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A comparison of the effect of SSI on base isolation systems and fixed-base structures for soft soil

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Abstract. This study investigated the effect of soil–structure interaction (SSI) on the response of base-isolated buildings. Seismic isolation can significantly reduce the induced seismic loads on a relatively stiff building by introducing flexibility at its base and avoiding resonance with the predominant frequencies of common earthquakes. To provide a better understanding of the movement behavior of multi-story structures during earthquakes, this study analyzed the dynamic behavior of multi-story structures with high damping rubber bearing (HDRB) behavior base isolation systems that were built on soft soil. Various models were developed, both with and without consideration of SSI. Both the superstructure and soil were modeled linearly, but HDRB was modeled non-linearly. The behavior of the specified models under dynamic loads was analyzed using SAP2000 computer software. Erzincan, Marmara and Duzce Earthquakes were chosen as the ground motions. Following the analysis, the displacements, base shear forces, top story accelerations, base level accelerations, periods and maximum internal forces were compared in isolated and fixed-base structures with and without SSI. The results indicate that soil-structure interaction is an important factor (in terms of earthquakes) to consider in the selection of an appropriate isolator for base-isolated structures on soft soils.

Keywords: HDRB; soil structure interaction; earthquake; soft soil; time history analysis

1. Introduction

The seismic demands on the superstructure are reduced through the isolator’s natural action of period elongation, increased damping and energy dissipation. One of the goals of seismic isolation is to shift the fundamental frequency of a structure to a value much lower than both the fixed-base frequency of the structure and the predominant frequency of the earthquake. This goal is achieved because of the low horizontal stiffness of the isolation systems. Another reason for using an isolation system is to provide additional energy dissipation, thereby, reducing the acceleration transmitted into the superstructure. Modifying the seismic structural response of a structure through the application of base isolation has been the subject of extensive research (Naeim and Kelly 1999, Skinner *et al*. 1993, Ibrahim 2008).

In dynamic SSI problems, discrete formulations such as the finite element method (FEM) (Wolf and Song 1996, Medina and Taylor 1983) or boundary element method (Mengi *et al*. 1994,

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Brebbia and Connor 1989) are commonly used. The finite element formulation with transmitting boundaries provides approximate results because some of the wave energy is trapped in the closed region. The usual method for treating SSI problems is to divide the unbounded medium into two regions: (1) near field and (2) far field. The near field is then discredited using standard finite elements, and the far field is discredited using either infinite elements (Zhang and Zhao 1987, Yerli *et al.*1999, Zhao 2010a) or artificial boundaries.

The two main methods used in SSI analysis are the direct method and substructure method. In the direct method, the response of the soil and structure is determined simultaneously by analyzing the idealized soil–structure system in a single step (Jaya and Meher 2002). The soil containing the structure is modeled up to the artificial boundary.

Many researchers have estimated the SSI effect on the elastic response of structures (Chopra and Gutierrez 1974, Wong and Luco 1976, Idriss *et al*. 1979). After the 1980s, SSI was studied thoroughly using impressive developments in numerical methods (Gazetas 1991, Wolf 1994). A detailed discussion on SSI effects and analysis techniques is presented by Johnson (2003). The soil–structure system is certainly more flexible than the commonly assumed fixed-base model. Hence, SSI effects have been reported in the literature as natural period elongation and added composite damping (Crouse and McGuire 2001).

Previous studies include Constantinou and Kneifati (1986), which investigated the effect of SSI on the dynamic characteristics of a base-isolated structure. Tsai *et al*. (2004) proposed that the soil compliance and damping should be taken into account in the analysis of base-isolated buildings. These findings are based on numerical analyses of FPS-isolated buildings, and they revealed that SSI results in a larger displacement and larger shear forces in some sections of the structure. Tian and Li (2008) investigated the dynamic responses of a multi-story building with and without a sliding base-isolation device to ground shock induced by an in-tunnel explosion. Spyrakos *et al*. (2009a) investigated the effect of SSI on the response of base-isolated buildings. An equivalent fixed-base system is developed that accounts for soil compliance and damping characteristics of the base-isolated building.

Although the viscous damping boundary presented more than 40 years ago is easy to implement in finite element code, this boundary cannot be used to simulate both the real geometry and the wave propagation behavior in the far field of an infinite domain. To overcome this fatal weakness, as demonstrated by the first monograph in the world (Zhao 2009), dynamic and transient infinite elements have been developed to solve wave propagation problems and a broad range of scientific and engineering problems (Zhao 2010a, b). In particular, Zhao *et al*. (1992, 1993) were the first to establish the coupled method of finite and dynamic infinite elements for solving wave scattering problems associated with many real scientific and engineering problems involving semi-infinite and infinite domains such as: (i) dynamic concrete gravity dam-foundation interaction and dynamic embankment dam-foundation interaction problems during earthquakes (Zhao *et al*. 1993, 1995); (ii) seismic free field distributions along the surfaces of natural canyons (Zhao and Valliappan 1993b, c); (iii) dynamic interactions between three-dimensional framed structures and their foundations (Zhao and Valliappan 1993d); and (iv) dynamic interactions between concrete retaining walls and their foundations (Zhao and Xu 1994). In addition, Zhao and Valliappan (1993e, f, 1994) were the first to develop the coupled method of finite and transient infinite elements for solving transient seepage flow, heat transfer and mass transport problems involving semi-infinite and infinite domains.

This study examines the effect of soil-structure interaction on base isolation systems. To this end, HDRB is used to analyze dynamic behavior of multi-story structures with an isolated base.

An isolator with three different lateral stiffnesses was compared with a “non-isolated” model to analyze appropriate isolator options. When developing these models, analyses are made by both considering and neglecting soil-structure interaction. The superstructure and base are modeled linearly and the HDRB is modeled nonlinearly because the superstructure is known to behave linearly in systems with a base isolation system. The behaviors of the specified models under dynamic loads are analyzed using SAP2000 computer software. The results are compared under three different types of earthquake effects, showing that including the soil-structure interaction improves the design of the base-isolated structures.

2. High Damping Rubber Bearings (HDRBs)

In 1982, the “Malaysian Rubber Producers Research Union” developed a new material by increasing the damping characteristics of natural rubber bearings. Damping was increased by adding carbon blocks and materials such as resin to the natural rubber bearings. When the damping rate reached 100%, the strain rate was between 10% and 20%. For shear strain rates higher than 20%, the material exhibited non-linear behavior. In this case, structures show high stiffness and damping properties against the effects of less intensive earthquakes and/or wind. For greater shear strain rates, the energy absorption rate also increases because the rubber is crystalline. The damping of the HDRB is somewhere between viscous damping and hysteric damping (Karabork 2011). As in layered rubber bearings, HDRBs extend the periods of the structural systems by means of lateral motions. These bearings also have a damping property, which helps them dissipate the energy of an earthquake. The rubber – steel composite bearing produced with this method is illustrated in Fig. 1.

Steel plates increase the vertical load bearing capacity of the HDRB; therefore, the vertical stiffness becomes greater than the horizontal stiffness. The horizontal stiffness depends on the number and thickness of the rubber plates; increasing the number of plates decreases the stiffness. Buckling may occur at increased height; therefore, height should be limited (Kelly 1999, Providakis 2008, Karabork 2011).

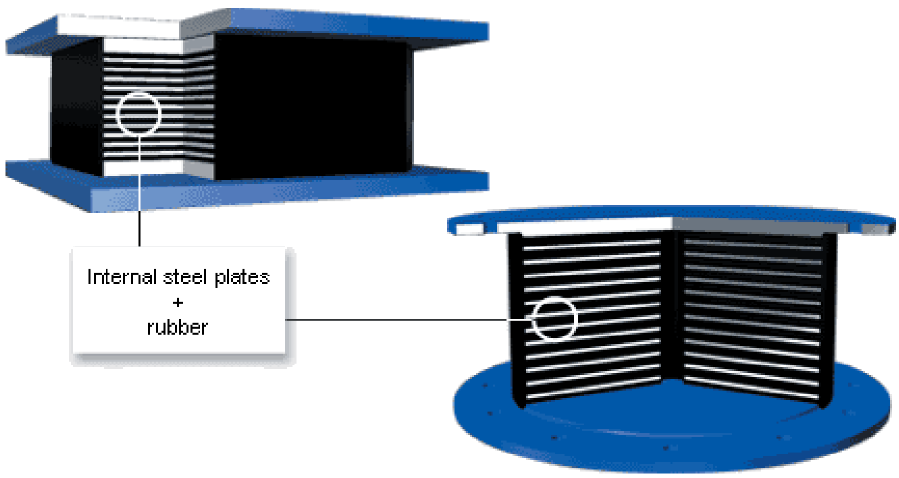


Fig. 1 High damping rubber bearings

Although the superstructure shows elastic behavior, the HDRB placed underneath the structure displays non-linear behavior. An analysis of multi-story structures can be carried out using the Two-Degrees-of–Freedom (2-DOF) method.

2.1 Numerical Model for the HDRB

The forces along the orthogonal directions mobilized during the motion of elastomeric bearings are described by Tsopelas *et al*. (1994) in Eq. (1)

$$F_x = \alpha \frac{F^y}{Y} U_x(t+\Delta t) + (1 \quad \alpha) F^y Z_x, \quad F_y = \alpha \frac{F^y}{Y} U_y(t+\Delta t) + (1 \quad \alpha) F^y Z_y \tag{1}$$

where α is the post-yielding to pre-yielding stiffness ratio, F_y is the yield force and Y is the yield displacement, as shown in Fig. 2. Z_x and Z_y are dimensionless variables governed by the following system of differential equations, proposed by Park *et al*. (1986).

$$\begin{Bmatrix} \dot{Z}_x Y \\ \dot{Z}_y Y \end{Bmatrix} = \begin{Bmatrix} A\dot{U}_x \\ A\dot{U}_y \end{Bmatrix} \quad \begin{bmatrix} Z_x^2 \left(\gamma \; Sgn\left(\dot{U}_x Z_x\right) + \beta\right) & Z_x Z_y \left(\gamma \; Sgn\left(\dot{U}_y Z_y\right) + \beta\right) \\ Z_x Z_y \left(\gamma \; Sgn\left(\dot{U}_x Z_x\right) + \beta\right) & Z_y^2 \left(\gamma \; Sgn\left(\dot{U}_y Z_y\right) + \beta\right) \end{bmatrix} \begin{Bmatrix} \dot{U}_x \\ \dot{U}_y \end{Bmatrix} \tag{2}$$

where A, γ and β are dimensionless qualities that control the shape of the hysteretic loop.

$$Z_{i+1} = Z_i + \left[\frac{1}{6}\left(K_1 + 2K_2 + 2K_3 + K_4\right)\right]\Delta T \tag{3}$$

where $K_1 = f(T_i, Z_i)$, $K_2 = f\left(T_i + \frac{1}{2}\Delta T, Z_i + \frac{1}{2}K_1 \Delta T\right)$, $K_3 = f\left(T_i + \frac{1}{2}\Delta T, Z_i + \frac{1}{2}K_2 \Delta T\right)$, $K_4 = f\left(T_i + \Delta T, Z_i + K_3 \Delta T\right)$.

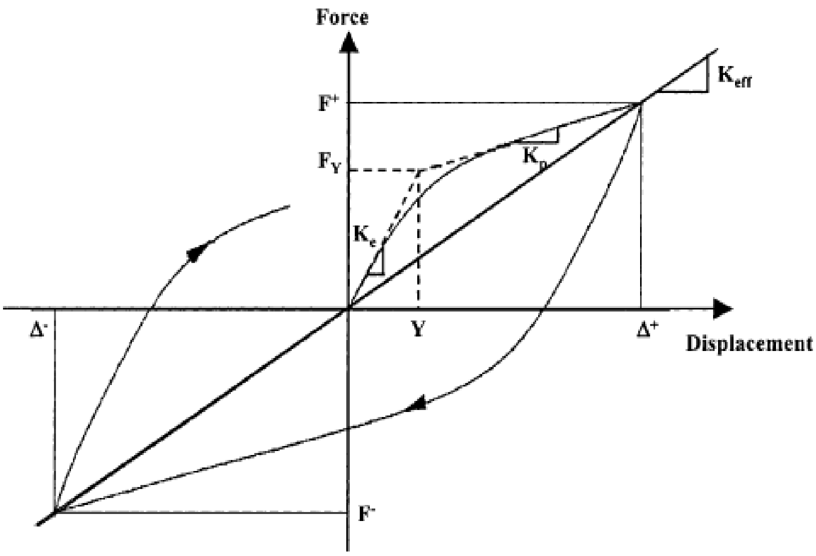


Fig. 2 Force-displacement diagram for an isolation system

3. Dynamic soil-structure interaction

Most previous studies assume a fixed support between the structure and the soil and that neither the structure nor the soils affect one another. However, in reality, the soil and structure move independently of each other. An analysis should consider SSI, particularly in the construction of heavy and rigid structures. Soil environments may have undesired behavioral effects on structures, depending on the properties of the seismic waves. Therefore, the properties of the soil on which the structure is built are of great importance.

In a system exposed to a dynamic load, damping ratio (C) and stiffness (K) are the two main parameters that affect the displacement of the system. Thus, in the SSI analysis, modeling should take into account not only the elasticity module, Poisson ratio, density and shear wave velocity of the soil but also the damping ratio and stiffness values. Including these parameters is important when selecting the mathematical model to be used for the analysis. The studies conducted to date have attempted to simplify the properties of the soil, which is regarded as an unlimited environment, by idealized methods. Determining the dynamic properties of the soil-structure for a common system by analytical methods is not easy. Such properties can be analyzed using numerical methods (Kramer 2003).

Consideration of SSI is usually a time-consuming and high-cost process. The first step is to decide whether to include the interaction in the calculations. The effects of the assumption that a rigid connection exists between the structure and the soil (as in fixed-base structure systems) on the results should be predictable. Some studies (Karabork and Dogus 2005, Spyrakos *et al*. 2009b) have suggested that SSI can be considered for Eq. (4) and, only in this case can the interaction result in significant changes in the behavior of the structure.

$$\frac{V_s}{(fh)} < 20 \tag{4}$$

where V_s is the shear wave velocity of the soil, "f" represents the free vibration frequency calculated on the basis of the assumption that a rigid connection exists between the structure and soil and "h" represents the height of the structure.

Analysis and modeling of the dynamic SSI is started by using FEM. The direct method and substructure method are the two main solution methods for SSI. In the direct method, the structure and the soil beneath it are modeled together. In the substructure method, the soil-structure system is divided into two substructures, unlimited soil and nonlinear soil around the structure (Wegner 2005, Zhao 2009).

In the direct method, the use of a well-established structure dynamic algorithm can solve SSI problems with the time history method (However, the radiation effect is not considered at each step in this method. Instead, these effects are considered in the frequency history in a multi-step method.). The direct method models and analyzes the soil-base-structure system in one step. As shown in Fig. 3, open field input motions are defined at the base and sides of the model.

The response of the system affected by this motion can be formulated as

$$[M]\{\ddot{u}\} + [K^*]\{u\} = -[M]\{\ddot{u}_{ff}(t)\} \tag{5}$$

where $\ddot{u}_{ff}(t)$ is the open field accelerations defined at the limit nodal points. The use of the direct method in SSI problems is only possible using a computer program that gives equal importance to both the soil and structure behavior simultaneously.

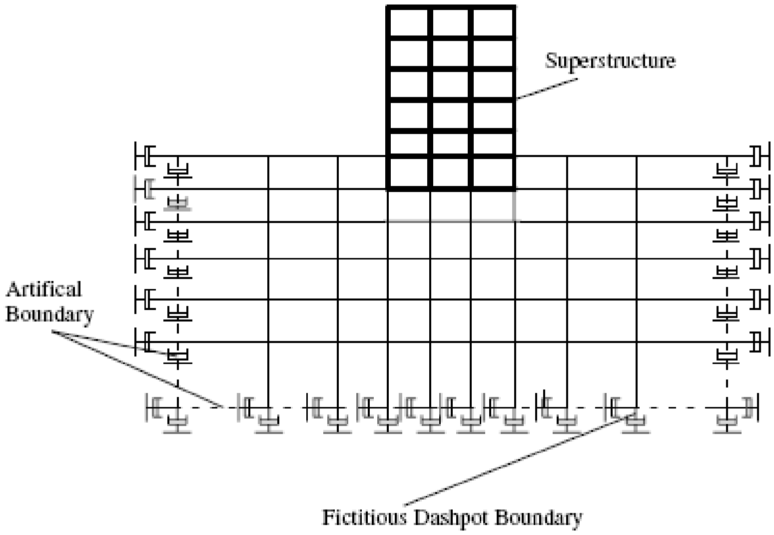


Fig. 3 Direct method for SSI analysis

Note that two types of wave propagation problems are commonly encountered in engineering practice (Zhao 2010a, b). One type of problem is the wave radiation problem (the machine foundation vibration is an example of this problem) (Zhao 2010a); the other problem is the wave scattering problem (the seismic response of a structure is an example of this problem) (Zhao 2010b). Because the viscous damping boundary model cannot be used to simulate both the real geometry and the wave propagation behavior in the far field of the infinite domain, the dynamic SSI interaction problem must be simplified by approximating the problem as a wave radiation problem when the viscous damping boundary model is used.

4. Modeling details

The superstructure model used in the scope of the present study is a plane, three-spacing, 15-story, reinforced concrete frame that is 12 m in width and 45 m in height (Fig. 4). All of the beams are 30 × 50 cm and all of the columns are 40 × 40 cm in size. By changing the lateral stiffness characteristics of the HDRBs, three different base-isolated structure models are developed. The behaviors of these models are compared with the behaviors of the “without isolator” model. Characteristics of the HDRB used in this study are given in Table 1.

The soft soil computer program models the two-dimensional shell as an element (Fig. 4). The total width of the base model is determined as 144 m and the total depth as 50 m. The shell mesh size is dense in the structure base and its close surrounding; however, this mesh is sparse in the area away from the structure. The used mesh where the network range for 26 m, 74 m and 144 m are about 1 m, 2.5 m and 5 m, were chosen, respectively. Also in model fictitious dashpots were used as artificial boundary conditions. The connection between the soil and superstructure is established with the help of the base, specified as 1 m in thickness. The ground of the base model is fixed by the fixed supports. The horizontal limits of the soil are modeled as link elements. Link elements with the properties of the soil are placed on these points to prevent the seismic waves that

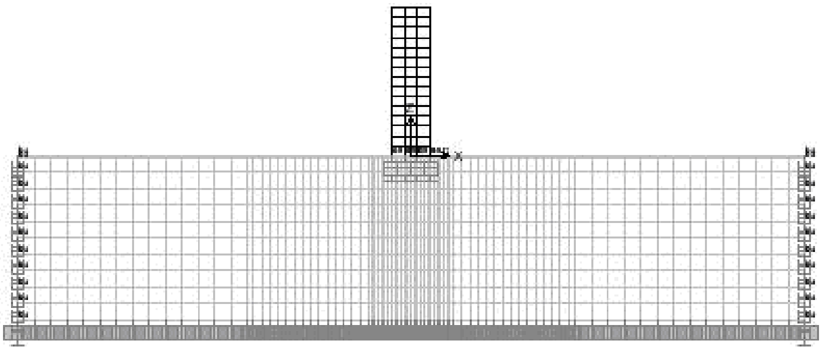


Fig. 4 Base isolation model developed when considering SSI

Table 1 Characteristics of the HDRB (Karabork 2011)

Stiffness	Vertical stiffness (K_v) (kN/m)	Initial stiffness (K_h) (kN/m)	Effective stiffness (K_{eff}) (kN/m)	Yielding force (F_y) (kN)	Post yield stiffness ratio (α)	Damping ratio (β) (%)
Flexible	1751.10^6	1.751.269	262.691	22.241	0,2	15
Medium	$1373.\ 10^6$	7.786.481	1.078.732	77.865	0,043	12
Stiff	$2746.\ 10^6$	12.454.446	1.863.264	124.544	0,055	10

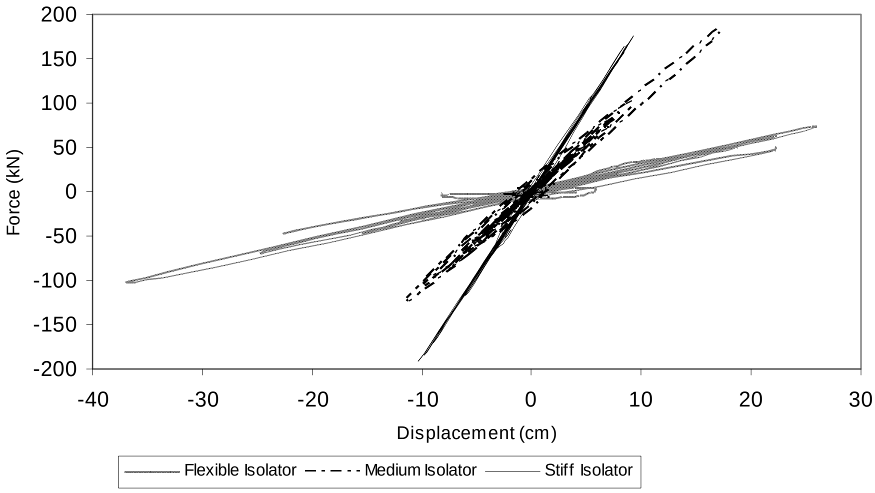


Fig. 5 Comparison of base hysteretic loops of different isolators

reach the sides from returning back into the soil. During the analysis, the base is deemed to be elastic. The dynamic characteristics of the soft base model are listed in Table 2.

Fig. 5 presents the force-displacement graphics of the isolators. Although flexible isolators have poor initial stiffness (K_h), the damping ratio (β) and energy absorption capacity of these isolators have high values. The stiff isolators have high initial stiffness, but the damping ratio and energy absorption capacity of these isolates are poor. Here, the flexible isolate does exhibit

Table 2 Soil Properties

	K_H (N/m)	K_Z (N/m)	C_H (Ns/m)	C_Z (Ns/m)	γ (kN/m^3)	μ	E (MPa)	V_s (m/s)
Mid-firm sand	1.19 10^8	0.153 10^9	0.036 10^7	0.075 10^7	18.00	0.3	30	400

Table 3 Peak response of various earthquake ground motions

Earthquake	Component	Magnitude (M_d)	PGA (cm/s^2)	PGV (cm/s)	PGD (cm)
Duzce 12 November 1999	N-S	7.2	407.68	39.28	7.17
Marmara 17 August 1999	E-W	7.4	373.76	52.14	12.73
Erzincan 13 March 1992	E-W	6.9	470.92	78.22	29.50

*PGA: peak ground acceleration; PGV: peak ground velocity; PGD: peak ground displacement

flexible behavior, indicating low lateral stiffness.

Where K_H and C_H are the horizontal components of the stiffness and damping ratio, K_Z and C_Z are the vertical components of the stiffness and damping ratio, μ is the Poisson ratio, γ is the density, E is the elasticity modulus and V_s is the shear wave velocity. The horizontal and vertical components of the stiffness and damping ratio can be written as shown in Eqs. (6) to (8)

$$K_H = 2(1+\mu)\beta_x[BL]^{1/2}; \quad C_H = \frac{0.576 K_H R_H}{V_s} \tag{6}$$

$$K_z = \frac{G\beta_z[BL]^{1/2}}{(1-\mu)}; \quad C_z = \frac{0.85 K_z R_z}{V_s} \tag{7}$$

where G is the shear modulus, B and L are the dimensions of the rectangular footing (in plan), R_H and R_Z are the horizontal and vertical translation of an equivalent circular footing radius, and β_x and β_z are constants associated with the horizontal and vertical translation, respectively.

$$R_H = \frac{(1+\mu)(7-8\mu)\beta_x[BL]^{1/2}}{16(1-\mu)}; \quad R_z = \frac{\beta_z[BL]^{1/2}}{4} \tag{8}$$

5. Numerical analysis

The models developed in this study were analyzed for two categories: those models developed by considering SSI and those models developed without considering SSI. Nonlinear dynamic analyses of the models are made with the SAP 2000 program, using the time history method. Three earthquake acceleration records were used as input excitations in the simulation, as mentioned in UBC (1997). Detailed information about these three earthquake records is given in Table 3. Normalized acceleration and displacement response spectra of these earthquakes are presented in Figs. 6 and 7. At the end of the analysis; displacements, base shear forces, top story

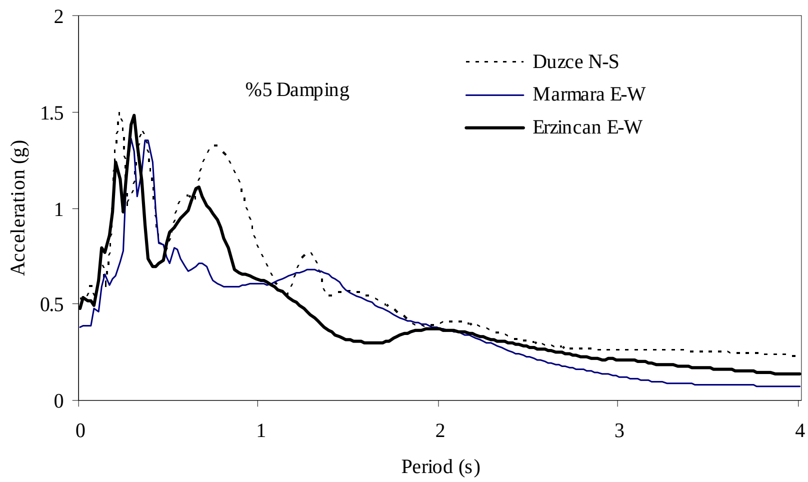


Fig. 6 Normalized acceleration response spectra of the three earthquakes

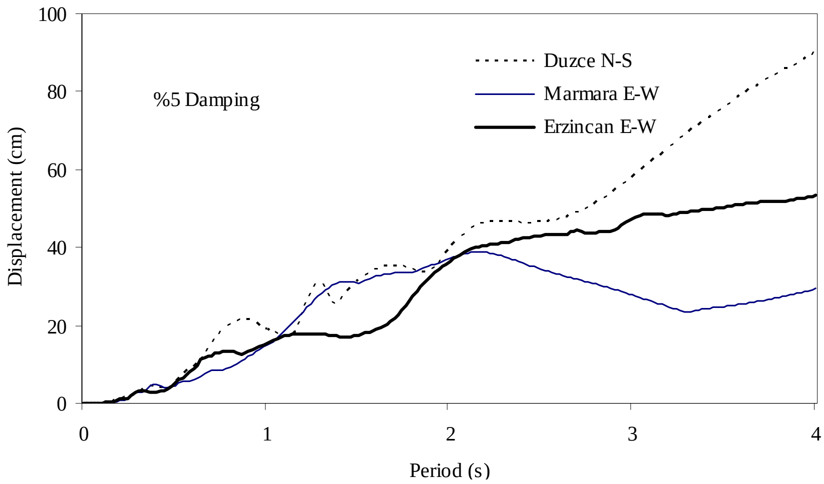
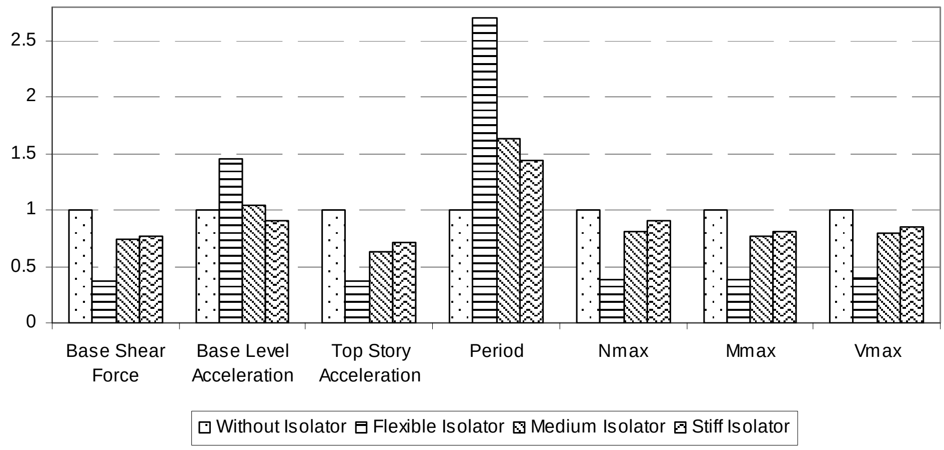


Fig. 7 Displacement response spectra of the three earthquakes

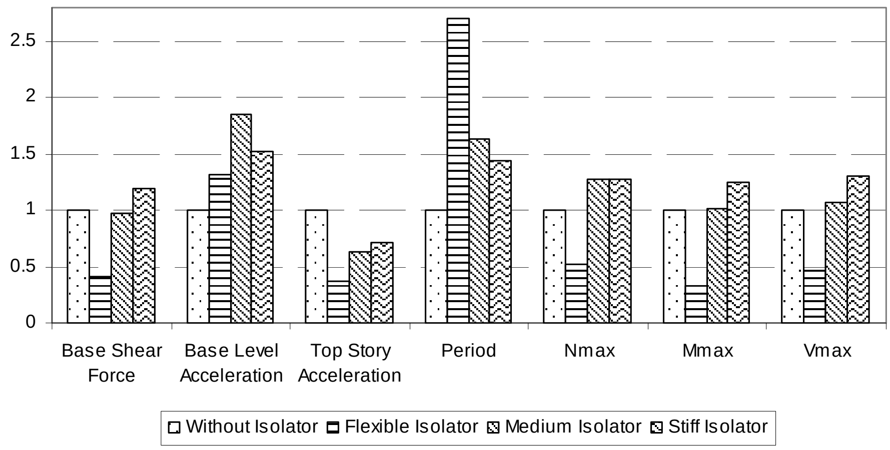
accelerations, base level accelerations, periods and maximum internal forces are compared in isolated and fixed-base structures with and without SSI.

Figs. 8 and 9 show the base shear forces, base level acceleration, top story acceleration and maximum internal forces values obtained during the analysis for Duzce, Marmara and Erzincan earthquakes and the exceeding isolator level. The values obtained for the "without isolator" condition are deemed one unit and proportioned to the other models. The maximum values of the without isolator condition are given in Table 4.

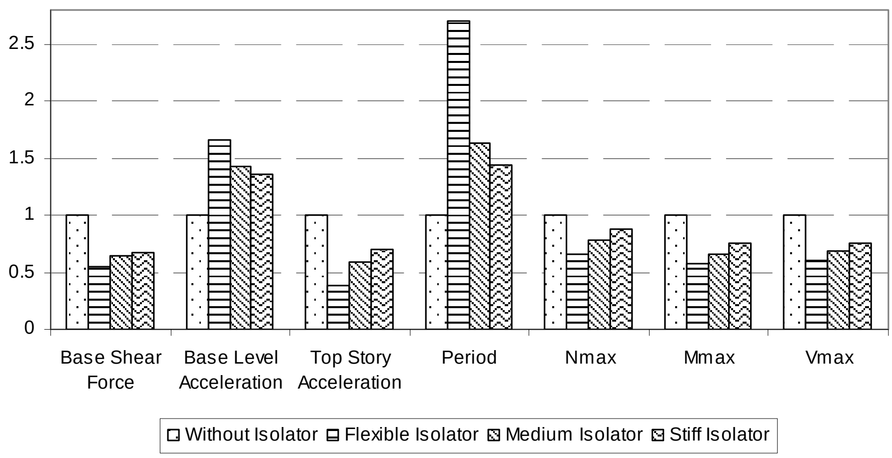
The use of a flexible isolator was shown to be effective in the models of the Duzce, Marmara and Erzincan earthquakes without SSI. When compared with the "without isolator model" for the Duzce earthquake, the flexible isolator and internal forces decrease by 60%, the base shear force and top story acceleration values decrease by 62%, and the base level acceleration value increases by 45%. When compared with the "without isolator model" for the Erzincan earthquake,



(a) Duzce Earthquake

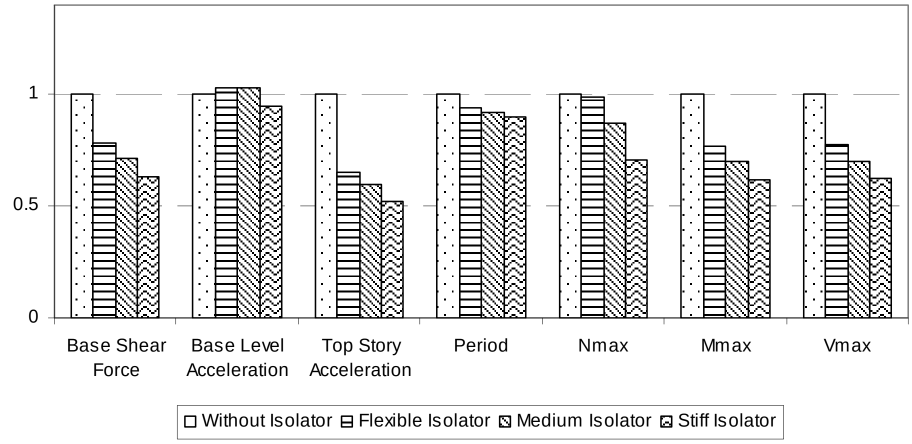


(b) Marmara Earthquake

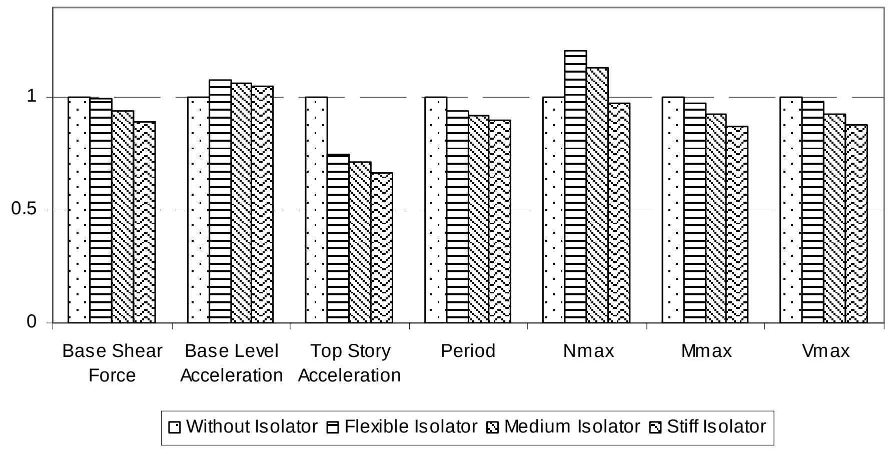


(c) Erzincan Earthquake

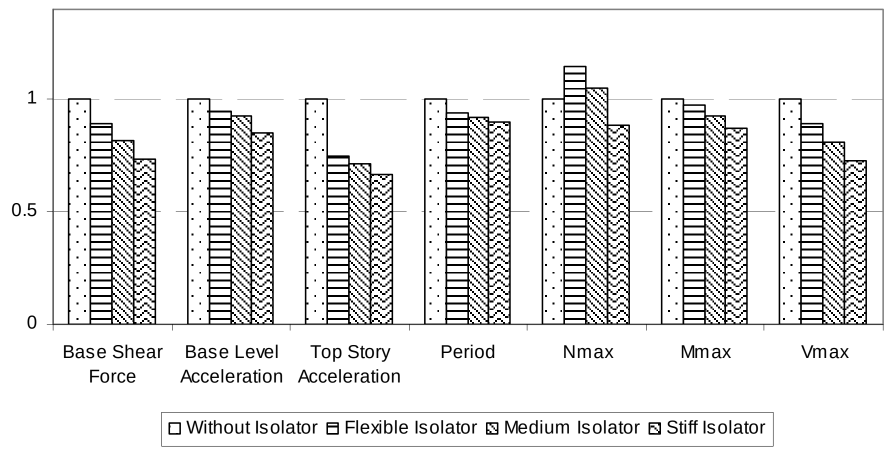
Fig. 8 Peak response under three earthquakes (without SSI)



(a) Duzce Earthquake



(b) Marmara Earthquake



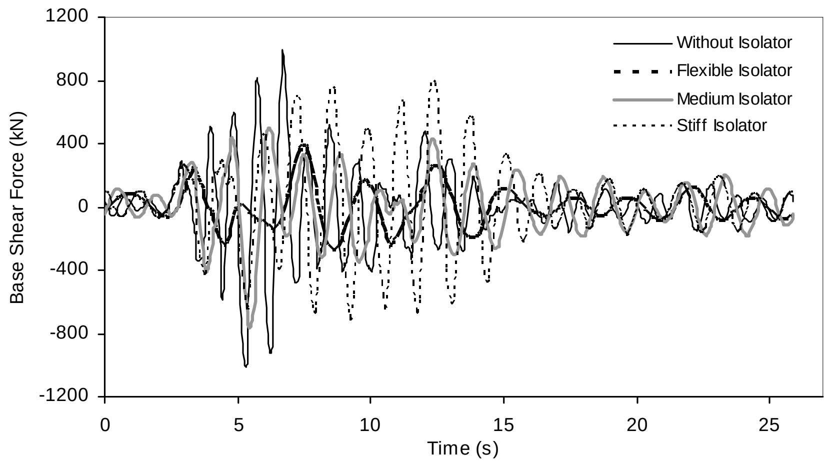
(c) Erzincan Earthquake

Fig. 9 Peak response under three earthquakes (with SSI) the flexible isolator and internal forces decrease by 38%, the base shear force decreases by 45%, the top story acceleration value decreases by 61%, and the base level acceleration value increases by 65%. Similarly, when compared with the "without isolator" model for the Marmara earthquake, the flexible isolator and internal forces increase by 55%, the base shear force increases by 58%, the top story acceleration value increases by 62%, and the base level acceleration value is increased by 31%. In addition, the natural vibration period of the structure is increased by 170%.

The efficiency of the isolation system in the structures built on soft soils is analyzed by taking SSI into account. An analysis of the Duzce, Marmara and Erzincan earthquakes showed that the isolation system with a stiff isolator was more effective than the structure model with fixed support. When compared with the structure model with fixed supports, the system with a stiff

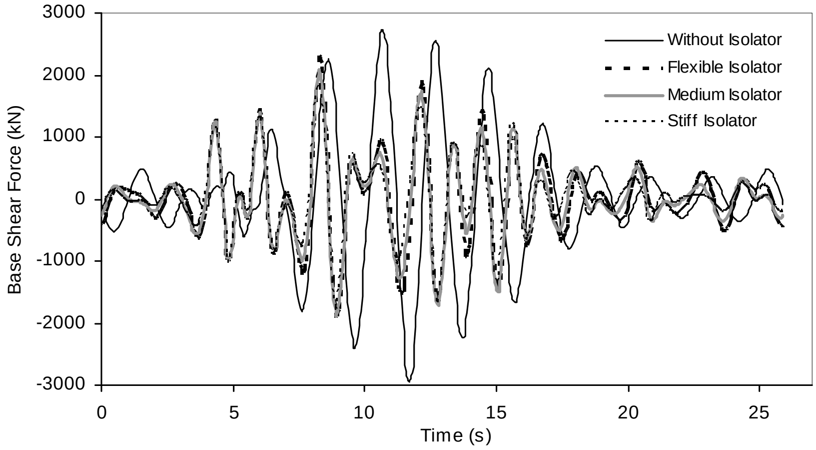
Table 4 Maximum values for the without isolator condition

	Base shear force (kN)	Base level acceleration (m/s^2)	Top story acceleration (m/s^2)	Period (s)	N_{max} (kN)	M_{max} (kNm)	V_{max} (kN)
Duzce Earthquake (without SSI)	1015.84	3.84	12.92	0.93	233.88	488.89	318.62
Marmara Earthquake (without SSI)	733.6	2.92	12.92	0.93	148.33	348.08	227.6
Erzincan Earthquake (without SSI)	847.92	3.21	14.46	0.93	171.47	405.54	264.49
Duzce Earthquake (with SSI)	2948.32	8.87	34.68	2.2	592.91	1414.1	980.51
Marmara Earthquake (with SSI)	2253	8.4	29.4	2.2	467.1	1088.49	753.65
Erzincan Earthquake (with SSI)	2655	8.61	29.4	2.2	530.43	1088.49	884.1



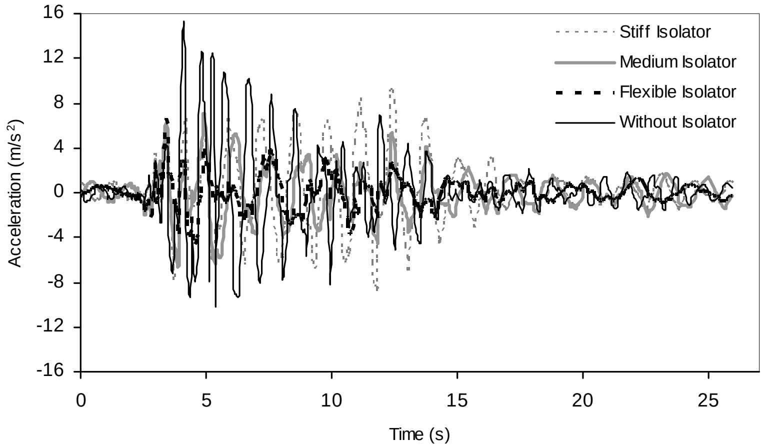
(a) Without SSI

Fig. 10 Variation of base shear force with time for Duzce Earthquake

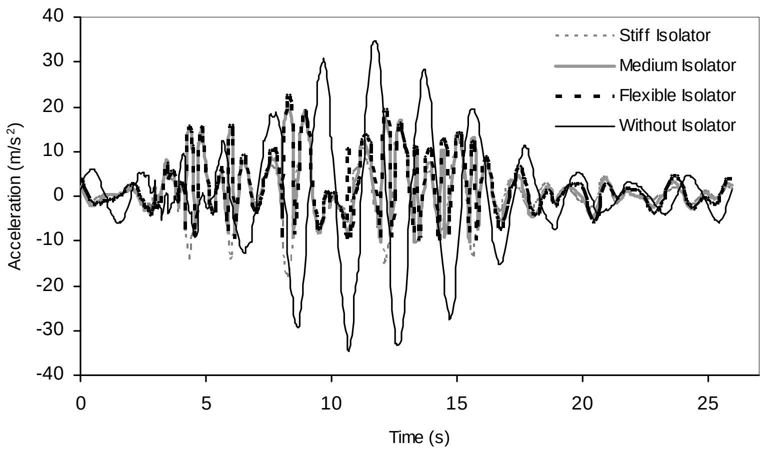


(b) With SSI

Fig. 10 Continued



(a) Without SSI



(b) With SSI

Fig. 11 Variation of top story acceleration with time for Duzce Earthquake

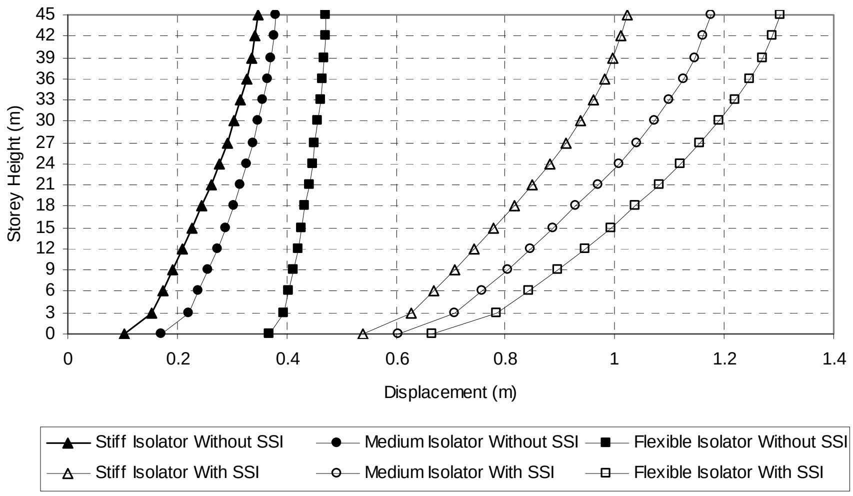


Fig. 12 Variation of displacements with structure height for Duzce Earthquake

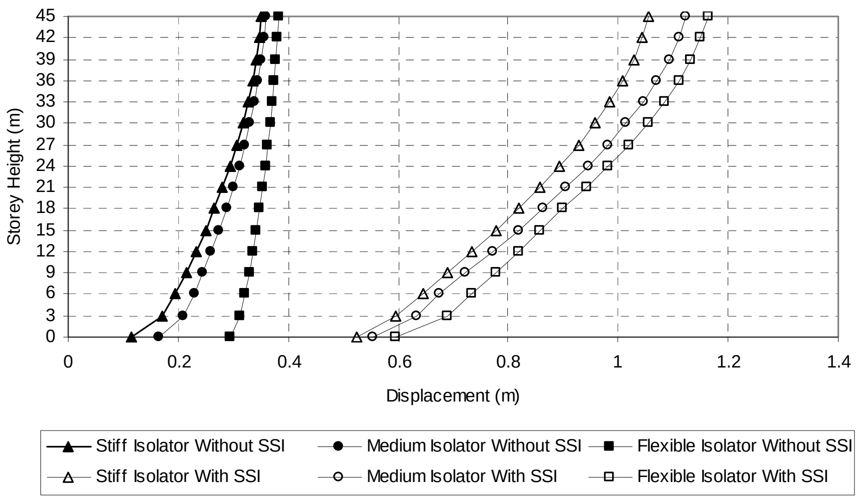


Fig. 13 Variation of displacements with structure height for Marmara Earthquake

isolator reduced the internal forces by 35%, base shear force by 37%, top story acceleration by 47% and base level acceleration by 5% for the Duzce Earthquake. When compared with the structure model with fixed supports, the system with a stiff isolator reduced the internal forces by 9%, base shear force by 10%, top story acceleration by 33% and base level acceleration by 5% for the Marmara Earthquake. When compared with the structure model with fixed supports, the system with a stiff isolator reduced the internal forces by 17%, base shear force by 26%, top story acceleration by 33% and base level acceleration by 14% for the Erzincan Earthquake. No significant difference was found between the period values because these period values apply to the whole structure-base-isolation system. Here, the efficiency of the isolation system in increasing the system period decreased. The efficiency of the isolation system decreased in parallel with the decrease in the stiffness of the HDRB. Fig. 10 and 11 show the changes recorded in the base shear

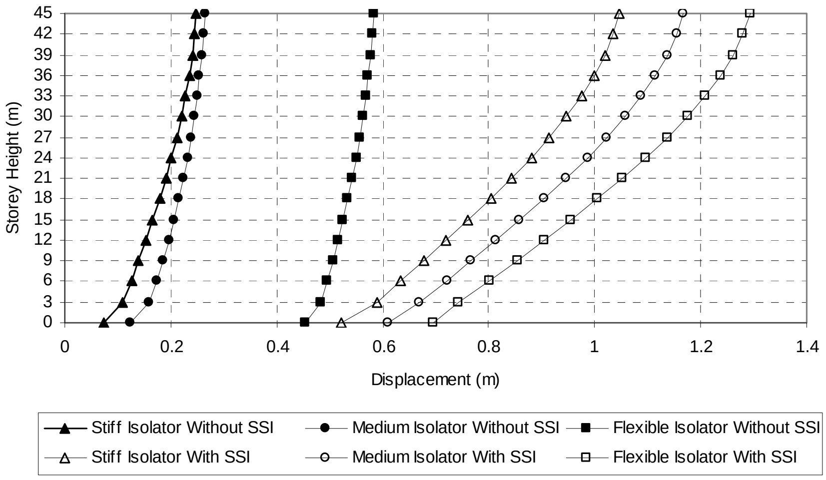


Fig. 14 Variation of displacements with structure height for Erzincan Earthquake

force and top story acceleration values, recorded in all models of the Duzce Earthquake, with SSI and without SSI.

Lateral displacement profiles of all of the models are given in Figs. 12 to 14. When the SSI effect is considered, story displacements increase. Displacements increase in line with the decrease in lateral stiffness. When SS effect is taken into consideration, between- story relative displacements values increase. The recorded displacement values are precise for the Marmara earthquake, while the range of displacement values increases for the Duzce and Erzincan earthquakes.

6. Conclusions

The structural systems developed in the present study were subjected to dynamic analysis by the time history method and by considering the fixed support and SSI factors for three acceleration values. Flexible isolators are effective in the fixed-support type systems without taking into consideration the structure interaction. Thus, the superstructure is exposed to a less intensive earthquake effect.

In the models that incorporated the soil-structure interaction, stiff isolators were shown to decrease the effects of an earthquake more significantly. In conclusion, special attention should be given to the selection of the appropriate isolator by considering both fixed support and SSI factors. When the SSI effect is not considered, the system efficiency is inversely proportional to the lateral stiffness of the isolator. However, when SSI efficiency is considered, efficiency is directly proportional to the stiffness of the isolator.

SSI has to be taken into consideration in base isolation structural systems built on soft soils. This practice is necessary because an isolation system may appear effective when the system is assumed to be of a fixed support type. However, the efficiency provided by the fixed support system decreases when the SSI factor is taken into consideration, even under the same conditions.

Because the viscous damping boundary cannot be used to simulate both the real geometry and the wave propagation behavior in the far field of the infinite domain (Zhao and Valliappan 1993a, Zhao *et al*. 1992), the coupled method of finite and dynamic infinite elements (Zhao 2010b) should be used for solving dynamic SSI problems involving semi-infinite and infinite domains in future research.

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