In the first step, a detailed discussion on the effects of elasticity modulus on seismic behavior of cantilever wall is given. In the second step, the effects of Poisson's ratio on dynamic behavior of cantilever wall subjected to the backfill and soil/foundation interactions are discussed. In the third step, the effects of mass density on seismic response of cantilever wall are evaluated.

Table 2 Summary of the maximum dynamic responses and their occurrence times considering variation of elasticity modulus

Maximum responses	Case					
	E1		E2		E3	
	t(s)	Value	t(s)	Value	t(s)	Value
u_t (m)	4.8	0.0245	4.8	0.0242	4.8	0.0240
S_{zb} (MPa)	9.7	6.1246	9.7	6.5181	5.35	6.9420
S_{yb} (MPa)	9.7	0.8080	5.35	0.8582	5.35	0.9143
S_{xb} (MPa)	9.7	2.4918	9.7	2.6455	9.7	2.7693
S_{zf} (MPa)	9.7	-6.1998	9.7	-6.5996	5.35	-7.0213
S_{yf} (MPa)	5.35	-0.4834	5.35	-0.5156	5.35	-0.5482
S_{xf} (MPa)	5.3	-0.4330	5.3	-0.4548	5.3	-0.4775

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

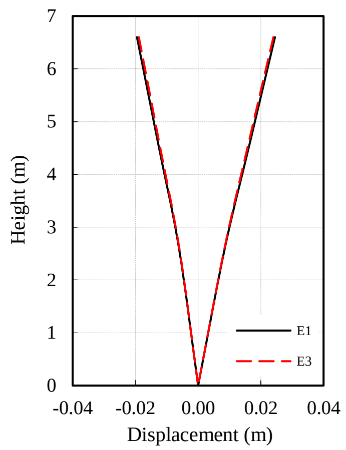

Fig. 7 Height-wise variation of lateral displacements of the cantilever wall depending on the variation of elasticity modulus

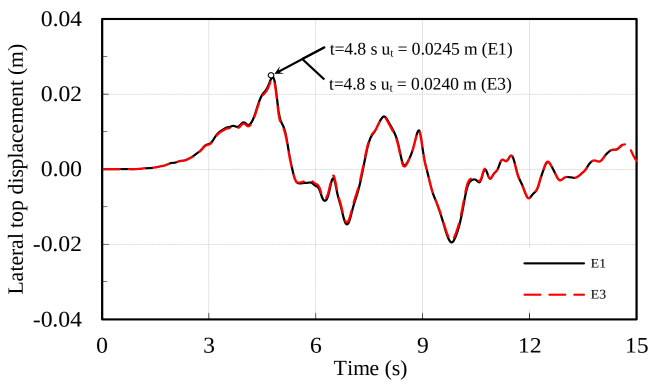

Fig. 8 Calculated lateral top displacement time histories of cantilever retaining wall depending on the variation of elasticity modulus

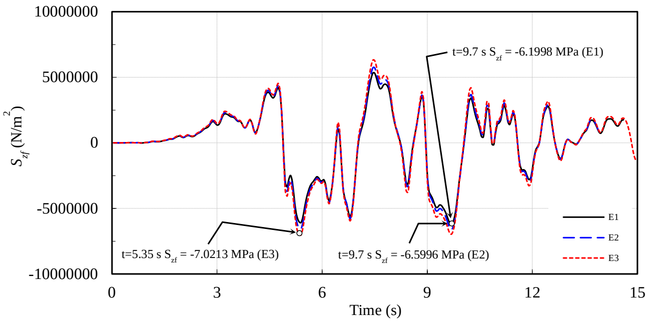

Fig. 9 Calculated stress time histories in z direction at the front face of the cantilever wall depending on the variation of elasticity modulus

6.1 Evaluation of the effects of modulus of elasticity

Table 2 summarizes the maximum top displacements and stress responses at the front and back faces of the cantilever wall and their occurrence times depending on the variation of elasticity modulus of wall. As can be seen from Table 2, while the lateral displacements are not practically affected by the variation of elasticity modulus of the wall, the stress responses change to a certain extent.

The effects of elasticity modulus on seismic response of cantilever wall are shown graphically, and discussed comparatively below. It is important to note here that since all results obtained from the analyses cannot be illustrated, some comparisons are selected to define the system behavior.

Fig. 7 portrays the height-wise variations of the lateral displacements of cantilever retaining wall for varying the elasticity modulus of the wall. It is worth noting here that these displacements represent the relative lateral displacements of the wall with respect to the ground. While the negative displacements refer to the movements away from the backfill, the positive ones refer to the movements toward the backfill. It is observed from this figure that as the elasticity modulus increases, the displacement response tends to decrease.

It is possible to assess the lateral displacements in terms of time history using the suggested model. Accordingly, the deviations of the displacements in time are illustrated and compared in Fig. 8 to clarify the changes of the lateral top displacement values due to the variation of elasticity modulus. It can be noted from Fig. 8 and Table 2 that while the maximum lateral displacement is estimated as 0.0245 m for E1, the same quantity is calculated as 0.0240 m for E3. Thus, it can be highlighted that the effect of the elasticity modulus can be ignored in the evaluation of the displacement behavior of the system so that the decrement in the displacement response is at a level of only 2% between E1 and E3. Furthermore, the computed time history results show that the maximum responses occur around the same time (t=4.8 s).

The estimated stress responses and their variations in time at the back and front faces of the cantilever retaining wall can be compared with each other to reveal the effects of modulus of elasticity. As an example, a comparison of stress time history responses in z direction for front face of the cantilever wall is shown in Fig. 9. As this figure depicts, the maximum stresses obtained at the critical section of the wall change with varying the modulus of elasticity. For example, while the peak stress, as compression, has the value of 6.1998 MPa for E1, it is calculated as 7.0213 MPa for E2. This reflects a stress increment of about 13% between E1 and E3 due to the variation

Table 3 Summary of the maximum dynamic responses and their occurrence times considering variation of Poisson's ratio

Maximum responses	Case					
	P1		P2		P3	
	t(s)	Value	t(s)	Value	t(s)	Value
u_t (m)	4.8	0.0242	4.8	0.0242	4.8	0.0242
S_{zb} (MPa)	9.7	6.5363	9.7	6.5181	9.7	6.5644
S_{yb} (MPa)	5.35	0.6892	5.35	0.8582	5.35	1.0324
S_{xb} (MPa)	9.7	2.6425	9.7	2.6455	9.7	2.6562
S_{zf} (MPa)	9.7	-6.6365	9.7	-6.5996	9.7	-6.6285
S_{yf} (MPa)	5.35	-0.4209	5.35	-0.5156	5.35	-0.6116
S_{xf} (MPa)	5.3	-0.4388	5.3	-0.4548	5.3	-0.4727

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

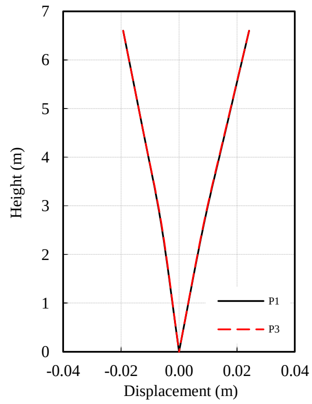

Fig. 10 Height-wise variation of lateral displacements of the cantilever wall depending on the variation of Poisson's ratio

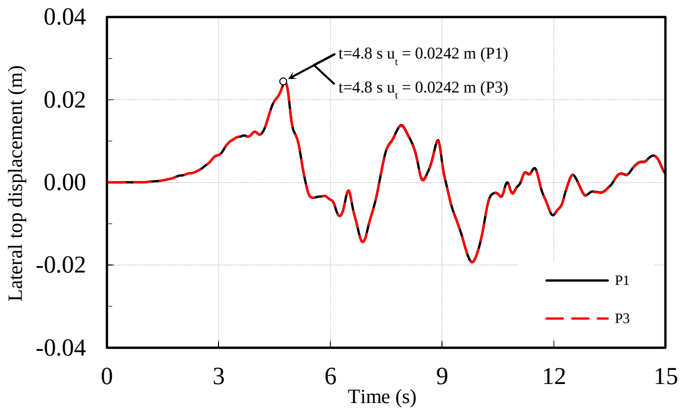

Fig. 11 Calculated lateral top displacement time histories of cantilever retaining wall depending on the variation of Poisson's ratio

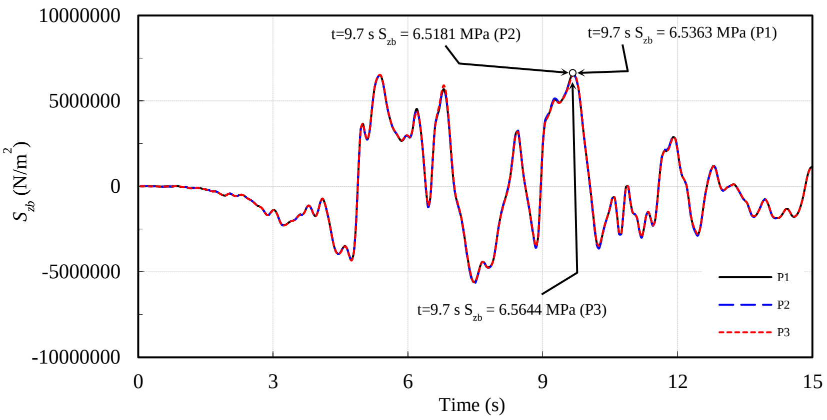

Fig. 12 Calculated stress time histories in z direction at the back face of the cantilever wall depending on the variation of Poisson's ratio

Table 4 Summary of the maximum dynamic responses and their occurrence times considering variation of mass density

Maximum responses	Case					
	D1		D2		D3	
	t(s)	Value	t(s)	Value	t(s)	Value
u_t (m)	4.8	0.0242	4.8	0.0242	4.8	0.0242
S_{zb} (MPa)	9.7	6.5291	9.7	6.5181	9.7	6.5233
S_{yb} (MPa)	5.35	0.8623	5.35	0.8582	5.35	0.8581
S_{xb} (MPa)	9.7	2.6379	9.7	2.6455	9.7	2.6498
S_{zf} (MPa)	9.7	-6.6100	9.7	-6.5996	9.7	-6.6052
S_{yf} (MPa)	5.35	-0.5179	5.35	-0.5156	5.35	-0.5157
S_{xf} (MPa)	5.3	-0.4560	5.3	-0.4548	5.3	-0.4552

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

of modulus of elasticity. If similar comparisons are made in the other directions from Table 2, the same trend can be clearly observed. For example, in *x* direction, the value of peak stress is 2.4918 MPa for E1, whereas the same quantity is calculated as 2.7693 MPa for E3, and a stress increment of nearly 11% takes place at the back face of the cantilever wall. These comparisons clearly show that the variation of the elasticity modulus affects the stress response of the system. Therefore, the material properties of structures are of importance, and should be measured with utmost care.

6.2 Evaluation of the effects of Poisson's ratio

Table 3 summarizes the maximum top displacements and stress responses at the front and back faces of the cantilever wall and their occurrence times depending on the variation of Poisson's ratio. As can be seen from Table 3, not only the lateral displacements but also the stress responses are not practically affected by the variation of Poisson's ratio of the wall material.

Figs. 10 and 11 depict the height-wise variations and time histories of the lateral displacements of cantilever retaining wall for varying the Poisson's ratio of the wall, respectively. It is observed from these figures that there is no response amplification/reduction depending on the Poisson's ratio variation. Accordingly, it can be clearly stated that the Poisson's ratio of the wall material has a negligible effect on seismic response of the backfill-cantilever wall-soil/foundation interaction system investigated here.

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 Poisson's ratio effects. The comparison of stress time history responses in *z* direction at the back face of the cantilever wall is shown in Fig. 12. As this figure depicts, the stresses obtained at the critical sections of the wall does not change with varying Poisson's ratio. Furthermore, in terms of the stress amplitude, the deviations between the three Poisson's ratio values exhibited that the responses are almost coincided. Thus, we can state that the Poisson's ratio variation does not have any considerable effects on the seismic response as already pointed out in the evaluation on lateral displacements.

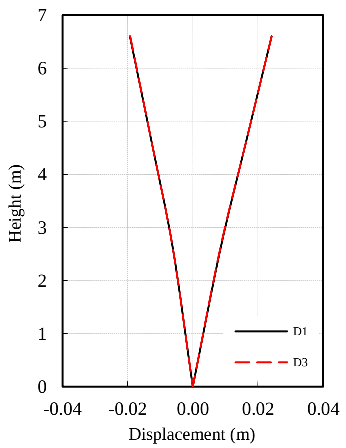

Fig. 13 Height-wise variation of lateral displacements of the cantilever wall depending on the variation of mass density

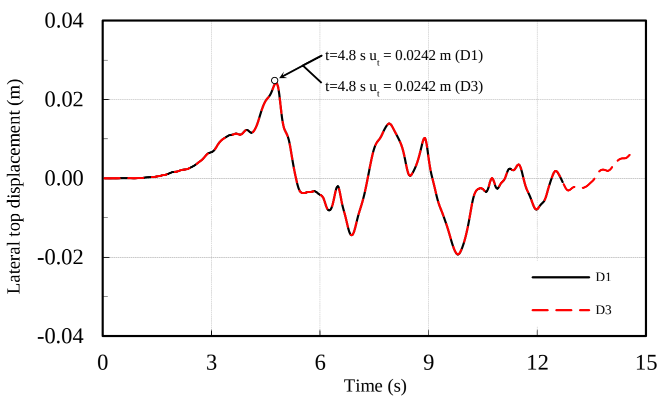

Fig. 14 Calculated lateral top displacement time histories of cantilever retaining wall depending on the variation of mass density

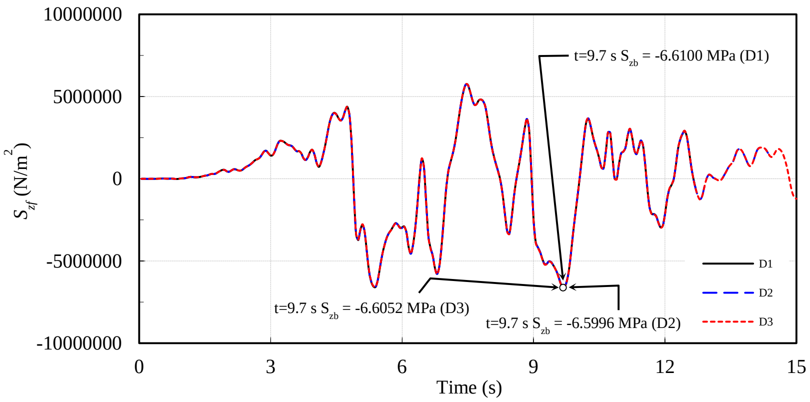

Fig. 15 Calculated stress time histories in z direction at the front face of the cantilever wall depending on the variation of mass density

6.3 Evaluation of the effects of mass density

Table 4 summarizes the maximum top displacements and stress responses at the front and back faces of the cantilever wall and their occurrence times depending on the variation of mass density. As can be seen from Table 4, both the lateral displacements and the stress responses are not practically affected by the variation of mass density of the wall material.

Figs. 13 and 14 show the height-wise variations and time histories of the lateral displacements of cantilever retaining wall for varying the mass density of the wall material, respectively. It is seen from these figures that there is no response amplification/reduction depending on the mass density variation, as clearly observed in the assessment of Poisson's ratio. In this connection, it can be clearly expressed that the mass density of the wall material has a negligible effect on seismic response of the cantilever retaining wall subjected to SSI.

In addition to the lateral displacement response, stress response of the cantilever wall considering the mass density variation is investigated in this section. The time history diagrams of the stress responses at the front face of the cantilever retaining wall in *z* direction for three mass density values are presented in Fig. 15. As depicted in Fig. 15, the stresses obtained at the critical sections of the wall do not remarkably change with varying mass density. For example, while the peak stress, as compression, has the value of 6.6100 MPa for D1, it is calculated as 6.5996 MPa for D2. If similar comparisons are made for the other directions from Table 4, the same tendency can be observed.

7. Conclusions

The problems relating to the vibration of soil and earth retaining structures have received great attention of geotechnical engineers in recent years, and significant advances have been made in this direction. New theoretical and experimental procedures have been developed for design of retaining walls due to the abundance and importance of them, and the complexity of their dynamic response. In the work at hand, a series of dynamic analyses were conducted to determine the effects of modulus of elasticity, Poisson's ratio, and mass density of the wall material on seismic behavior of the cantilever retaining wall through the 3D FEM considering SSI. Three different values of elasticity modulus, Poisson's ratio, and mass density were taken into account in the analyses. 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.

The analysis results showed that the modulus of elasticity has a considerable effect on seismic behavior of the cantilever retaining wall, and this phenomenon should be considered in design process. However, it was observed that there is no response amplification/reduction depending on the variation of Poisson's ratio and mass density. Accordingly, it can be stated that effects of Poisson's ratio and mass density of the wall material on the seismic response of the cantilever retaining wall can easily be ignored during seismic assessment. This situation may result from relatively narrow ranges of Poisson's ratio and mass density.

The present results are valid only for the interaction system investigated here. In this connection, more numerical examples considering other types of retaining walls, different soil properties and foundation conditions are needed for generalization of the results.

Acknowledgments

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