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A review of soil–foundation–structure interaction and structure–soil–structure interaction effects based on numerical simulations

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Abstract: This study presents a comprehensive review of dynamic soil–foundation–structure interaction (SFSI) and structure–soil–structure interaction (SSSI) as addressed in modern seismic design codes and numerical simulation approaches. The investigation focuses on how SFSI and SSSI phenomena affect the vibrational response of structures under seismic loading. We employed a systematic methodology to select relevant literature and code provisions, analysing their treatment of interaction effects and comparing simulation results. Numerical studies, design standards, and experimental validations were considered to evaluate the implications of ignoring or incorporating interaction effects in structural design. Key findings indicate that while most codes provide procedures to account for SFSI, SSSI effects remain largely overlooked. Furthermore, current methods often assume linear soil behaviour, limiting their applicability to real-world conditions. Experimental results from centrifuge modelling and shaking table tests demonstrate that nonlinear soil behaviour and foundation flexibility can significantly alter seismic response. The SFSI has been found to reduce seismic demand through energy dissipation mechanisms such as rocking, while SSSI can either amplify or mitigate response depending on relative mass and stiffness. Despite these critical impacts, current design practices often neglect such interactions, particularly in densely built environments. These findings underline the necessity of integrating SFSI and SSSI into seismic analysis frameworks for safer and more accurate performance-based design. The review highlights the need for comprehensive models and experimental validation to support the development of more resilient design practices.

Keywords: design codes, earthquake, soil-foundation-structure interaction, stiffness of soil, structure–soil–structure interaction

INTRODUCTION

Soil-foundation-structure interaction (SFSI) is a terminology which can be explained as the dynamic interaction between an isolated structure and surrounding soil through its foundation during an earthquake. Through SFSI effects, motion recorded at foundation, which is usually referred to as foundation input motion (FIM), differs from the free-field (FF) motion. There are two distinct mechanisms in SFSI, kinematic interaction (KI) between soil and foundation, and inertial interaction (II) of structure vibration. The difference between foundation input and free field motion is mainly attributed to the KI; while the II caused the structure's movements and foundation deformations including settlement, rocking and sliding.

This difference causes the change in the structure response both in time and frequency domains. Effects of SFSI must be considered when designing structures considering seismic impacts. Therefore, recent design codes such as ASCE (2017b) and ATC (Applied Technology Council) (2005) have provided procedures to include SFSI during seismic rehabilitation of structures. A huge number of researchers have investigated seismic SFSI effects of an isolated soil-foundation-structure (SFS) system (Veletsos and Damodaran Nair, 1975; Kim and Stewart, 2003; Mylonakis and Gazetas, 2000; Stewart, Seed and Fenves, 1999), and the understanding of SFSI effects is therefore vigorous. These effects include: (1) period lengthening of the SFS system and resonance at low-frequency earthquake; (2) increased foundation deformations include settlement, rocking, and sliding which can lead to serviceability problems and structure damage (Kim and Stewart, 2003; Gajan *et al.*, 2005; Gajan and Kutter, 2008; Mylonakis and Gazetas, 2000; Mylonakis, Nikolaou and Gazetas, 2006; Stewart, Fenves and Seed, 1999; Stewart, Seed and Fenves, 1999; Veletsos and Damodaran Nair, 1975).

Though robust knowledge of SFSI effects and the guiding procedures are provided in available seismic design codes, SFSI effects are generally ignored in engineering practice because of safe purposes due to the period lengthening and increasing SFS system damping effects of SFSI. These effects generally result in a lower spectral response in structure compared to a procedure without considering SFSI (Kim *et al.*, 2015).

Furthermore, it has been found that full SFSI effects are not considered in design procedures in recent seismic codes. First, the nonlinearity caused by uplifting of shallow foundation and yielding of soil beneath which can lead to serious structural damage (Harden, Hutchinson and Moore, 2006). Second, in seismic design codes, the ground type is generally classified to estimate ground motion using average shear wave velocity up to a depth of 30 m ($V_{s,30}$). This is insufficient for SFSI analysis because the appearance of a soft layer near to the foundation could have considerable influence on SFSI and structure behaviour during seismic loading (Rayhani and El Naggar, 2007; Rayhani and El Naggar, 2012).

Furthermore, SFSI induces deformations of foundation includes rocking, sliding, and settling behaviours under seismic loading (Limkatanyu and Kwon, 2022; Fatahi, Tabatabaiefar and Samali, 2024). These foundation behaviours could not only reduce the structure response through rocking and sliding damping but also mobilise ultimate bearing capacities of foundation (Anastasopoulos *et al.*, 2022; Drosos *et al.*, 2022). Therefore, several experimental studies have been performed to reveal rocking mechanism during earthquake and seismic loading. By conducting experiments using centrifuge models with various soil conditions, foundation dimensions, structure characteristics, and loading types, some studies observed that the moment-to-shear ratio ($M/(H{\cdot}L)$) is one of the parameters whose effects control not only the rocking and sliding behaviours of the footing but modifications in the mobilisation of the bearing capacity under couple vertical, horizontal, and moment loadings. Also, degradation of rocking stiffness was found to be a power function of the rocking angle. Drosos *et al.* (2022) and Ko *et al.* (2023) concluded that the vertical safety factor (FSV), which is proportional to critical contact area ratio of the foundation (A/Ac, where A is foundation area and A_c is critical contact area between foundation and soil at ultimate condition of rocking structure) is a key parameter to decide whether the foundation uplifts or settles in response to rocking. More also, they indicated that seismic acceleration of the structure could be reduced by the uplifting and nonlinear rocking response of the foundation during an earthquake. By performing geo-centrifuge tests for structures with various natural frequencies, Ngo *et al.* (2021) showed a reduction in the seismic response of a structure with a rocking foundation in comparison to a fixed-base structure. They also concluded that the effects of foundation rocking on structure response during an earthquake is undeniable and as a result, should not be ignored during an SFSI analysis.

Therefore, it is desirable to investigate further and extend the understanding of SFSI and also of structure-soil-structure interaction (SSSI).

Recently, structures are built close to each other in many cities and since two or more structures affect each other during an earthquake, a number of researches have been conducted to investigate the phenomenon of structure-soil-structure interaction (SSSI) or dynamic cross interaction (DCI) (Lou *et al.*, 2021; Aldaikh *et al.*, 2025) with the hope of guiding engineers to avoid the hazards of unforeseen SSSI effects. A detrimental SSSI effect was observed on the response of structure which is shorter or lighter when placed adjacent to a taller or more massive structure (Chen, Masienikov and Johnson, 2020; Kitada, Hirotani and Iguchi, 2021; Ogut, 2021; Aldaikh *et al.*, 2025). Also, when the distance between structures is smaller than the foundations width, SSSI effects have been found to be more significant (Aldaikh *et al.*, 2025). The rocking restriction condition from a more massive structure to foundation of a lighter structure during earthquake when the two structures located close to each other because a reduction in permanent settlement of less massive structure at foundation near more massive structure, that results in the less massive structure rotated away the more massive structure (Mason *et al.*, 2023). The rotation of the structure away from the adjacent structure were also reported when two structures were located at close distance between them base on a centrifuge experiment (Knappett, Madden and Caucis, 2025).

MATERIALS AND METHODS

SIMULATION-BASED EVALUATION METHODS

The initial purpose of SSSI researches were to investigate the interaction of several buildings in a configuration of a nuclear power plant (NPP) during an earthquake. Luco and Contesse (1973), Lee and Wesley (2003), Wong and Trifunac (2005), and Murakami and Luco (2007) performed analytical researches based on wave-propagation method and found that seismic interaction between closely inter-spaced structures can alter dynamic response of a single structure at frequency near the resonant frequencies of adjacent structures. However, these studies based on elastic half-space theory and soil nonlinear response was not considered, which reduces the application of their results in engineering practices. After that, Behnamfar and Sugimura (2009) investigated SSSI effects between twin buildings located on surface of an elastic half-space and found that SSSI effects increased structural response of short buildings and decreased response of tall buildings. Despite adopting several simplifying assumptions, these initial analytical approaches made basic contributions to the understanding of SSSI effects. The effects of structure–soil–structure interaction (SSSI) have been found to depend primarily on the inter-spacing between structures, the types of foundations and structures, the relative mass and stiffness of the structures, their group layout, and the properties of the soil.

Besides, a large number of numerical studies have been performed recently and the outcome showed that foundation-soil-foundation and/or SSSI effects could modify the seismic response of soil-foundation and/or soil-foundation-structure systems during seismic loading (Imamura *et al.*, 1992; Matthees and Magiera, 2002; Mulliken and Karabalis, 2008; Padrón, Aznárez and Maeso, 2019). However, these numerical studies were mostly based on linear soil behaviour and/or validated by previous analytical results without an experiment or a prototype model. Later, Nakagawa *et al.* (2008) performed large-scale vibration tests to investigate the interaction between buildings in a cluster of NPP and found the SSSI effects at fundamental frequencies of adjacent structures. They also found a decrease in response of the reactor building when the control building was installed. Furthermore, Kitada, Hirotani and Iguchi (2021) performed shaking table tests of small-scaled nuclear structures located on silicon rubber soil. However, these models were excited with a low-level earthquake, which led to a linear soil response and SFSI. Recently, Trombetta *et al.* (2013) performed centrifuge tests with a single and group of three structures to investigate SFSI and SSSI from a deep foundation to an inelastic and shallow-foundation structure. Knappett, Madden and Caucis (2025) studied SSSI effects between two structures with similar and different fundamental periods on a homogeneous ground. Previously, there were several requirements to be performed so to investigate SSSI effects including (1) comprehensive physical models performed for SSSI effects between two and/or three structures in different setting layouts such as distance between structures and direction of earthquake motions; (2) SSSI on ground which have several layers and layered effects. The phenomenon of (SSSI) is still not fully understood, and its underlying mechanisms have yet to be clearly identified.

Soil-foundation-structure interaction (SFSI) provisions currently considered in several seismic design standards (ASCE, 1998; FEMA, 2000; American Society of Civil Engineers, 2015; NIST, 2012; ASCE, 2017a). The procedures of linear static, linear dynamic, nonlinear response history and pushover will be presented. The kinematic interaction and foundation radiation damping were recently considered based on ASCE (2017a) standard with nonlinear response history and linear dynamic procedures, which implied the fully SFSI effects were adopted in this code.

In this study, a general overview and evaluation of several analysis methods are presented, including linear static, linear dynamic, nonlinear response history, and pushover analyses, to assess the behaviour of soil–foundation–structure systems under seismic loading. Linear static and dynamic methods evaluate structural response assuming elastic behaviour of both soil and structure. The nonlinear response history procedure allows for more realistic simulation by accounting for material and geometric nonlinearity. Pushover analysis estimates structural capacity and potential failure mechanisms under gradually increasing lateral loads. Additionally, the spring-based method is reviewed as a simplified approach to model foundation flexibility and energy dissipation in soil–structure interaction.

These methods are widely used in both design practice and academic research related to seismic response. Their advantages, limitations, and applicability will be briefly discussed in the context of structure–soil–structure interaction (SSSI). This overview provides a foundation for selecting suitable methods in future numerical or experimental investigations.

DIRECT METHOD

The direct method is the most common method used for fully nonlinear analysis. The finite-element method (FEM), boundary-element method (BEM), and finite-different method (FDM) are usually implemented for seismic analysis considering soil-foundation-structure interaction. A variety of computer programs can be used to perform a direct method in time domain such as ABAQUS (Anastasopoulos *et al.*, 2014), ANSYS (Kim, Lee and Lee, 2016), FLAC3D (Rayhani and El Naggar, 2007; Rayhani and El Naggar, 2012), LS-DYNA (Coleman, Bolisetti and Whittaker, 2016), and OpenSees (Karimi and Dashti, 2016). In these programs, soil, foundation, and structures are modelled simultaneously and number of works need to be resolved such as (1) a nonlinear soil model that is able to capture dynamic nonlinear behaviour of soil under a cyclic loading (such as simplified nonlinear kinematic hardening model in ABAQUS developed by Gazetas, Anastasopoulos and Garimi (2014), hypoplastic constitutive model proposed by Mašín (2005), (2) an artificial boundary condition that could prevent the reflection of incident wave when it comes to boundary (i.e., free-field boundary and absorbed boundary in FLAC3D (Itasca Consulting Group, 2013), infinite boundary in ABAQUS, and reaction boundary in ANSYS (Kim, Lee and Lee, 2016), and (3) the contact between soil and foundation that could show the sliding, gaps, and reduction of contact area between soil and foundation (Gajan and Kutter, 2009; Rayhani and El Naggar, 2007; Rayhani and El Naggar, 2012).

SUMMARY AND RESULTS OF RESEARCH REVIEW ON DYNAMIC SOIL-FOUNDATION-STRUCTURE INTERACTION

LINEAR STATIC PROCEDURE

The linear static procedure is usually called the equivalent lateral force method, and it is provided in ASCE (2017a). In this procedure, several seismic design parameters have been defined, including seismic base shear (V), structural period (T), vertical and horizontal distributions of seismic forces, overturning moment, story drift (Δ), and P-delta effects. The kinematic interaction is neglected and the inertial interaction with period lengthening and modification of damping is considered. Results of SFSI in a decrease in base shear as:

$$\widetilde{V} = V - \Delta V = V - \left(C_s - \frac{C_s}{B_{SSI}}\right)\bar{W} \geq \alpha V \tag{1}$$

where: V = fixed-base structure base shear, $\widetilde{V}$ = fixed-base structure base shear, considering soil–foundation–structure interaction (SFSI), ΔV = reduction in base shear due to SFSI effects, C_s and $\widetilde{C}_s$ = coefficients of seismic response which is defined based on design spectral response acceleration and R factor of structure at fixed-base and flexible-base conditions, respectively; the values of V, R, and C_s are defined in chapter 12 of ASCE (2017a), "Minimum design loads for buildings and other structures"; $\bar{W}$ = effective seismic weight of structure, which is generally taken as 70% total weight of structure, α = reducing coefficient related to foundation damping, defined as:

$$\alpha = \begin{cases} 0.7 & \text{for } R \leq 3 \\ 0.5 + \frac{R}{15} & \text{for } 3 < R < 6 \\ 0.9 & \text{for } R \geq 6 \end{cases} \tag{2}$$

where: R = modification factor that depends on structure types and material.

$$B_{SSI} = \frac{4}{5.6 - \ln(100\beta_0)} \tag{3}$$

where: B_{SSI} = adjusting factor for design response spectra.

Effective viscous damping of SFSI system (β_0) could be defined according to Veletsos and Damodaran Nair (1975), Stewart, Fenves and Seed (1999), and NIST (2012) as follows:

$$\beta_0 = \beta_f + \frac{1}{\left(\frac{\widetilde{T}}{T}\right)^n}\beta \tag{4}$$

where: β_f = the foundation damping factor with the distribution of hysteretic and radiation damping of soil-foundation system (β_f value varies from 0 to 25% acc. to Stewart, Seed and Fenves, 1999), β = the damping of fixed-base structure which should depend on material type and behaviour of a structure, and it is generally taken as 5%, the exponent n depends on the type of structural damping, it is taken as 3 for viscous damping, and 2 for the others (Givens, 2013).

The β_f in Equation (4) with n exponent taken as 2 and effective period lengthening ratio is adopted as:

$$\beta_0 = \beta_f + \frac{1}{\left(\frac{\widetilde{T}}{T}\right)^2_{\text{eff}}}\beta \tag{5}$$

The flexible-base frequency ($\widetilde{T}$) is defined as Equation (6):

$$\widetilde{T} = (2\pi)^2\frac{m\Delta}{F} = (2\pi)^2 m\left(\frac{1}{k_s} + \frac{1}{k_x} + \frac{h^2}{k_\theta}\right) \tag{6}$$

where: π = the mathematical constant Pi, approximately 3.142; m = the mass of the superstructure (typically in kg or kN·s^2·m^{-1}); F = the restoring force or applied force at the point of dynamic equilibrium; k_s = the soil translational stiffness, representing the resistance of the soil beneath the foundation to vertical or horizontal movement; k_{sx} = the foundation stiffness in the horizontal direction, typically associated with the structure's own base; k_θ = the rotational stiffness of the soil–foundation system (kN·m·rad^{-1}), capturing the foundation's resistance to rocking or rotation; h = the height from the foundation level to the centre of mass of the superstructure.

Therefore, $\widetilde{T}$ could be defined based on fixed-base period, (T) and structural stiffness (k) as follows (Veletsos and Meek, 1974; Stewart, Fenves and Seed, 1999; NIST, 2012):

$$\frac{\widetilde{T}}{T} = \sqrt{1 + \frac{k_s}{k_x} + \frac{k_s h^2}{k_\theta}} \tag{7}$$

The effective period lengthening ratio is:

$$\left(\frac{\widetilde{T}}{T}\right)_{\text{eff}} = \left\{1 + \frac{1}{\mu}\left[\left(\frac{\widetilde{T}}{T}\right)^2 - 1\right]\right\}^2 \tag{8}$$

where: μ = expected ductility demand, which is defined from type and material of the structure, β_f = the foundation damping which is defined by Wolf (1985) with exponent n taken as 2 defined as follows:

$$\beta_f = \frac{\left(\frac{\widetilde{T}}{T}\right)^2 - 1}{\left(\frac{\widetilde{T}}{T}\right)^2}\beta_S + \beta_{rd} \quad \text{and} \quad \beta_{rd} = \frac{1}{\left(\frac{\widetilde{T}}{T}\right)^2}\beta_x + \frac{1}{\left(\frac{\widetilde{T}}{T_\theta}\right)^2}\beta_\theta \tag{9}$$

The hysteretic damping ratio of soil, β_s, defined from soil site and effective peak acceleration, as illustrated in Table 1. The β_{rd} is foundation radiation damping, including sliding and rocking components, which are the rest of the parts in Equation (9). Table 2 shows equation to define sliding period (T_y), rocking periods (T_θ), sliding damping (β_y), and rocking damping (β_θ) of foundation.

Table 1. Hysteretic damping ratio of soil, β_s

Site class	Effective peak acceleration, $S_{DS}/2.5$[a)]			
	$S_{DC}/2.5 = 0$	$S_{DC}/2.5 = 0.1$	$S_{DC}/2.5 = 0.4$	$S_{DC}/2.5 \geq 0$
A	0.01	0.01	0.01	0.01
B	0.01	0.01	0.01	0.02
C	0.01	0.01	0.03	0.05
D	0.01	0.02	0.07	0.15
E	0.01	0.05	0.20	–[b)]
F	–[b)]	–[b)]	–[b)]	–[b)]

[a)] Using linear interpolation for values of $S_{DS}/2.5$.
[b)] Doing geotechnical investigation and dynamic response analyses for the ground.
Explanations: S_{DS} = design spectral response acceleration parameter.
Source: ASCE (2017a).

LINEAR DYNAMIC ANALYSIS

The linear dynamic procedure is a force-related method, which is similar to the linear static force method. In this procedure, the SSI effects modify the design response spectrum of structure and modal base shear, $\widetilde{V}_t$. The value of could be defined from value of reduced base shear considering SSI effects, $\widetilde{V}$, which was illustrated in previous sections, and it should not be below αV_t with V_t being modal base shear of structure in fixed-base condition; α parameter is defined in Equation (2).

With regards to SFSI effects, the design response spectrum was modified as follows:

$$\begin{cases} \bar{S}_a = \left[\left(\frac{5}{B_{SSI}} - 2\right)x + 0.4\right]xS_{DS} & \text{for } 0 < T < T_0 \\ \bar{S}_a = \frac{S_{DS}}{B_{SSI}} & \text{for } T_0 \leq T \leq T_s \\ \bar{S}_a = \frac{S_{D1}}{B_{SSI} \cdot T} & \text{for } T_s \leq T \leq T_L \\ \bar{S}_a = \frac{S_{D1}T_L}{B_{SSI} \cdot T^2} & \text{for } T > T_L \end{cases} \tag{10}$$

where: S_{DS} and S_{DI} = the design spectral response acceleration at a short period and at 1–s period, respectively, T = the fundamental period of the fixed-base structure, T_0, T_s, and T_L represent characteristic periods of the design response spectrum: $T_0 = 0.2T_s$; $T_s = S_{DI}/S_{DS}$ and T_L = the long-period transition period as defined in ASCE (2017a).

The damping of the soil-foundation system under SFSI effects is considered through the parameter of B_{SSI}, which is defined in Equation (3).

Table 2. Foundation characteristics in Equation (9)

Parameter	Rectangular foundation	Circular foundation
T_y or T_r	$2\pi\sqrt{\frac{M}{K_y}}$	$2\pi\sqrt{\frac{M}{K_r}}$
T_θ	$2\pi\sqrt{\frac{M(h_{\text{eff}})^2}{\alpha_\theta K_\theta}}$	$2\pi\sqrt{\frac{M(h_{\text{eff}})^2}{\alpha_\theta K_\theta}}$
K_y (K_r for circular foundation)	$\frac{0.5GB}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65}+0.8\frac{L}{B}+1.6\right]$	$\frac{8Gr_f}{2-\nu}$
K_θ	$\frac{GB^3}{8(1-\nu)}\left[3.2\left(\frac{L}{B}\right)+0.8\right]$	$\frac{8Gr_f^3}{3(1-\nu)}$
β_y (β_r for circular foundation)	$\frac{2\frac{L}{B}}{\frac{K_y}{GB}}\cdot\frac{a_0}{2}$	$\frac{\pi}{\frac{K_r}{Gr_f}}\cdot\frac{a_0}{2}$
β_θ	$\frac{\frac{4\psi}{3}\frac{L}{B}a_0^2}{\frac{8K_\theta}{GB^3}\left[\left(2.2-\frac{0.4}{\left(\frac{L}{B}\right)^3}\right)+a_0^2\right]}\frac{a_0}{2a_\theta}$	$\frac{a_0}{2a_\theta}$
a_0	$\frac{\pi B}{\tilde{T}V_S}$	$\frac{\pi r_f}{2\tilde{T}V_S}$
ψ < 2.5	$\left[\frac{2(1-\nu)}{1-2\nu}\right]^{0.5}$	$\left[\frac{2(1-\nu)}{1-2\nu}\right]^{0.5}$
a_θ	$1.0-\frac{0.55+0.01a_0^2\sqrt{\frac{L}{B}-1}}{2.4-\frac{0.4}{\left(\frac{L}{B}\right)^3}+a_0^2}$	$1.0-\frac{0.35a_0^2}{1.0+a_0^2}$

Explanations: T_y = translational vibration period of the rectangular foundation, T_r = translational vibration period of the circular foundation, T_θ = rotational vibration period of the foundation, K_y or K_r = translational stiffness of the soil–foundation system, K_r = circular foundations, K_θ = rotational stiffness of the soil–foundation system, β_y or β_r = translational damping coefficient due to foundation–soil interaction, β_r = circular foundations, β_θ = rotational damping coefficient due to foundation–soil interaction, a_0 = dimensionless frequency parameter representing dynamic interaction effects, ψ = Poisson-related parameter for rotation, typically used when ψ < 2.5, a_θ = dimensionless rotational parameter used in calculating rotational damping and stiffness, π = mathematical constant Pi (≈ 3.1416), M = structure mass, h_{eff} = the effective structure height, which is height of a one-story structure or 70% height of multistory structure; L, B, and r_f = length, width, and radius of foundation, V_s and G = average shear wave velocity and shear modulus of soil beneath structure over a depth of $B/2$ for rectangular foundation or r_f for circular foundation considering the reduction of V_s and G with shear strain compared to small-strain shear velocity and shear modulus, ν = Poisson's ratio, typical values for soil of 0.3–0.45, $\widetilde{T}$ = normalised fundamental period – a dimensionless period used in dynamic soil–structure interaction (SSI) analysis.
Source: American Society of Civil Engineers (2015) and ASCE (2017a).

NONLINEAR RESPONSE HISTORY PROCEDURE

In nonlinear response history procedures, response of structure could be evaluated through numerical integration of the equation of motion. The effects of soil–foundation–structure interaction (SFSI) were incorporated into this procedure through both a scaled site-specific response spectrum that considers kinematic interaction, and a mathematical model that includes damping in the soil–foundation system. The soil-foundation damping system was taken as the form of Wolf (1985) and was described in the above sections. Kinematic SFSI effects reduce foundation input motion via two mechanisms comprising of base slab averaging and embedment effects. ASCE (2017a) recommended using modification factors RRS_{bsa} and RRS_e for base-slab averaging and embedment effects, respectively, for the reduction of foundation input motion compared to free-field motion. Both the base-slab averaging, and embedment effects could be considered simultaneously for mat foundation with an embedment depth, but combined modification factor, $RRS_{bsa}{\cdot}RRS_e$ is not permitted to be lower than 0.7.

The base slab averaging effects used with structures with mat foundation located on the site class C, D, or E. The modification factor, RRS_{bsa} is calculated for each period (T) as follows:

$$RRS_{bsa} = 0.25 + 0.75\left\{\frac{1}{b_0^2}\left[1 - \left(\exp\left(-2b_0^2\right)\right)B_{bsa}\right]\right\}^{0.5} \tag{11}$$

where:

$$B_{bsa} = \begin{cases} 1 + b_0^2 + b_0^4 + \frac{b_0^6}{2} + \frac{b_0^8}{4} + \frac{b_0^{10}}{12} & \text{for } b_0 \le 1 \\ \left[\exp\left(2b_0^2\right)\right]x\left[\frac{1}{\sqrt{\pi}b_0}\left(1 - \frac{1}{16b_0^2}\right)\right] & \text{for } b_0 > 1 \end{cases} \tag{12}$$

$$\mathrm{b_0} = 0.0023(\mathrm{b_e}/\mathrm{T}) \tag{13}$$

and effective foundation size:

$$b_0 = \sqrt{BxL} \le 80 \text{ m} \tag{14}$$

For embedment effects, the modification factor, RRS_e is defined for each period, T, as:

$$RRS_e = 0.25 + 0.75x\cos\left(\frac{2\pi \mathrm{e}}{\mathrm{TV_s}}\right) \tag{15}$$

where: e = foundation depth, and V_s = average shear wave velocity of soil at embedment depth with considering a reduction of shear stiffness with the amplitude of motion.

PUSHOVER METHOD

Pushover methods have been recommended in seismic design standards, such as coefficient method, displacement modification method, and linearisation method (FEMA, 2015). Two required components include a design response spectrum and pushover curve (capacity spectrum) plotted in spectral acceleration (S_a) and spectral displacement (S_d) axes (Fig. 1).

A pushover analysis with an incremental lateral load pattern could be performed to obtain a pushover curve for a multi-degree-of-freedom (MDOF) soil-foundation-structure system. The system is pushed monotonically until roof displacement approaches a target value (Fig. 1). The level of inelasticity of the structure could be obtained by plotting lateral base shear, H versus roof displacement, Δ. The pushover curve obtained from the analysis is plotted on S_a–S_d space with the design response spectrum to find the seismic performance point of structure. To consider structural ductility (inelastic structural response), an inelastic demand spectrum, which is obtained using the response modification factor (R). The inclusion of SFSI effects on the demand spectrum is quantified by ASCE (2017a). The kinematic interaction reduced the foundation input motion (FIM) spectrum from the free-field (FF) motion spectrum by base-slab averaging and embedment effects, with the two modification factors, RRS_{bsa} and RRS_e, respectively, were defined in Equations (1) and (8). The inertial interaction is accounted for by an increase in the fundamental period (flexible-base period) and damping of system defined in Equations (6) and (7).

SPRING-BASED METHOD

Following the need for a simple model that could illustrate the most significant behaviour of a soil-foundation-structure system, several spring-base models have been reported in literature relying on some simplifications. Firstly, a rigid foundation is generally considered, and its responses presented by a representative point, which usually is the center of interaction surface between soil and foundation. Secondly, the soil is modelled by uncoupled springs with their stiffness which can either constant or dependent on frequency or deformation.

- **Uncoupled spring model**

In the uncoupled spring model, the soil-foundation system is replaced by three pairs of springs and dashpots to simulate sliding, rocking, and settlement deformation as shown in Figure 2.

The sliding stiffness (k_x), vertical stiffness (k_z), and rocking stiffness (k_θ), commonly used to present for each spring. These values of stiffness are functions of shape and dimensions of foundation, embedment depth, and stratification and properties of soil. Table 2 illustrates the equations of spring stiffness at static conditions, which is stiffness at a zero frequency, for surface and embedded foundations. Static stiffness and damping coefficient of spring modelled for surface foundation and embedded foundation on a layered ground are summarised in Gazetas, Anastasopoulos and Garini (2014) and Mylonakis, Rovithis and Parashakis (2011).

Even though uncoupled spring model is a simple and rapidly estimated model, the simplified assumptions of the model could give rise to errors: (1) stress in foundation and its effects on stress of soil around is unable to be determined because foundation is replaced by springs and dashpots; (2) coupling effects in

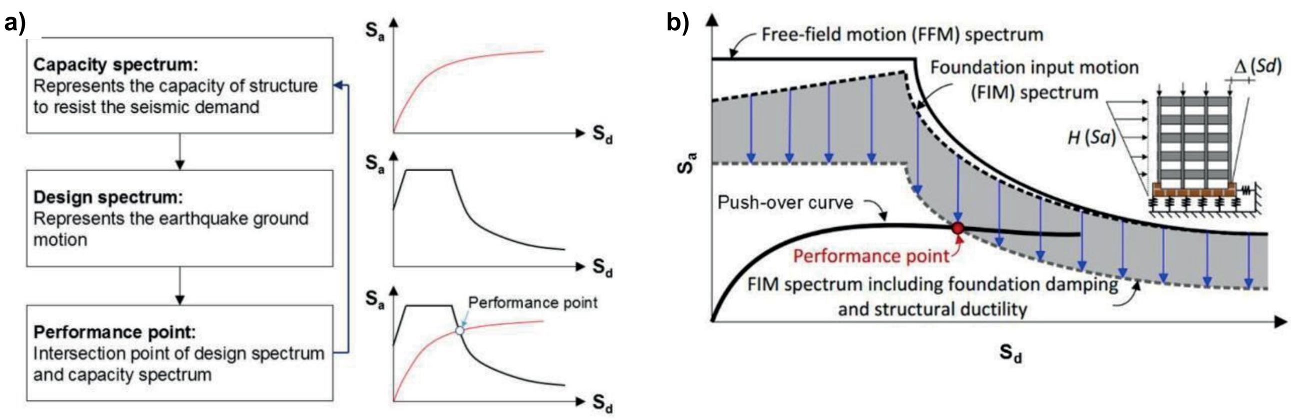

Fig. 1. Nonlinear performance of structure with pushover method considering SFSI effects: a) pushover procedure, b) consideration of SFSI on pushover procedure; source: own elaboration

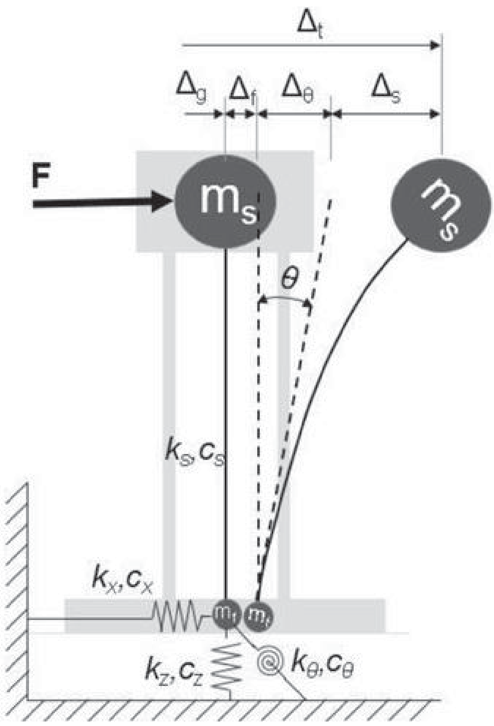

Fig. 2. Simple model for analysis of inertial interaction with deflection; m_s = mass of the superstructure, m_f = mass of the foundation; m_r = rotational mass or additional mass at the interface; F = applied lateral force on the structure; Δ_t = total lateral displacement at the top of the structure; Δ_s = structural deformation (relative lateral displacement due to frame bending); Δ_θ = displacement due to rotation (rocking) of the foundation; Δ_f = displacement of the foundation (horizontal translation); Δ_g = ground displacement (horizontal movement of the supporting soil); θ = rotation angle of the foundation due to rocking; k_s, c_s = structural stiffness and damping in the lateral direction; k_x, c_x = horizontal translational stiffness and damping of the soil or foundation interface; k_z, c_z = vertical stiffness and damping of the soil or foundation interface; k_θ, c_θ = rotational stiffness and damping of the foundation–soil system); source: Mylonakis *et al.* (2006)

foundation capacity (i.e., coupling vertical-horizontal-moment capacity of foundation – *V-H-M*), which was found by Houlsby, Cassidy, and Einav (2005), Cremer, Pecker and Davenne (2001), and Gajan and Kutter (2009), cannot be simulated because these springs and dashpots work independently of each other (Trombetta, 2013). The uplift at one side of the foundation during a strong motion and its effects on increasing in bearing pressure on another side of the foundation and in the sliding response as indicated in Figure 3. Yielding in soil and a slip surface probably occur because of increase in applied stress. Because of the yielding in soil and the reduction in the contact area between soil and foundation, stiffness of the soil-foundation system could be reduced that indicates the coupling capacity of the foundation.

- **Multi-spring model**

To reduce the disadvantages of the uncoupled spring model, a multi-spring model has been proposed, which is generally referred as to the Winkler model. In this model, the soil is modelled as a group of independent springs and foundation is modelled as a flexible beam as shown in Figure 4. Under an eccentric load (q), a different settlement is experienced in the foundation, which means the vertical and rotational stiffness is coupled.

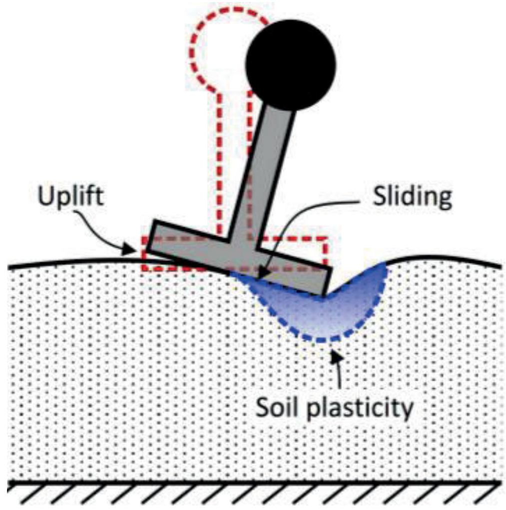

Fig. 3. Sliding and uplifting of foundation, and soil plasticity during a strong motion; source: Trombetta (2013)

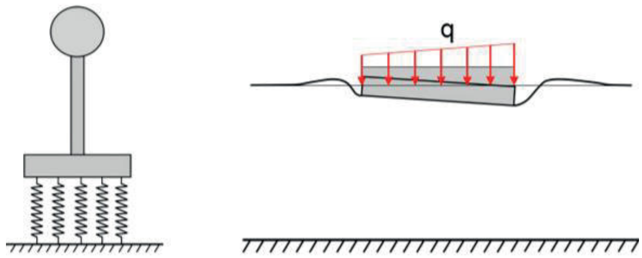

Fig. 4. Multi-spring model and foundation settlement and rotation under eccentric load; q = eccentric load; source: ASCE (2017a)

This type of model has been implemented in seismic design code such as ASCE (2014). In this standard, the foundation (with length and width of L and B, respectively) is divided into three zones (i.e., two end zones and a middle zone) supported by spring with different stiffness in each zone such as k_{end} and k_{mid} as indicated in Figure 5. The length of the end zone is $L_e = B/6$ and of middle zone is $L_m = L - B/3$. The stiffness per unit length of each zone is estimated from the shear modulus of soil (G) and poison's ratio (υ) as:

$$k_{\mathrm{end}} = \frac{6.83G}{1-\nu} \tag{16}$$

$$k_{\mathrm{mid}} = \frac{0.73G}{1-\nu} \tag{17}$$

As shown in Equations (16) and (17), the stiffness of spring in end zone is approximately nine times those in middle zone. Harden *et al.* (2005) and Ngo *et al.* (2019) modified length of end zone by setting the vertical stiffness of a $B.L_e$ plate (k_z) in a relationship with the rotational stiffness of a $B.L$ foundation ($k_{\theta y}$). The length of the end zone (L_e) of rectangular foundation be defined as:

$$L_e = 0.5 - L\left[\frac{1}{8}\left(1 - C^K_{R-V}\right)\right]^{1/3} \tag{18}$$

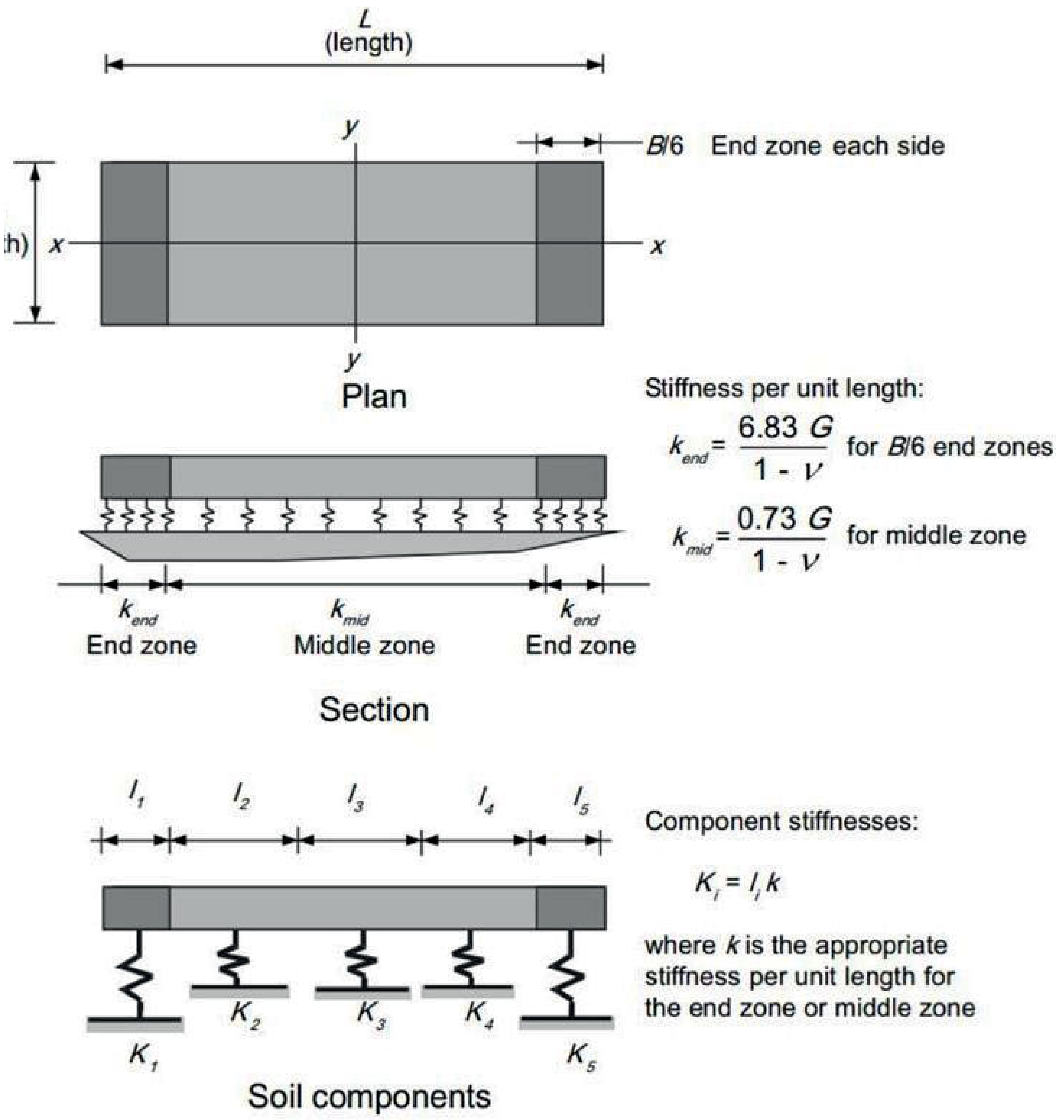

Fig. 5. Zones in the multi-spring model; L = length of the foundation (in the y-direction), B = width of the foundation (in the x-direction), $B/6$ = width of each end zone, taken as one-sixth of the total width, x, y = coordinate directions for the foundation layout, k_{end} = stiffness per unit length for the end zones, k_{mid} = stiffness per unit length for the middle zone, G = shear modulus of the soil, ν = Poisson's ratio of the soil, l_1, l_2, ..., l_5 = lengths of subzones within the foundation footprint, these divide the total length L into segments; K_1, K_2, …, K_5 = spring stiffness values for each soil subcomponent under the foundation; $K_i = l_i/k$ = equivalent spring stiffness, where: l_i = length of segment i, k = stiffness per unit length (either k_{end} or k_{mid} depending on the location); source: ASCE (2014)

$$C_{R-V}^{K} = \frac{K_{\theta y} - K_z I_y / A}{K_{\theta y}} \tag{19}$$

where: C_{R-V}^{K} = the rotational stiffness deficit ratio, which is the ratio of the rotational stiffness capacity difference to the rotational stiffness, I_y = the moment of inertia of foundation ($I_y = BL^3/12$), A = area of the foundation.

The stiffness of end- and middle- springs could be defined as:

$$k_{\text{mid}} = k_z = \frac{K_z}{LB} \text{ and } k_{\text{end}} = k_{\text{mid}} + \frac{K_\theta}{I_y} C_{R-V}^{K} \tag{20}$$

In the multi-spring model, foundation is simulated, and therefore, it is possible to define the distribution of stress in foundation. However, the multi-spring model above contains some limitations such as (1) lack of the coupling between horizontal with neither vertical nor rocking responses, and (2) nonlinearity behaviour of soil (i.e., decrease in soil stiffness with a progressive deformation) was not modelled.

Houlsby, Cassidy and Einav (2005) and Ngo *et al.* (2021) developed a generalised Winkler model using both nonlinear behaviour of stress-displacement response and a relationship between vertical and horizontal stiffness.

DISCUSSION

Comparative evaluation of four numerical methodologies frequently employed in seismic analysis, assessing them in terms of accuracy, computational efficiency, practical applicability, and capability to model nonlinear soil behaviour is presented in

Table 3. The linear static procedure (LSP) provides a simplified and computationally efficient approach, making it suitable for preliminary structural design and code-based seismic evaluations. However, its accuracy is constrained due to the assumption of fixed-base conditions, neglecting kinematic soil-structure interaction and nonlinear soil effects. The linear dynamic analysis (LDA) enhances accuracy by incorporating modal responses and soil-foundation-structure interaction (SFSI) effects. Although it requires additional computational resources compared to static methods, it is widely utilised in performance-based seismic design and response prediction. Nevertheless, LDA remains limited in capturing nonlinear soil behaviour, as it primarily modifies response spectra rather than explicitly modelling nonlinearities. The nonlinear response history analysis (NRHA) is the most rigorous approach, as it fully captures time-dependent seismic response, including material and geometric nonlinearities. While computationally intensive due to its reliance on step-by-step numerical integration, NRHA is indispensable for high-fidelity seismic risk assessments and the analysis of complex structures. It explicitly accounts for soil damping, foundation flexibility, and interaction effects, making it the most comprehensive method for evaluating soil-structure interaction. The pushover analysis (PA) offers a balance between accuracy and computational efficiency by estimating inelastic demand through an iterative nonlinear static procedure. It is particularly useful for performance-based seismic assessments and structural retrofit design. However, while it can represent some aspects of nonlinear behaviour, it does not comprehensively model time-dependent seismic effects.

The four evaluation methods differ in terms of accuracy, computational efficiency, and practical applicability, depending on the structural complexity and seismic conditions considered. The linear static procedure and pushover method are relatively simple and computationally efficient, making them suitable for preliminary design and code-based evaluations; however, their ability to capture dynamic characteristics and nonlinear soil–structure interaction is limited. The linear dynamic analysis improves accuracy by incorporating modal responses and soil–foundation–structure interaction (SFSI) modifications but still assumes linear elastic behaviour of the system. In contrast, the nonlinear response history analysis provides the most comprehensive and realistic simulation, as it fully accounts for time-dependent seismic responses and nonlinearities in both soil and structure. Nevertheless, its high computational demand often restricts its use to critical structures or projects requiring advanced seismic performance assessment. In engineering practice, the selection or combination of these methods is typically based on design stage, importance level of the structure, available data, and required performance objectives.

Even though numerical and experimental researches on the SSSI effects have been developed for the structures, the field is far from mature. Additional experimental and numerical studies need to be performed to contribute to data and seismic design codes on the effects of SSSI in a crowded environment. There are several suggestions for future work presented below.

1. It is a need to expand the diversity of structures with various height, mass, and aspect ratios between structures in order to cover a wide range of layout in a large city environment. The numerical analyses could contribute to the works with nonlinear models of soil, structures, and interfaces.
2. Additional works with robust experimental case studies such as (1) embedment foundations, (2) multi-degree-of-freedom (MDOF) structures, (3) nonlinear structure, (4) different foundation types (i.e., deep and pile foundations), and (5) structure–soil–structure interaction (SSSI) between structure and tunnel besides.
3. Further study should be made on reducing SSSI effects on less massive structures in SSSI. Several solutions could be considered, such as (1) replacing soil beneath structure by an absorbing layer formed by high material damping, (2) reducing transmitted waves from adjacent structures by replacing soil between structures by a high damping mate.

Table 3. Comparison of seismic analysis methods

Method	Accuracy	Computational efficiency	Practical applicability	Nonlinear soil behaviour
Linear static procedure	moderate, assumes fixed-base conditions and ignores kinematic interaction	high, as it simplifies seismic forces into equivalent static loads	suitable for preliminary design and code-based evaluations	limited, does not capture nonlinear soil effects
Linear dynamic analysis	higher than static methods, accounts for modal responses and SFSI modifications	moderate, as modal analysis requires additional computations	applied in performance-based design and seismic response predictions	limited, as it does not include nonlinear behaviour but modifies response spectra
Nonlinear response history procedure	high, captures full time-dependent seismic response and nonlinearities	computationally intensive due to time-step integration	required for complex structures and advanced seismic risk assessments	strong, includes soil damping, foundation flexibility, and interaction effects
Pushover method	moderate, provides inelastic demand estimates	moderate, requires iterative nonlinear analysis	used for performance-based seismic assessment and retrofit design	captures some nonlinear behaviour, but does not fully model time-dependent effects

Source: own elaboration

CONCLUSIONS

This study presents a systematic review of the dynamic response of structures subjected to soil–foundation–structure interaction (SFSI) and structure–soil–structure interaction (SSSI), based on current seismic design codes and numerical simulation approaches. While SFSI has been partially integrated into several modern codes, SSSI remains largely unaddressed, despite its demonstrated impact on structural behaviour during seismic events.

The review indicates that SFSI can reduce seismic demand due to foundation flexibility and energy dissipation through rocking mechanisms. In contrast, SSSI may amplify or attenuate structural response depending on the relative stiffness, mass distribution, and spatial arrangement of adjacent buildings. However, most existing studies address SFSI and SSSI independently and rely on simplified assumptions – particularly linear soil behaviour – which limit the reliability and applicability of their conclusions in real-world conditions.

Key limitations identified include the absence of experimentally validated models that simultaneously capture both SFSI and SSSI, inconsistency in the definition of key parameters, and the lack of practical guidelines for incorporating these effects into seismic design procedures. Future research should focus on the development of integrated numerical models calibrated against physical tests, particularly under nonlinear and layered soil conditions. Such efforts are essential to improve analytical accuracy and facilitate the implementation of interaction effects in performance-based seismic design, especially in high-density urban environments.

CONFLICT OF INTERESTS

All authors declare that they have no conflict of interests.

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