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Backfill and subsoil interaction effects on seismic behavior of a cantilever wall

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Abstract. The main focus of the current study is to evaluate the dynamic behavior of a cantilever retaining wall considering backfill and soil/foundation interaction effects. For this purpose, a three-dimensional finite element model (FEM) with viscous boundary is developed to investigate the seismic response of the cantilever wall. To demonstrate the validity of the FEM, analytical examinations are carried out by using modal analysis technique. The model verification is accomplished by comparing its predictions to results from analytical method with satisfactory agreement. The method is then employed to further investigate parametrically the effects of not only backfill but also soil/foundation interactions. By means of changing the soil properties, some comparisons are made on lateral displacements and stress responses. It is concluded that the lateral displacements and stresses in the wall are remarkably affected by backfill and subsoil interactions, and the dynamic behavior of the cantilever retaining wall is highly sensitive to mechanical properties of the soil material.

Keywords: soil-structure interaction; viscous boundary; finite element analysis; analytical verification

1. Introduction

The dynamic interaction between structure and soil medium is of significant interest to both geotechnical and structural engineers in many engineering problems. Seismic behavior of a cantilever retaining structure is a rather complex problem in spite of its structural simplicity. What makes that behavior so sophisticated is the dynamic interaction between both the wall and backfill soil, and the wall and underlying ground. Accordingly, seismic behavior of a retaining wall-soil system is a function of backfill soil properties, relative stiffness of the wall/soil system, wall fixity conditions, foundation stability, and characteristics of applied earthquake motions (Dewoolkar *et al.* 2001).

Despite the numerous investigations that have been performed to date, the dynamic response of retaining walls is far from being well understood, and significant unresolved issues are still available. Damage to retaining structures can also be great due to an incomplete understanding of the complex backfill and soil/foundation interactions occurring during an earthquake. Evidence of a lack of understanding comes from post-earthquake investigations (notably in Kobe (Japanese Geotechnical Society 1998) and Chi-Chi (EERI 2001)) which have reported extensive damage on

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a number of retaining structures, which were thought to have been properly designed against seismic motions (Giarlelis and Mylonakis 2011). Furthermore, seismically induced retaining wall failures were reported in Japan (Bardet *et al.* 1995), in Alaska and Chile (Seed and Whitman 1970), and in California (Jennings 1971).

Previous analyses on seismic behavior of soil-retaining systems can be roughly divided into two major groups: (a) limit-state analyses, in which the wall is considered to displace and/or rotate adequately at the base to fully mobilize the shearing strength of the backfill, and (b) elastic analyses, in which the wall is considered to be fixed at the base, while the backfill material is considered to respond within the linearly elastic range of deformations. A representative of the first group is the well-known Mononobe-Okabe (M-O) method (Mononobe and Matsuo 1929, Okabe 1924) and its various variants (Seed and Whitman 1970, Richards and Elms 1979, Nadim and Whitman 1983), which have found widespread acceptance in codes (e.g., ATC 1981, Eurocode 8 1994). Representatives of the other group are the studies of Matsuo and Ohara (1960), Wood (1973, 1975), Arias *et al.* (1981), Veletsos and Younan (1994a, b, 1995, 1997) and Younan and Veletsos (2000).

The realistic seismic response of soil-wall system is not sufficiently captured in limit state analyses, because these are based on the assumption of constant acceleration in the backfill and ignore the wave propagation in the large soil medium. Therefore, many investigators have directed their attention and resorted to the elastic solutions in order to gain insight into the dynamic response of soil-wall system. The elastic response of this system was examined previously by Matsuo and Ohara (1960). However, the accuracy of their solution could not be confirmed, and they presented no numerical solutions. In a series of valuable studies, Wood (1973, 1975) provided analytical solutions and comprehensive numerical data for the response of a stratum of finite length excited uniformly along its base and its two vertical boundaries. Scott (1973) proposed a simple model for approximating the responses both of the semi-infinite stratum and of the bounded system. However, this model ignores the radiation damping capacity of the medium, and thus does not adequately describe the action of the system. Arias *et al.* (1981) used a simplified representation of the elastic medium and provided relatively simple analytical expressions for the wall pressures. Then, Veletsos and Younan (1994a, b) presented simple approximate expressions for the response of rigid walls retaining soil with a semi-infinite and uniform viscoelastic layer. In a next step, Veletsos and Younan (1995, 1997) and Younan and Veletsos (2000) extended their investigations and took into account the effect of wall flexibility on dynamic pressures and associated forces and moments. The accuracy of these solutions has been verified with finite element analyses carried out by Wu and Finn (1999) and Psarropoulos *et al.* (2005). Theodorakopoulos *et al.* (2001a, b) extended these solutions for investigation of the dynamic response of rigid walls retaining a semi-infinite, uniform, fully-saturated poroelastic backfill. In addition to these studies, Elgamal *et al.* (1996) described finite element simulations and dynamic full-scale tests to determine dynamic characteristics of a cantilever wall-backfill system. Madabhushi and Zeng (2007) investigated the seismic response of a cantilever wall with dry and saturated backfills. More recently, Giri (2011) used a pseudo-dynamic method to compute the distribution of seismic earth pressure on a rigid cantilever wall supporting dry cohesionless backfill, and concluded that backfill surface inclination affects significantly the magnitude of active earth pressure. Kontoe *et al.* (2012) examined the seismic response of a large system comprising a Lock chamber and three neighbouring water saving basins with their associated retaining walls and cut slopes. 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. Cakir and Livaoglu (2012) proposed a simplified analytical model for seismic analysis of backfill-water tank wall-fluid systems, and accomplished the model verification by comparing the predictions to results from finite element simulations. Cakir and Livaoglu (2013) also carried out numerical and experimental investigations on determination of modal characteristics of backfill-rectangular tank-fluid system, and found close agreement between theory and experiment. Khajehzadeh *et al.* (2013) introduced a new version of gravitational search algorithm based on opposition-based learning for optimum design of reinforced concrete cantilever retaining walls.

Literature investigation shows that most of the studies about retaining walls have primarily concentrated on the determination of earth pressure distributions. However, relatively little work has been conducted on the structural behavior of cantilever walls considering soil-structure interaction in three dimensional space. Thus, a new study that can take into consideration the backfill–structure-soil/foundation interaction effects in three dimensions is necessary for seismic analysis of cantilever walls. Furthermore, in codes (e.g., TEC 2007, IS-1893 2002, Eurocode 8 2003) about earth-retaining structures, there is no specific numerical method with regard to how the backfill and subsoil interactions can be taken into account, and the analysis of retaining walls are generally carried out by using pseudo-static approaches although these approaches do not entirely consider the interaction effects. The main goal of this study is to propose a three-dimensional FEM for seismic analysis of cantilever retaining walls, to provide verification of the FEM under fixed-base and elastic soil assumptions by comparing its predictions to results from proposed analytical formulation, and to further investigate the dynamic behavior of backfill-cantilever wall-soil/foundation interaction system.

2. Backfill-cantilever wall system considered

The scheme of the backfill-cantilever wall system investigated is shown in Fig. 1. It consists of a semi-infinite, uniform layer of elastic material that is free at its upper surface, is bonded to a rigid base, and is retained by a cantilever wall. The wall is considered to be fixed at the base. The heights of the wall and stratum are considered to be the same, and they are denoted by H. The properties of the wall are described by its thickness, mass density, moment of inertia, Young's modulus and Poisson's ratio. The backfill properties are regarded constant, and defined by mass density, shear modulus and Poisson's ratio. Furthermore, dry-cohesionless soil is considered in the analyses.

3. Numerical modeling

Computer-aided analyses of stresses, strains and displacements of soil structures have made remarkable advances in recent years. 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). Accordingly, the finite element methods have also become a valuable tool in geotechnical engineering to evaluate the behavior of complex structures. Fig. 2 shows the proposed FEM for the problem of cantilever retaining wall under investigation, which contains different aspects of the model. The finite element modelling and analyses were performed using

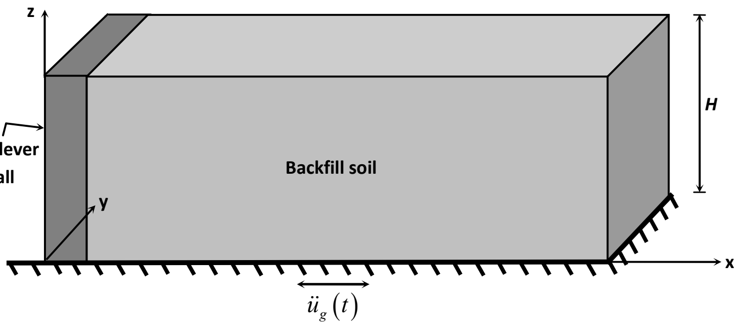


Fig. 1 Backfill-wall system considered

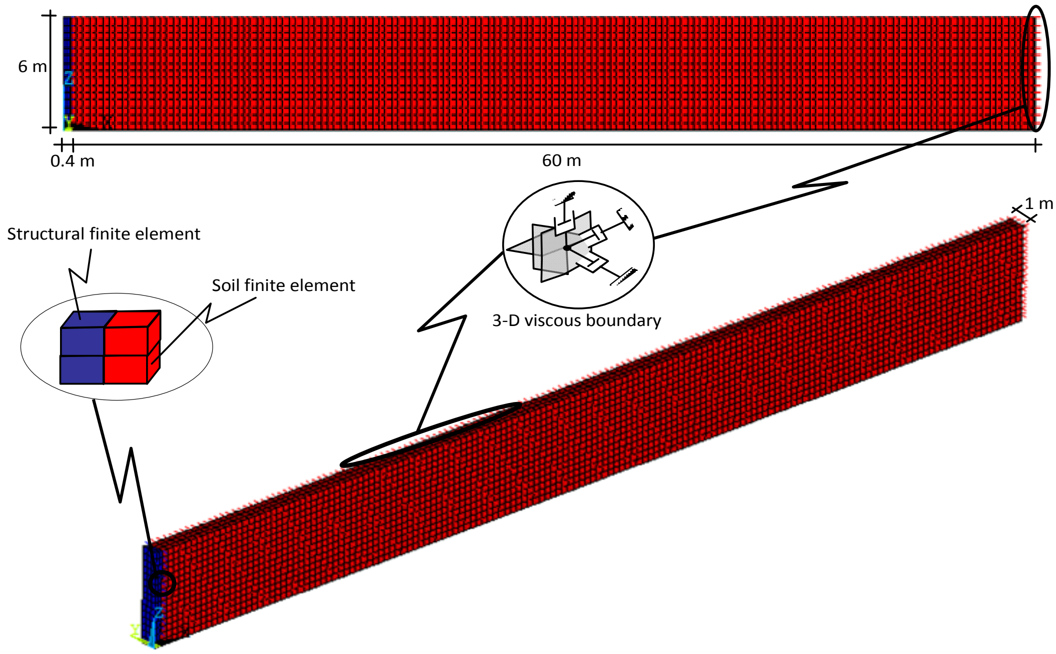


Fig. 2 The FEM for backfill-cantilever wall system

the general purpose structural analysis computer program ANSYS 10 (2006). It is clearly known from the literature that to simplify the soil-structure interaction analysis, three-dimensional problems are often modelled by considering a two-dimensional slice with the same material properties. This assumption, although convenient, is potentially dangerous for the following reasons. First, the specific radiation damping computed for the two-dimensional case overestimates the actual three-dimensional case for finite frequencies. Second, the contact area of a reasonably selected two-dimensional model will be larger than that of the three-dimensional case which will further increase radiation damping (Wolf 1994, Wolf and Song 2002). In this context, it is not possible to obtain a two-dimensional representation that will approximate both the dynamic stiffness and damping over a reasonable range of frequencies (Luco and Hadjian 1974), and because the damping is grossly overestimated, two-dimensional modelling of a three-dimensional case cannot be recommended for actual engineering applications (Wolf and Song 2002). Therefore, three-dimensional modelling of interaction system for a cantilever wall length of 1m is adopted in this study.

The cantilever wall itself is discretized by 3-D solid elements (SOLID65) defined by eight nodes having three translational degrees of freedom in each node in the finite element procedure. The discretization of the retained soil is also made by 3-D structural solid elements (SOLID185) 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 (1994a) 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 the verification of the FEM is provided by analytical formula, elaborate seismic analyses will be carried out later considering not only backfill and soil/foundation interactions but also the behavior of backfill-wall interface in the Section 6.

In many earthquake engineering and seismological problems, wave propagation investigation in a large soil medium is necessary, and the simulation of the infinite medium is extremely significant. The general approach of treating these problems 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. As an alternative to modelling very large soil volumes and to limit the model to a reasonable size, one possible approach is to just truncate the computational domain at some distance away and to impose appropriate boundary conditions. Such a boundary is called an artificial boundary. 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) and Kuhlemeyer and Lysmer (1973) suggest applying viscous tractions that must absorb reflected energy along the artificial boundary. The advantage of this approach lies in the fact that the applied stresses are frequency independent. This technique is widely used since it is easy to implement and gives satisfactory results for dilatational and shear waves. Later, different boundary models have been used and developed.

In this study, the viscous boundary model, which was successfully employed in the finite element modeling of the elevated and rectangular tanks carried out by Livaoglu and Dogangun (2007) and Livaoglu *et al.* (2011), 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 10 H, a value which is believed to approximate adequately the behavior of the semi-infinite layer (Veletsos and Younan 1994a, Psarropoulos *et al.* 2005). In this context, the dashpots were also placed 10 H away from the wall in three dimensions to improve the accuracy of the simulation where H is the height of the cantilever wall. The modal analyses were performed assuming elastic material responses in this study. Furthermore, it is worth mentioning that both the viscous boundary and the nonlinear properties of the material considered in the proposed models are for clear understanding of dynamic response of the system during time history analysis.

4. Analytical verification through the modal analysis

4.1 Deterministic analysis

Spring-dashpot-mass models with frequency-independent coefficients and with a few degrees of freedom can be constructed for practical use in many engineering problems. To demonstrate the obtainable accuracy of FEM proposed, as a practical example, a spring-dashpot-mass model to calculate the modal properties of cantilever wall retaining a semi-infinite soil layer on rigid base is addressed in Fig. 3. For the estimation of the stiffness and mass values for backfill soil, the method presented by Veletsos and Younan (1994a) was adopted. Furthermore, the mass of the cantilever wall is taken into account, and the system is represented by spring-dashpot-mass model with two degrees of freedom in this study while Veletsos and Younan had regarded the wall as massless. Spring-dashpot-mass system is connected to the wall at a height = 0.637 H where H is the height of the wall. To obtain a simplified model and to permit a comparative study, the approach is based on the simplifying assumption that complete bonding is assumed at the wall-soil interface. In the light of these explanations, the mathematical model and modal representation of the system are depicted in Fig. 3.

To perform the modal analysis, the concerned design parameters as stiffnesses and masses of the system, must be determined 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

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

where ρ is the mass density for medium, H is the height of both the wall and the soil layer, 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

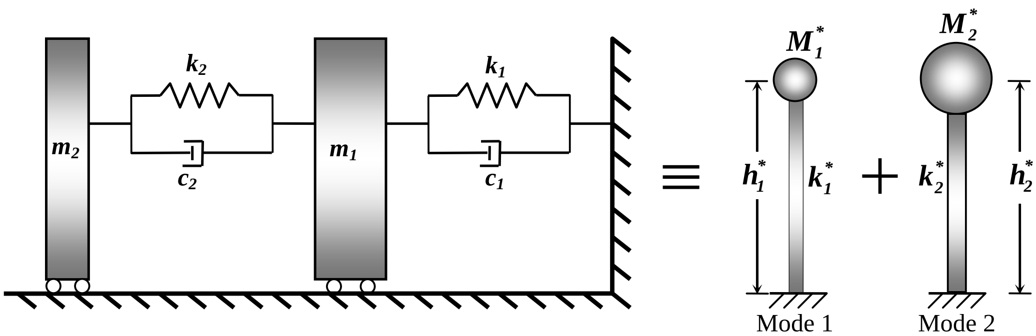


Fig. 3 Proposed mathematical model and modal representation of the system

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

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

The mass of the wall is represented by m_2, and the structural 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 wall modes, respectively.

The development of the simplified analytical solution may be derived from a physical interpretation of the solution to the differential equation. Considering dynamic equilibrium of the masses by using D'Alembert's principle, from the Fig. 3, 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 applied external forces. It is worth emphasizing that since the natural frequencies of the system in the analytical modal analysis are determined by using undamped free vibration equation of motions which can be derived from the preceding equations, any data on both the damping matrix and the external forces are not given herein. However, these data will be included later in the section on further investigations in which the seismic analyses of the system are carried out through the FEM developed.

The obtained equations can be solved by utilizing the modal analysis technique. The modal properties such as effective modal masses (M_1^*, M_2^*), heights (h_1^*, h_2^*) and stiffnesses (k_1^*, k_2^*) should be determined (see Fig. 3). These modal properties can be estimated using Eqs. (5) and (6) (Chopra 2007).

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

where

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

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

5. Numerical example and comparison of results

5.1 Deterministic analysis

To demonstrate the proposed methodologies, modal analyses of the system considered are carried out in this section. The comparison between results from the FEM simulation and the analytical formula is also provided in this section. A computer code was developed by the author

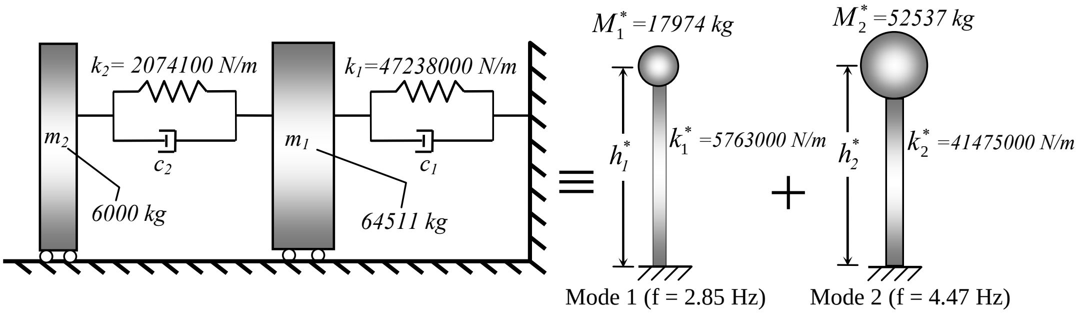
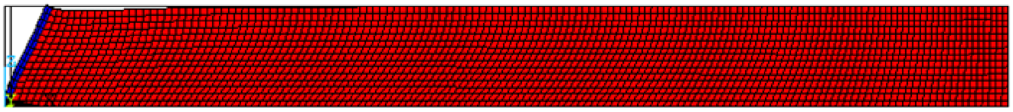
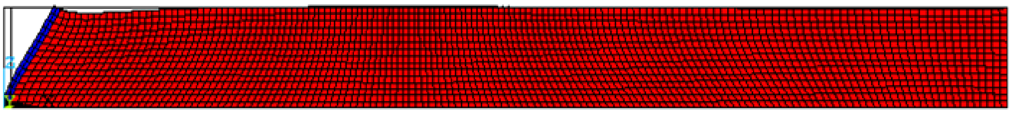


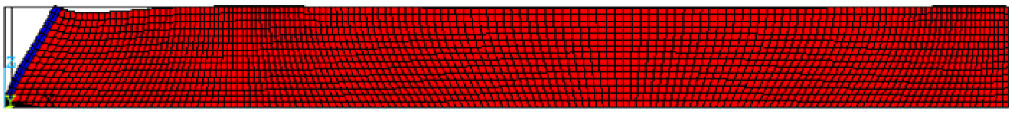
Fig. 4 The proposed simplified model and modal characteristics of backfill-cantilever retaining wall system



Mode 1 (f=4.45 Hz)



Mode 2 (f=4.89 Hz)



Mode 3 (f=5.61 Hz)

Fig. 5 The finite element mode shapes and corresponding frequencies of the system

for the modal analysis of the system considered to obtain modal solutions by means of analytical formula.

In the numerical example, a 6 m–high cantilever retaining wall with a constant thickness of 0.4 m is considered. As previously stated, the critical minimum distance from the face of the wall is taken as 10 H = 60 m. The Young's modulus, Poisson's ratio and unit weight of the concrete were considered as 28000 MPa, 0.2 and 25 kN/m^3, respectively. The Young's Modulus, Poisson's ratio and the unit weight of the soil were taken to be 50 MPa, 0.3 and 18 kN/m^3, respectively. Through using the proposed analytical model, the values of mass and stiffness, which are necessary to compute the natural frequencies of the interaction system, are presented in Fig. 4. The modal characteristics such as the effective modal masses, stiffnesses and modal frequencies can also be seen in Fig. 4.

As Fig. 4 demonstrates, the mode frequencies were computed as 2.85 and 4.47 Hz, respectively. It is worth stating here that the first and second modes represent the backfill and wall modes, respectively. 25% of the total effective mass is represented by the backfill mode, and 75% of it is represented by the wall mode. It means that the modes are sufficient to represent all system behavior.

Table 1 Comparison of analytical and numerical results

Mode categories	Mode descriptions	Modal frequencies (Hz)	
		Analytical model	FEM
Backfill	Backfill mode	2.85	---
Structure	First mode	4.47	4.45
	Second mode	---	4.89
	Third mode	---	5.61

Similarly, the modal characteristics of the same system considered may be determined by means of the proposed FEM. Fig. 5 depicts the mode shapes of the system, and only the first three vibration modes, which have the ability to represent all system behavior based on effective modal masses, were identified. The frequencies of the modes given in Fig. 5 were estimated as 4.45, 4.89 and 5.61 Hz.

A comparison between the results estimated by using both the three-dimensional FEM and the proposed analytical model is shown in Table 1. It is worth noting here that only the comparison of the modes related to structure is given because this study is mainly focused towards the cantilever wall behavior and modes related to the structure in accordance with the purpose of the study. In this connection, when a comparison is made for the first structural mode, it is seen that the FE result is in good agreement with the analytical value so that the mode frequency is computed as 4.47 Hz from the analytical model while the same quantity is estimated as 4.45 Hz from the numerical model. Indeed, this reflects successful estimation, the analytical verification provides strong support for the FEM, and this makes the model attractive for use in further investigations.

6. Further investigations

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

Rational modeling of the wall-backfill interaction requires utilizing special interface elements between the wall and adjacent soil. Thus, as a special interface element (COMBIN39), nonlinear spring is used between the backfill and wall allowing for the opening and closing of the gaps (i.e., debonding and bonding) to model backfill-wall interaction. 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 taken into account to simulate the behavior of backfill-cantilever wall interaction surface. Moreover, the idealization of complicated behavior of soil is often necessary to develop simple mathematical constitutive laws for practical applications. Of course other smooth surfaces have been proposed but because of 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. The relative simplicity of the Drucker-Prager model, which can reflect some characteristics of soil behavior, explains why this model is widely used. Therefore, elasto-plastic behavior of soil is described by well-known Drucker-Prager yield criteria (Fig. 6). The internal friction angle for the cohesionless soil is considered as $\phi = 30°$ 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. 7). In the FE procedure, Rayleigh damping is taken into consideration in the Newmark's direct step-by-step integration method (Newmark 1959). The damping values for both structure and soil are taken as 5%.

The results of seismic analyses are presented in two parts. In the first part, a detailed discussion on the effects of backfill interaction on seismic behavior of cantilever wall supported on a rigid foundation is described. In the second part, the combined effects of backfill and soil/foundation interactions on dynamic behavior of cantilever wall are discussed.

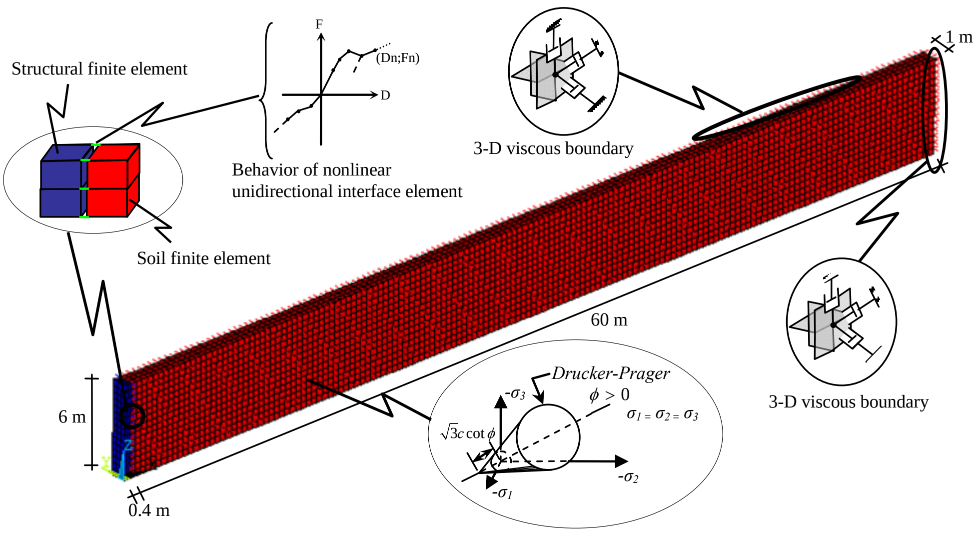


Fig. 6 FEM configuration of backfill-wall system considering interface elements and elasto-plastic soil behavior

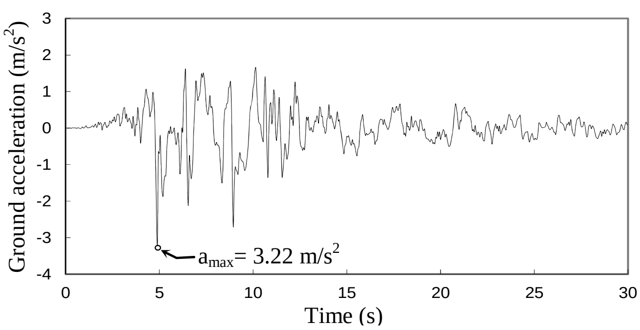


Fig. 7 Ground motion recorded at Yarimca Station during 1999 Kocaeli Earthquake

6.1 Seismic behavior of the cantilever retaining wall with rigid base

In this section, seismic analyses with variation of the backfill soil stiffness were conducted using the suggested FE procedure in order to evaluate the backfill interaction effects under fixed-base assumption. To illustrate the variation of the dynamic behavior of cantilever retaining wall depending on different soil conditions, five different backfill soil stiffnesses were considered in the analyses. The Poisson's ratio (v) and the unit weight (γ) of the cohesionless backfill soil are taken as 0.3 and 18 kN/m^3, respectively. Moreover, the Young modulus (E) for the backfill soil was taken into account as 25, 50, 100, 200 and 300 MPa, respectively.

Using the proposed FE procedure, it is possible to determine not only the lateral displacements along the height of the retaining wall but also the stress responses at critical sections of the wall. Table 2 reports the maximum top displacements and the stress responses at both the front and back faces of the cantilever wall and their occurrence times depending on the variation of backfill soil stiffness.

Table 2 indicates that the responses of the system are different from each other so that the maximum values of both lateral displacements and stresses changed with changing soil conditions. Furthermore, the computed time history results show that the maximum responses practically occur around the same time (t = 9.7-9.8 s) for all soil conditions. The effects of the backfill interaction on seismic response of cantilever wall were illustrated, and their implications were discussed comparatively below.

The maximum displacement responses along the height of the cantilever wall are shown in Fig. 8. The most important point arising from the comparisons is that the variation of backfill soil stiffness notably affects the displacement response of the system so that the displacement response of the cantilever wall increases while the backfill stiffness decreases.

To clarify the changes of the lateral displacement due to the soil stiffness variation, the deviations of the displacements in time are illustrated and compared in Fig. 9. As Fig. 9 demonstrates, while the maximum lateral top displacement was calculated as 0.0011 m at 9.7 s for

Table 2 Summary of the maximum dynamic responses and their occurrence times for fixed-base model

Maximum Responses	Young modulus for backfill soil									
	E = 25 MPa		E = 50 MPa		E = 100 MPa		E = 200 MPa		E = 300 MPa	
	t (s)	Value	t (s)	Value	t (s)	Value	t (s)	Value	t (s)	Value
u_t (m)	9.7	0.0032	9.75	0.0026	9.75	0.0020	9.8	0.0014	9.7	0.0011
S_{zb} (MPa)	9.7	-1.9543	9.7	-1.6654	9.7	-1.3673	9.7	-1.1146	9.7	-0.9843
S_{yb} (MPa)	9.7	-0.4436	9.7	-0.3780	9.7	-0.3103	9.7	-0.2529	9.7	-0.2234
S_{xb} (MPa)	9.7	-0.2649	9.7	-0.2255	9.7	-0.1849	9.7	-0.1506	9.7	-0.1328
S_{zf} (MPa)	9.7	1.9542	9.7	1.6651	9.7	1.3670	9.7	1.1144	9.7	0.9840
S_{yf} (MPa)	9.7	0.4436	9.7	0.3780	9.7	0.3103	9.7	0.2529	9.7	0.2233
S_{xf} (MPa)	9.7	0.2647	9.7	0.2254	9.7	0.1849	9.7	0.1505	9.7	0.1328

*u_t: Maximum lateral top displacement of cantilever wall; S_{zb}, S_{yb} and S_{xb}: Maximum 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}: Maximum stresses estimated on the front face of the cantilever wall in *z, y* and *x* directions, respectively

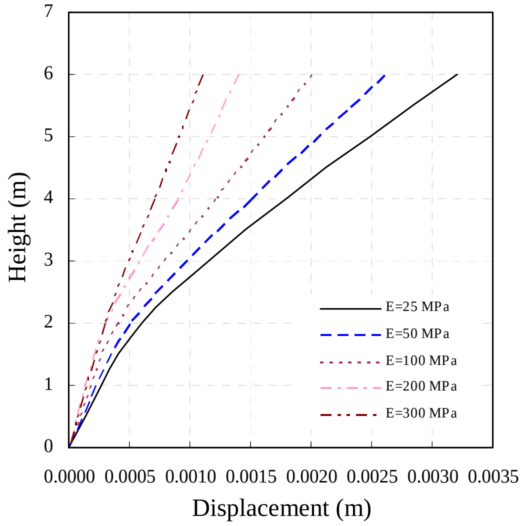


Fig. 8 Computed lateral displacements along the height of the cantilever retaining wall

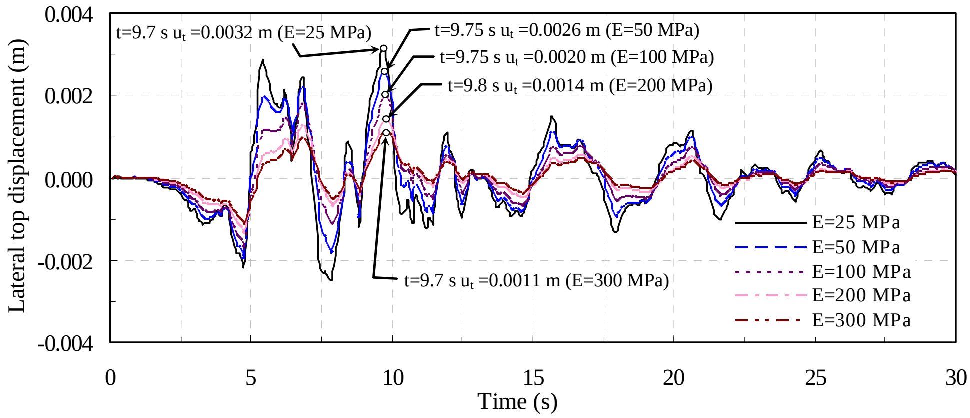


Fig. 9 Computed lateral top displacement time histories of cantilever retaining wall considering variation of backfill soil stiffness

E = 300 MPa, the same quantity was computed as 0.0032 m at 9.7 s for *E* = 25 MPa. Therefore, it can be stated that backfill-cantilever wall interaction affects the system behavior so that the dramatic increment in the displacement response is almost at a level of 190% between the *E* = 25 MPa and *E* = 300 MPa. If similar comparison is made between *E* = 100 MPa and *E* = 200 MPa, similar trend and backfill interaction effect can be observed so that the increase in the displacement response is almost 43%. These variations reveal a significant backfill interaction influence on the response, and it is clear that effect of backfill-structure interaction on the displacement response is highly dependent on the backfill properties. Consequently, these comparisons confirmed that the exclusion of the accurate backfill properties may cause underestimation or overestimation of the displacement response, and this, in turn, fairly affects the

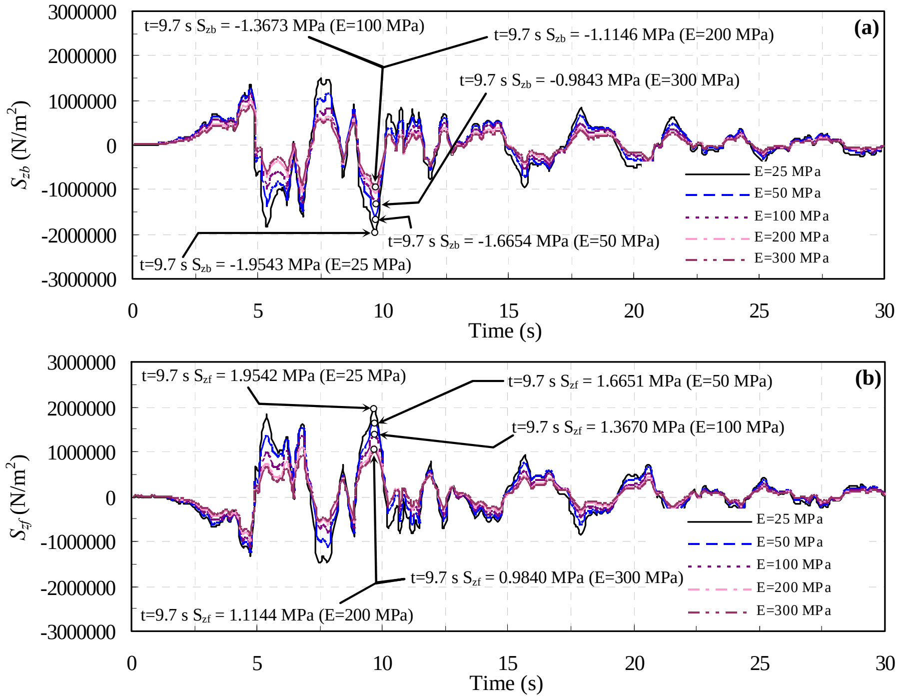


Fig. 10 Stress time history responses in *z* direction at (a) back face; (b) front face of the wall

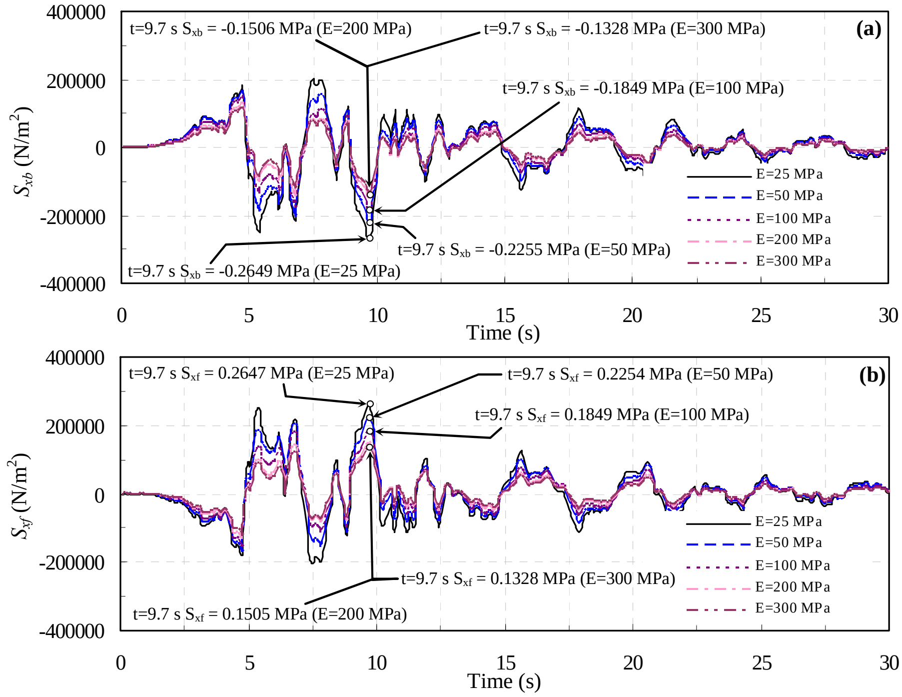


Fig. 11 Stress time history responses in x direction at (a) back face, (b) front face of the wall

design process due to the displacement sensitivity of cantilever retaining walls.

The estimated stress responses and their variations in time at the back and the front faces of the cantilever retaining wall are also given in this section. As previously stated, the maximum stress responses and their occurrence times are summarized in Table 2. The peak responses of the stresses in the wall, as anticipated, occurred at the base. Here, it has to be stated that because all results of the analyzed models cannot be illustrated, some comparisons were selected to describe the system behavior. In this context, the comparisons of stress time history responses in z and x directions for both back and front faces of the cantilever wall are shown in Figs. 10-11, respectively. As these figures depict, the maximum stress responses obtained at the critical sections of the wall generally tend to decrease as an absolute value with increasing of the backfill soil stiffness. For example, at the back face of the wall in z direction, while the peak stress, as compression, has the value of 1.9543 MPa for E = 25 MPa, it is calculated as 0.9843 MPa for E = 300 MPa. This reflects a decrement of about 50% in stress value due to the variation of backfill soil stiffness. If similar comparisons are made in x direction at the front face of the cantilever wall, the same trend and backfill interaction effect can be observed so that the maximum value of stress is calculated as 0.1328 MPa for E = 300 MPa while the same quantity is computed as 0.2647 MPa for E = 25 MPa, and the increase in the stress response is almost 100%. These results obtained from the time history diagrams show that the responses due to the variation of backfill soil properties are highly magnified. In addition, the deviations exhibited in Figs. 10-11 show that the responses are almost coincided for front and back faces of the wall in terms of the stress amplitude for the same backfill soil conditions. The only difference is that while the estimated stresses are obtained as tension at the front face, the stresses are acquired as compression at the back face of the wall. As a result, it is found that the effect of backfill interaction is significant on the structural response of soil-wall systems, and may cause a considerable increase in time domain peak response values.

6.2 Seismic behavior of the cantilever retaining wall with flexible foundation

As stated before, in order to evaluate the effects of deformability of foundation soil on seismic response of cantilever wall, additional FE model with flexible foundation boundary condition is investigated in this study. The soil medium on which a structure is built may interact with the structure during seismic action. Furthermore, dynamic soil-structure interaction can modify remarkably the stresses and displacements of the structural system from the values that could have been developed if the structure were built on a rigid foundation. The significant features that distinguish the dynamic soil-structure interaction system from other general dynamic structural systems are the unbounded nature and the nonlinear behavior of the soil medium. Therefore, the problem depicted in Fig. 12 reveals a complex phenomenon that incorporates both the backfill and soil/foundation interaction effects.

The corresponding finite element mesh of the backfill-cantilever wall-soil/foundation interaction system is shown in Fig. 13. In addition to the modeling of the superstructure, the soil/foundation system is modeled with 3-D structural solid elements (SOLID185), and the artificial viscous boundaries have been placed in three dimensions on the boundaries of soil/foundation media.

To account for the effect of soil-structure interaction and to determine the overall response of the cantilever retaining wall supported on flexible foundation to the dynamic excitation, five different soil/foundation stiffnesses were used in the analyses. The Poisson's ratio (ν) and the unit

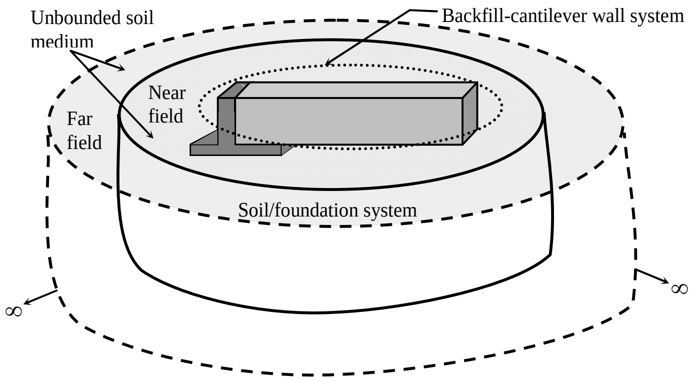


Fig. 12 Schematic representation of the problem investigated for dynamic backfill-structure-soil/foundation interaction

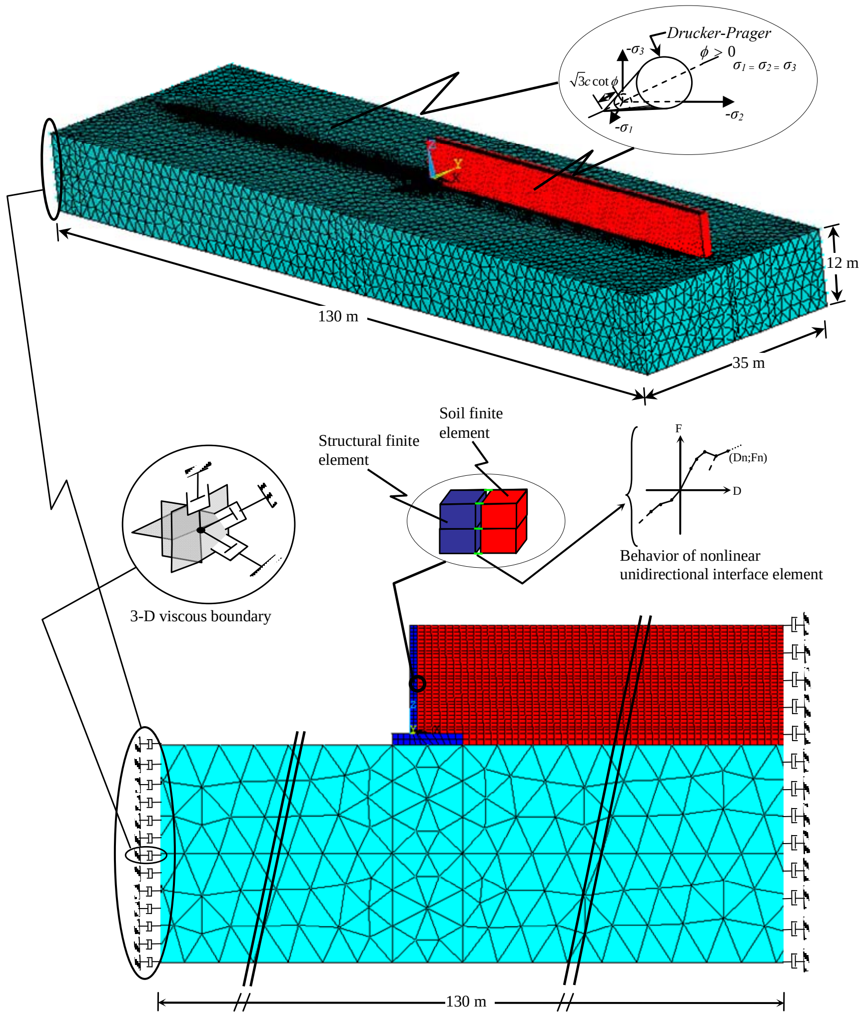


Fig. 13 FEM of backfill-cantilever wall-soil/foundation interaction system

weight (γ) of the foundation soil are taken as 0.3 and 18 kN/m^3, respectively. Furthermore, the Young modulus (E) for the foundation soil was taken into account as 75, 150, 300, 500 and 750 MPa, respectively. Utilizing the proposed FEM, the time history dynamic analyses were employed to compute the probable seismic behavior of the reinforced concrete cantilever retaining wall. Table 3 summarizes the calculated peak responses and the corresponding times where the maximum lateral top displacements and stresses occurred.

Table 3 shows that effect of soil-structure interaction is significant on the structural response of the cantilever retaining wall so that the peak responses are different from each other depending on the variation of soil/foundation conditions. Another sign of the soil-structure interaction influence on the response is that the occurrence times of response changed with changing soil stiffness. The

Table 3 Summary of the maximum dynamic responses and their occurrence times for soil / foundation interaction model

Maximum Responses	Young Modulus for foundation soil									
	E = 75 MPa		E = 150 MPa		E = 300 MPa		E = 500 MPa		E = 750 MPa	
	t (s)	Value	t (s)	Value	t (s)	Value	t (s)	Value	t (s)	Value
u_t (m)	4.8	0.0441	4.8	0.0226	4.75	0.0103	9.75	-0.0057	9.75	-0.0033
S_{zb} (MPa)	5.35	8.3560	5.4	6.4711	9.7	4.1748	9.7	2.6299	9.7	1.5944
S_{yb} (MPa)	5.35	1.1073	5.35	0.8094	9.7	0.5165	9.7	0.3228	9.7	0.1943
S_{xb} (MPa)	5.35	3.3999	9.7	2.4239	9.7	1.5457	9.7	0.9546	9.7	0.5644
S_{zf} (MPa)	5.35	-8.4602	5.4	-6.5508	9.7	-4.2306	9.7	-2.6657	9.7	-1.6166
S_{yf} (MPa)	5.35	-0.6532	5.35	-0.4891	9.7	-0.3104	9.7	-0.1957	9.7	-0.1192
S_{xf} (MPa)	5.35	-0.5388	5.35	-0.4443	5.35	-0.2939	5.3	-0.2026	5.35	-0.1307

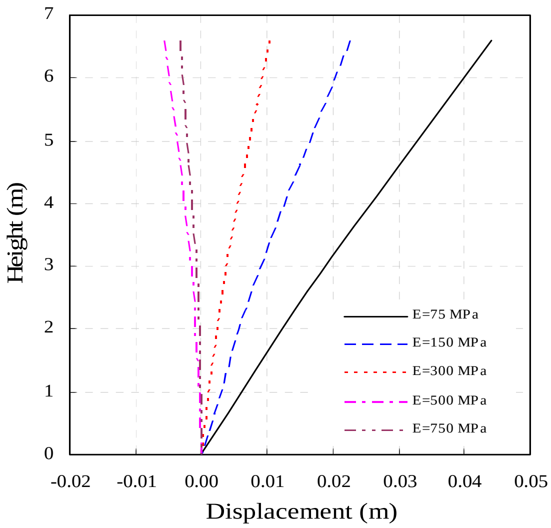


Fig. 14 Comparison of the lateral displacements along the height of the cantilever retaining wall considering soil/foundation interaction effects

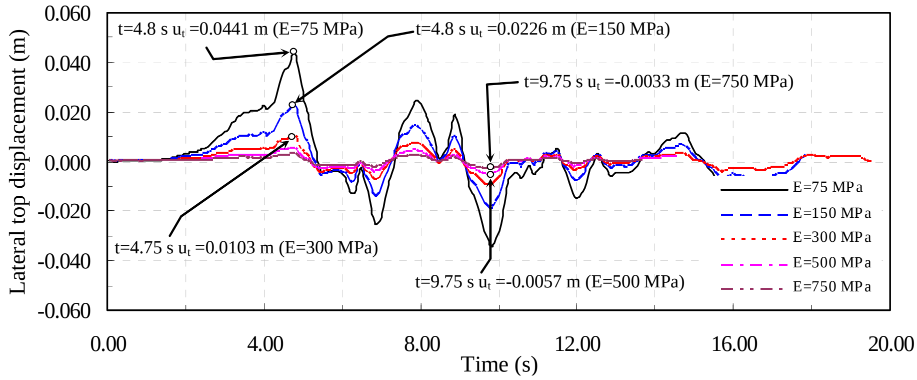


Fig. 15 Calculated lateral top displacement time histories of cantilever retaining wall considering soil/foundation interaction effects

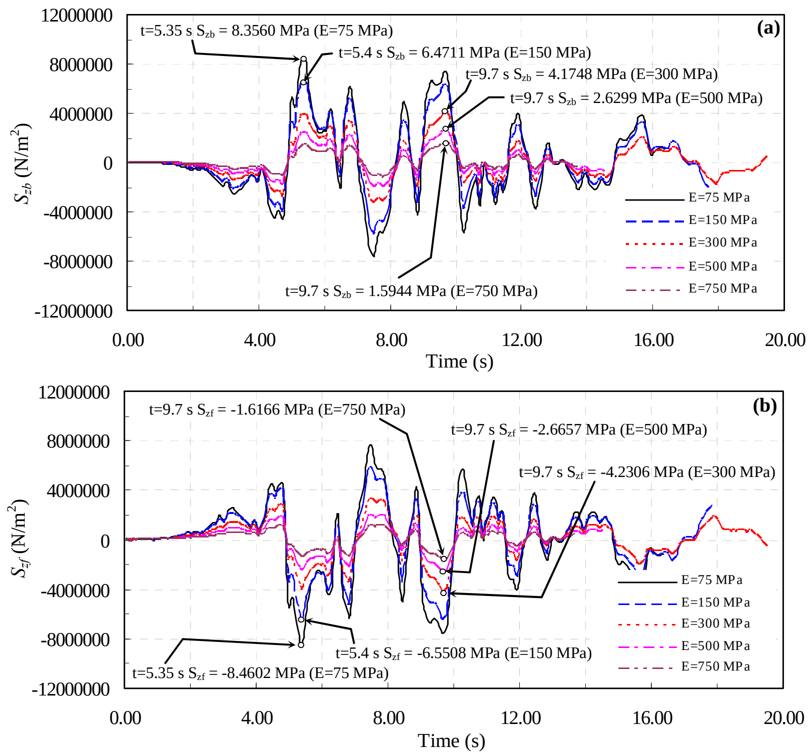


Fig. 16 Stress time history responses in z direction at (a) back face; (b) front face of the wall considering soil/foundation interaction effects

effects of the soil-structure interaction on seismic response of cantilever wall were illustrated, andtheir implications were also discussed comprehensively below.

A comparison among the heightwise variations of lateral displacements of cantilever wall under different soil/foundation conditions is presented in Fig. 14. It is worth emphasizing here that these displacements represent the relative lateral displacements of the wall with respect to the ground. Effects of soil-structure interaction on the displacement response of the wall are clearly observed from this figure, and it is clear that the structural response is highly dependent on the soil/foundation stiffness variation so that the displacements decrease with increasing soil stiffness.

To clarify the changes of the lateral top displacement values due to the flexible foundation conditions, the deviations of the displacements in time are illustrated and compared in Fig. 15. As Fig. 15 depicts, the response amplification has occurred in different soil/foundation conditions. For example, while the maximum lateral top displacement was estimated as 0.0103 m for E = 300 MPa, the same quantities were computed as 0.0226 m and 0.0441 m for E = 150 MPa and E = 75 MPa, respectively. It is clear that soil-structure interaction has led to the dramatic increments of about 328% and 120% in peak displacement responses for E = 75 MPa and E = 150 MPa in comparison with E = 300 MPa, respectively. When similar comparisons are made for the other conditions, similar tendency and soil-structure interaction effects can be observed. Therefore, it is concluded that the response amplification or reduction pattern due to the deformable foundation is highly dependent on the stiffness of soil/foundation system, and the time history diagrams describe different behaviors of the structure.

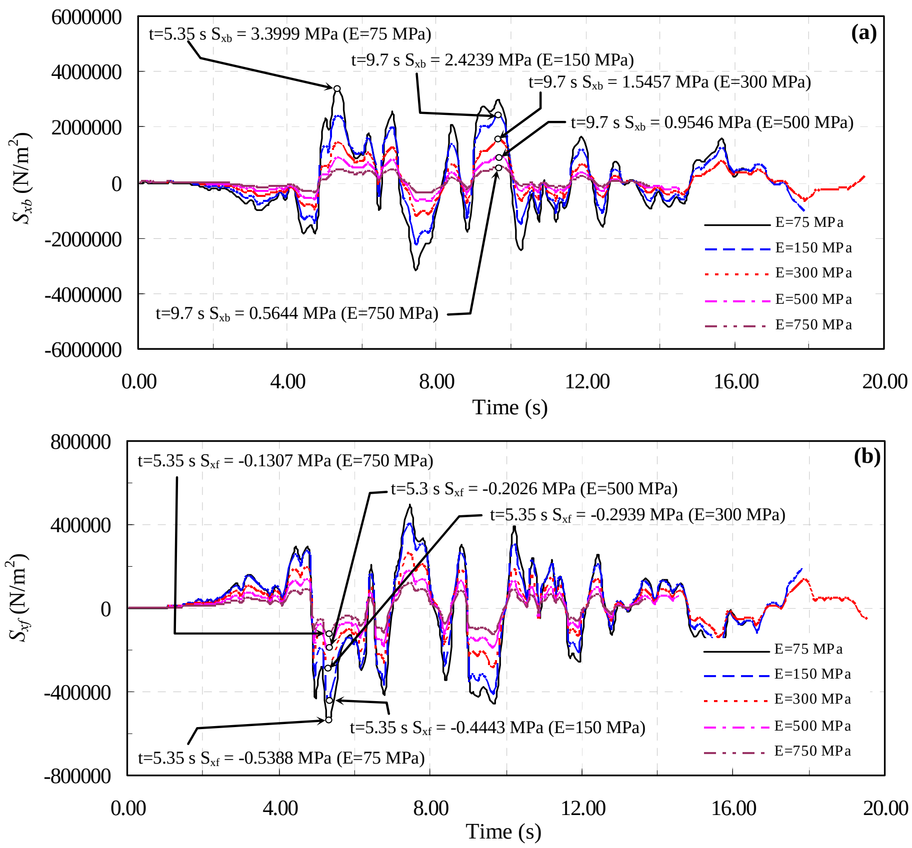


Fig. 17 Stress time history responses in x direction at (a) back face; (b) front face of the wall considering soil/foundation interaction effects

Similarly, the transient stresses for cantilever wall due to the horizontal excitation can be compared with each other to introduce the soil-structure interaction effects. Again, it has to be said here that since all results obtained from the finite element models cannot be illustrated, some comparisons were selected to define the system behavior. Therefore, the time history diagrams of stress responses in z direction for both back and front faces of the cantilever wall are presented in Fig. 16. As Fig. 16 demonstrates, the maximum stress responses due to soil-structure interaction are highly magnified, and the responses generally tend to increase with decreasing of the soil/foundation stiffness. For example, at the back face of the wall in the z direction, while the peak stress, as tension, has the value of 1.5944 MPa for E = 750 MPa, it is estimated as 8.3560 MPa for E = 75 MPa. This reveals an increment of about 424% in stress value due to the decrement of soil/foundation stiffness. Moreover, the deviations given in Fig. 16 indicate that the responses are almost coincided for front and back faces of the wall in terms of the stress amplitude for the same foundation soil conditions. The only difference is that while the estimated stresses are obtained as compression at the front face, the stresses are calculated as tension at the back face of the wall. For example, while the peak stress has a value of 2.6299 MPa at the back face, it has a value of -2.6657 MPa at the front face for E = 500 Mpa.

It is clearly known from the literature that the tensile strength of concrete is relatively low, and a good approximation for the tensile strength is that it is approximately 10% of the compressive strength. Such a tension amplitude, for concrete, may cause to crack on the tension side of the wall section. It means that the tension cracks may grow at the critical sections on the back face of the cantilever wall because, for example, the tensile stress is calculated as 8.3560 MPa in case of E = 75 MPa. The bearing capacity of concrete is, however, fully adequate in terms of the compressive strength because the C20 concrete class which has 20 MPa compressive strength is generally used for such a type of structure.

If the same comparisons are made in the x direction as seen in Fig. 17, effects of soil-structure interaction can be clearly observed. For instance, the changing of the Young's Modulus of soil from 750 MPa to 75 MPa causes a stress increment of about 502% at the back face of the wall. Furthermore, the maximum responses at back and front faces of the wall differ remarkably. For example, in case of E = 150 MPa, while the stress value is estimated as 2.4239 MPa for back face, the same quantity is calculated as 0.4443 MPa for front face, and this variation also reflects a significant soil-structure interaction effects.

It is found that the soil-structure interaction can play an important role on the behavior because of the response amplification/reduction effect of it.

7. Conclusions

The paper utilizes a three-dimensional finite element method to study the effects of not only backfill but also soil/foundation interactions on seismic behavior of cantilever retaining wall. Modelling the backfill soil as elastic and homogeneous material, the numerical results are shown to have excellent agreement with the proposed analytical solution for fixed-base assumption. The method is then extended to further investigate parametrically the effects of aforementioned interactions, in which both the elasto-plastic soil behavior is taken into account by the Drucker-Prager yield criterion and the interface behavior is considered by using interface elements between the wall and soil to allow for debonding. In spite of the complexities associated with accounting for interaction effects, the model can provide a reasonable quantification of the design requirements and may be used as a reliable tool to perform a broad suite of parametric studies.

Effects of backfill and underlying ground on the dynamic response of the system are investigated by comparing the results among five different soil stiffnesses. The results are presented in terms of the lateral displacements and stresses in the wall obtained from nonlinear time history analyses. It is obvious that the seismic response of cantilever retaining wall is significantly affected from the soil-structure interaction, and it is found to be very sensitive to changes in soil stiffness. Therefore, the exclusion of the accurate soil properties may cause underestimation or overestimation of the response, and this, in turn, may lead to unsafe seismic design of R/C cantilever retaining walls. In other words, cantilever retaining walls must not be designed only according to the typical projects since the local soil conditions may change the system behavior.

This study shows that the dynamic behavior of cantilever retaining walls may change depending on backfill and soil/foundation interactions which should be taken into account in current codes of practice. Accordingly, it is recommended that more numerical examples should be analyzed and assessed for different soil types and foundation conditions so that the results from the procedure presented here can be generalized.

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