

RESEARCH ARTICLE

Incorporating soil-structure interaction into simplified numerical models for fragility analysis of RC structures

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Abstract

Simplified building models are a valuable option for seismic assessment at the regional scale. These models often use calibrated springs to model column behaviour, and recent advances have made them suitable for capturing torsional response in Reinforced-Concrete Moment-Resisting-Frames. Nevertheless, their validation is typically achieved using fixed-base models, which do not include the influence of soil-structure interaction (SSI). This study introduces a novel approach to quantify the accuracy of a recently developed simplified model while accounting for dynamic SSI, using a newly implemented, refined 3D Finite Element non-linear soil model in OpenSees. The accuracy of the simplified structural model is assessed by comparing the results of non-linear dynamic analyses with those of a refined model in terms of (i) a peak structural demand parameter such as the interstorey-drift ratio and (ii) fragility curves computed from cloud analysis and accounting for collapse cases. The study presents details of the proposed refined approach for 3D soil modelling in OpenSees, focusing on implementing free-field boundary conditions and structure-to-soil connections. Results show that the accuracy of the simplified model is maintained, even in the presence of SSI, and it successfully captures the overall structural response measured at peak demand. For the proposed case study, the difference between the simplified and refined models' fragility curves' medians is 4% and 2% for fixed and SSI models, respectively. The simplified structural model, combined with the refined soil model for SSI effects, presents an innovative and conservative, yet computationally efficient, alternative for seismic risk analysis, even in the presence of structural irregularity.

KEYWORDS

3D soil model, irregular buildings, reinforced concrete structures, seismic fragility, simplified structural model, soil-structure interaction

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1 | INTRODUCTION

Simplified models have proven to be valuable in structural engineering for efficiently estimating both the structural demand and the fragility of buildings (e.g., Refs. 1, 2), especially when a large number of ground motions are applied to the building, as in the case of cloud³ or multiple stripe^{4,5} analyses, or when large-scale seismic assessments with a considerable number of buildings are required (e.g., Refs. 6, 7). More specifically, single-degree-of-freedom (SDOF) and multiple-degree-of-freedom (MDOF) systems have been proposed. While SDOF models offer computational efficiency, MDOF models overcome some of the obvious limitations of SDOF models and provide accurate results while still requiring minimal computational effort compared to refined models. Most approaches consider low-rise, regular buildings, although some studies have proposed methods suitable for tall buildings as well (e.g., Ref. 2). Recently, Blasone et al.⁸ proposed a novel approach that captures the torsional behaviour in low-rise reinforced concrete moment resisting frames (RC MRF) by using a column-based approach relying on calibrated springs with a coupled torsional behaviour at the storey level.

In typical applications, simplified approaches consider fixed-base (FB) models and ignore the influence of soil-structure interaction (SSI). However, the significance of accounting for dynamic SSI is well recognised, although multiple approaches exist to model the soil and the foundation in each application. When SSI is explicitly modelled, questions arise about whether simplified models are still valid and applicable. Given the above-mentioned simplified modelling framework, this paper addresses two critical questions emerging in the context of simplified building models combined with SSI.

One primary concern is whether simplified models maintain their validity in estimating structural demand in the presence of SSI. Estimating the structural demand is crucial for assessing whether a building can withstand the imposed loads and deformations without experiencing damage or failure. As seismic forces interact with the soil, the dynamic response of structures becomes inherently more complex.⁹ Therefore, quantifying the impact of SSI on the predictive accuracy of simplified building models is crucial for ensuring reliable results in seismic-prone regions.

A second question concerns vulnerability assessment. In this context, a usual method is based on fragility curves, which indicate the likelihood of damage at different seismic intensity levels. These curves provide a probabilistic representation of how vulnerable a structure is to seismic events. Multiple studies have investigated the use of fragility curves, including SSI (e.g., Ref. 10). The question addressed herein is whether the introduction of SSI affects the capability to estimate fragility curves with simplified models. The engineering demand parameter employed to represent the global structural response is the maximum inter-storey drift ratio (IDR). IDR has been a widely utilised metric for developing fragility curves using simplified models.¹¹

A state-of-the-art numerical analysis methodology that combines a large three-dimensional (3D) soil model with either a simplified or a more refined building model is used to explore the above questions. The modelling approach for the 3D soil continuum is described in detail, with the objective of establishing a rigorous, cutting-edge methodology for soil modelling in OpenSees that allows horizontal and vertical variation of soil properties. The model is implemented automatically through Python scripting in the visual interface provided by STKO.¹² By investigating the application of the method to a realistic case study, the study aims to provide further insights into understanding and evaluating the advantages and limitations of simplified models in the context of seismic vulnerability assessment, including SSI effects.

2 | METHODOLOGY

2.1 | Motivation and objectives

Conventional approaches to consider dynamic SSI (e.g., Refs. 13–15 as depicted in Figure 1A) typically involve the independent modelling of buildings and soil. Interactions are considered through various methods, including the use of non-linear soil springs (e.g., Refs. 16, 17), lumped-parameter models (e.g., Refs. 18–20) based on impedance functions (e.g., Refs. 21, 22), and macro-elements (e.g., Refs. 23–25). Examples of coupled soil-structure numerical models typically consider sophisticated structural models (e.g., Refs. 10, 26–28). The proposed study seeks to advance these frameworks by considering a coupled modelling approach that integrates a 3D soil continuum model with a simplified representation of the superstructure, as schematically illustrated in Figure 1B. A novel implementation approach has been designed for this purpose to streamline the overall modelling process. The proposed approach allows non-linear dynamic analyses of irregular RC structures while explicitly accounting for (a) wave propagation within a non-homogeneous soil continuum that experiences shear stiffness degradation and damping increase with increasing cyclic shear strain amplitudes, (b) radiation

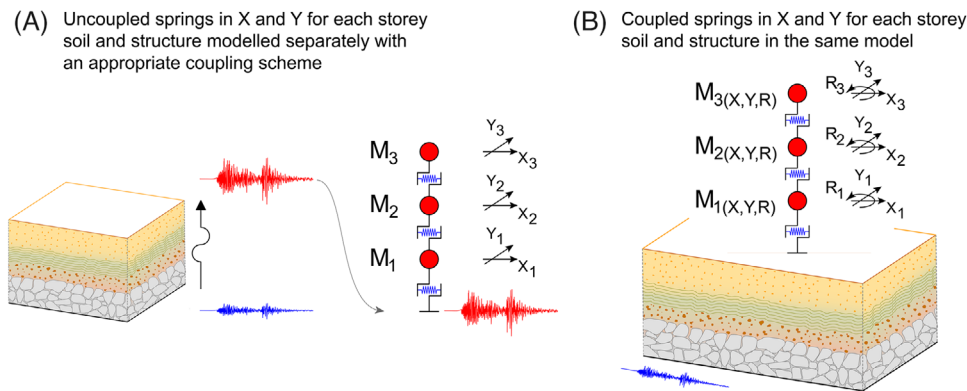


FIGURE 1 (A) Lu et al.¹⁴ method, in which soil and structure are modelled separately with an appropriate coupling scheme; (B) proposed method, in which soil and structure are included in the same model.

damping along the model boundaries and the non-rigid bedrock, as well as (c) SSI. For the superstructure, two different modelling techniques (presented in Sections 2.2 and 2.3) are tested. The coupled model of the structure and the 3D soil continuum is implemented in OpenSees²⁹ using the pre-processing and post-processing tools of STKO,¹² as well as a novel code developed with the programming language Python to speed up the implementation of the non-linear 3D soil elements, the varying properties of the non-homogeneous deposit, and the appropriate boundary conditions into the model. The complete approach is detailed in Section 3.

2.2 | Simplified building model

To optimise modelling efficiency, particularly for large-scale analyses involving multiple buildings, simplified building models are commonly employed using SDOF equivalence (e.g., Refs. 30–32). These models are typically chosen for their ease of implementation and rapid analysis runtime compared to more refined ones, such as standard lumped or distributed plasticity models. The simplifications in these models often compromise accuracy and make them frequently unsuitable for capturing essential features such as torsional effects, shear failures or the presence of infills. In recent years, efforts have been made to overcome the above shortcomings through the development of simplified MDOF models that can capture the main building response characteristics (e.g., Refs. 1, 2, 14, 33, 34). An additional enhancement is the use of a modelling technique suitable for torsional flexible buildings, for example, by replacing the uncoupled storey springs¹⁴ with coupled storey springs. This latter model is explored herein, using the approach recently developed by Blasone et al.⁸ Blasone et al.⁸ use two uncoupled shear force—IDR springs (one for each horizontal direction) for each column of the building. The structure stiffness matrix is assembled by the structural software using the assumption of rigid floor diaphragms, thus implementing a simplified 3D shear model of the building. The two column springs' curves are calibrated through a genetic algorithm based on the results of a more refined structural model (see Ref. 8). Though the overall model is a shear model, the column springs' curves account for the effects associated with the column and the beam flexibility (see Section 4.1 for further details).

2.3 | Refined building model

To benchmark the accuracy of the simplified structural model by Blasone et al.⁸ in capturing the SSI effects when a detailed soil model is included, a refined building model is considered, using a high-fidelity modelling approach for RC MRFs. The refined model is presented in Section 4.2.

2.4 | Soil model and SSI

Modelling the soil as a non-linear 3D continuum allows for the accurate simulation of wave propagation, site amplification, and kinematic and inertial interaction with the superstructure while accounting for soil inhomogeneity and the presence

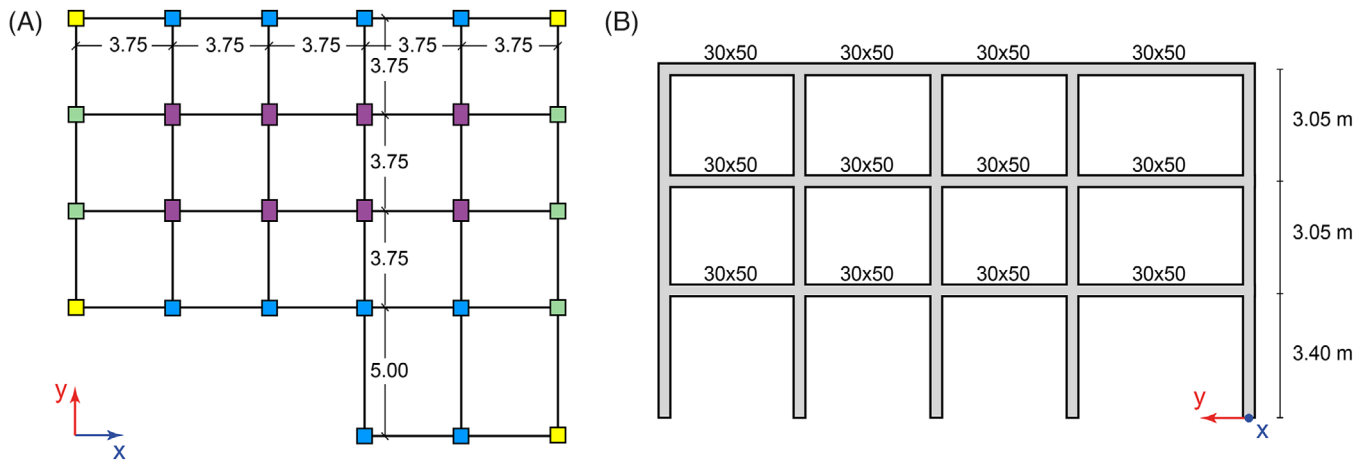


FIGURE 2 Case study building (A) typical floor plan with distinct colours for different sections⁸ and (B) elevation of one of the long frames along the Y direction.

of a non-rigid bedrock. The main assumptions inherited herein include the absence of groundwater, allowing the use of total stress analysis, and the fact that soil dynamic response is governed by vertically propagating SH waves. The numerical modelling approach for the soil continuum is described in Section 4.3. SSI is modelled using appropriate connections with the superstructure model, as presented in Section 4.4.

2.5 | Seismic fragility analysis

Seismic fragility analysis is a valuable tool for assessing the vulnerability of structures under seismic action. Various methodologies, such as incremental dynamic analysis (IDA) proposed by Vamvatsikos and Cornell,³⁵ are commonly employed in research applications for generating fragility curves. IDA becomes impractical due to the computational intensity of the soil model used in this study. Instead, cloud analysis³ is employed for fragility assessment, significantly reducing computational time. This method has proven to be a reliable tool for accurately estimating the seismic fragility of structures.

3 | CASE STUDY

The proposed methodology is applied to the case study of a well-documented structure placed on realistic, non-homogeneous soil conditions. More specifically, the building modelled and validated by Blasone et al.⁸ is considered. The structure's characteristics are discussed in Section 3.1. The selected site is part of the Italian municipality of Benevento, namely an area of particular interest in the northern part of the Southern Apennines seismic belt. The corresponding soil profile has been well characterised by previous studies,^{36,37} thus allowing the development of an accurate 3D model. The soil characteristics are described in Section 3.2.

3.1 | Structure

The case study structure is the “L-shaped” three-storey RC MRF presented in Blasone et al.⁸ The structure is an adaptation of a regular structure designed in the 1970s and presented in Iervolino and Dolce.³⁸ The original floor plan was modified to generate a plan-irregular structure while maintaining regularity in elevation. The resulting building is plan-irregular according to EC8 4.2.3.2 as the area between the outline of the floor and a convex polygonal line enveloping the floor is approximately 22% and, therefore, exceeds 5% of the floor area. Moreover, the structural eccentricity, as defined according to EC8, exceeds the limit of 0.3 for the maximum torsional radius.³⁹ Figure 2 shows the typical floor plan and elevation of the case study building. The building has square column sections in the perimeter, while all internal columns are rectangular. All beam spans are 3.75 m, except for a 5 m bay along the transversal (Y) direction, as indicated in Figure 2A. All beams have the same $30 \times 50 \text{ cm}^2$ sections. Columns at each level have different reinforcement and, in some cases,

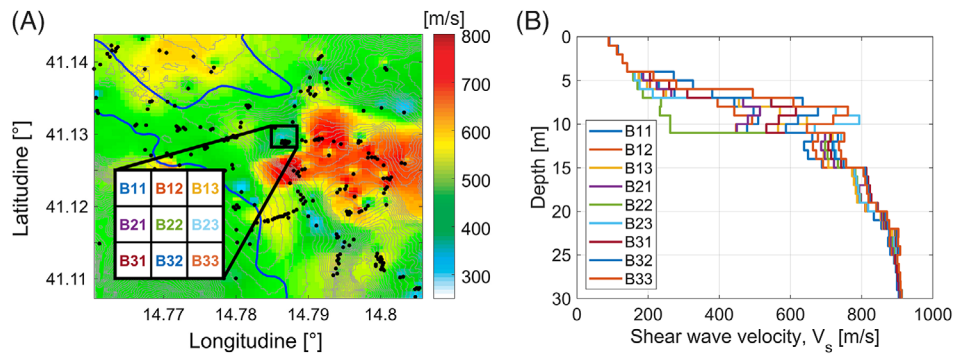


FIGURE 3 (A) $V_{S,30}$ based on refined data with selected soil blocks and (B) corresponding V_S profiles.

change in geometry along the building height. The inter-storey height is 3.40 m on the first floor and 3.05 m on the second and third floors. Detailed drawings of each element section are available in the Appendix of Blasone et al.⁸

3.2 | Soil

For the realistic representation of SSI effects, this study is based on the actual soil conditions of a site situated in Benevento, Italy, which belongs to a seismically active region in the northern part of the Southern Apennine seismic belt.⁴⁰ The soil model used is based on detailed geotechnical data presented by De Risi et al.³⁶ and Penna.³⁷ Two primary data resources were employed to build a refined geotechnical model of the site: open-access data from the U.S. Geological Survey (USGS) Global VS30 Server and 263 site investigations. The first dataset provides maps of average shear wave velocities for the upper 30 meters, $V_{S,30}$, with a resolution of 30 arcseconds, while the second dataset includes 30 Down-Holes and 70 standard penetration tests (SPT). The studied area is approximately 9 km² and is located near the confluence of the Calore and Sabato rivers, ranging in elevation from 118 to 338 m above sea level (Figure 3A). The area is organised into a grid of 50 × 50 m² blocks, with well-documented $V_{S,30}$. In addition, detailed information is available along the depth, in terms of share-wave velocity profiles (V_S), and appropriate shear modulus degradation G/G_{max} and damping increase ξ with cyclic shear strain amplitude γ curves have been assigned to each soil layer of the deposit.

The selected study area is characterised by a heterogeneous geological structure with alternating alluvial and palustrine layers and consists of a 3 × 3 grid of 50 × 50 m² (i.e., a total area of 150 × 150 m²) in the central part of the city of Benevento. Figure 3A,B illustrate the selected region from the refined $V_{S,30}$ map and the corresponding V_S profiles, respectively. The case study structure is positioned on the central block, labelled as B22, which has the lowest average shear wave velocity. Further details of soil properties along the depth are provided in Table A1. The level of information available ensures a suitable characterisation of the horizontal and vertical variability of V_S , allowing the development of a realistic 3D detailed model.

4 | NUMERICAL MODEL

The coupled structure-soil model was entirely implemented in OpenSees using the pre- and post-processing software STKO and its embedded option to automate modelling and analysis procedures using the programming language Python. This option was used to speed up the implementation of the detailed soil model characteristics. As explained in Section 3.1, two modelling approaches were considered for the structure: simplified and refined. These are presented in Sections 4.1 and 4.2, respectively. The soil modelling approach is described in Section 4.3. Finally, the details of connecting the superstructure and subsoil models are discussed in Section 4.4.

4.1 | Simplified model of the structure

In the simplified model, the column-based 3D MDOF model of Blasone et al.⁸ was used. This approach is particularly suitable for the selected case-study structure for two main reasons: it can capture the translational and torsional

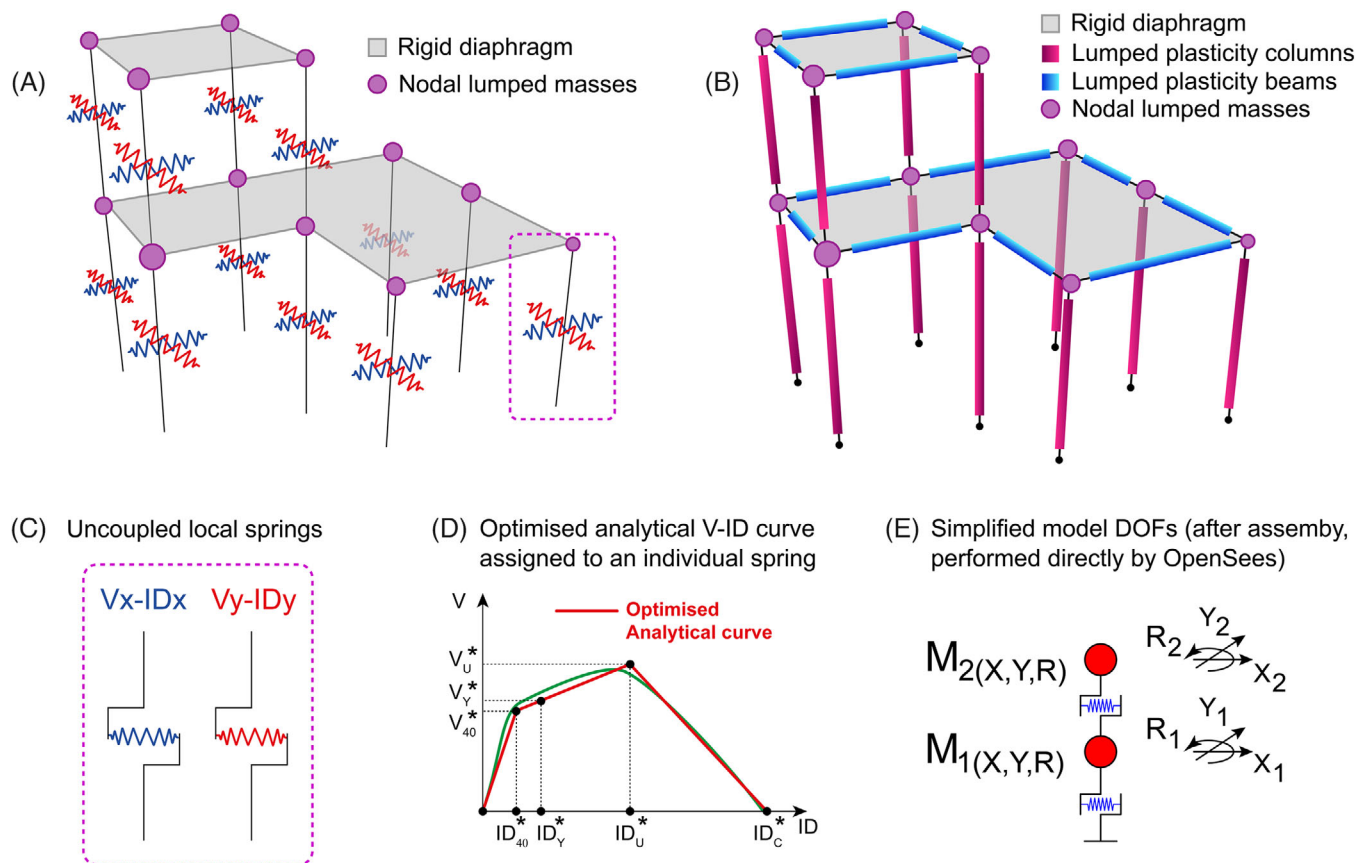


FIGURE 4 (A) Column-based 3D MDOF simplified model; (B) refined model; (C) scheme of uncoupled local column springs; (D) optimised analytical V-ID curve of an individual spring; (E) simplified DOFs after assembly in OpenSees.

behaviour of plan-irregular RC MRFs, and it allows the implementation of a proper SSI model directly in OpenSees. This is because each column is included in the model, though in a simplified manner, through two non-linear translational springs. Coupled torsional behaviour is thus obtained at the storey level, while the local behaviour remains uncoupled and easier to model. The resulting simplified model is 3D, with a rigid diaphragm constraint at each storey. Figure 4A,C show the column-based simplified model with the two springs for each column. The non-shear type behaviour is achieved by calibrating the columns' backbone curves for shear-drift (V-ID) using a genetic algorithm (Figure 4D) applied to the results of static analyses on a more refined model. After assembly, performed directly by OpenSees, the model has only three degrees of freedom (DOFs) per storey, as shown in Figure 4E. Blasone et al.⁸ validated the simplified model for both non-linear static and dynamic analyses. Over 150 unscaled ground motions were extracted from the SIMBAD database and employed. The analyses of the simplified model were, on average, 10–20 times faster than the corresponding analyses of a benchmark refined model, with peaks that reached even 100 times. This makes the simplified model suitable for applications where many ground motions are applied to the building (and possibly to many buildings).

4.2 | Refined model of the structure

In the refined approach, a lumped-plasticity model was used (Figure 4B). This implies a simplification compared to distributed plasticity, but the differences are assumed to be negligible since the considered building has only three storeys. Shear failures were accounted for through a pre-classification approach,⁴¹ and the *hingedBeam* element in STKO was used to model both beams and columns. The plastic hinges were modelled using the cyclic model with degradation *ModIMK-PeakOriented* by Lignos and Krawinkler.⁴² Axial loads were added as concentrated forces, and masses were added as nodal masses, equal in both horizontal DOFs. A floor diaphragm constraint was applied at each storey level. The refined model

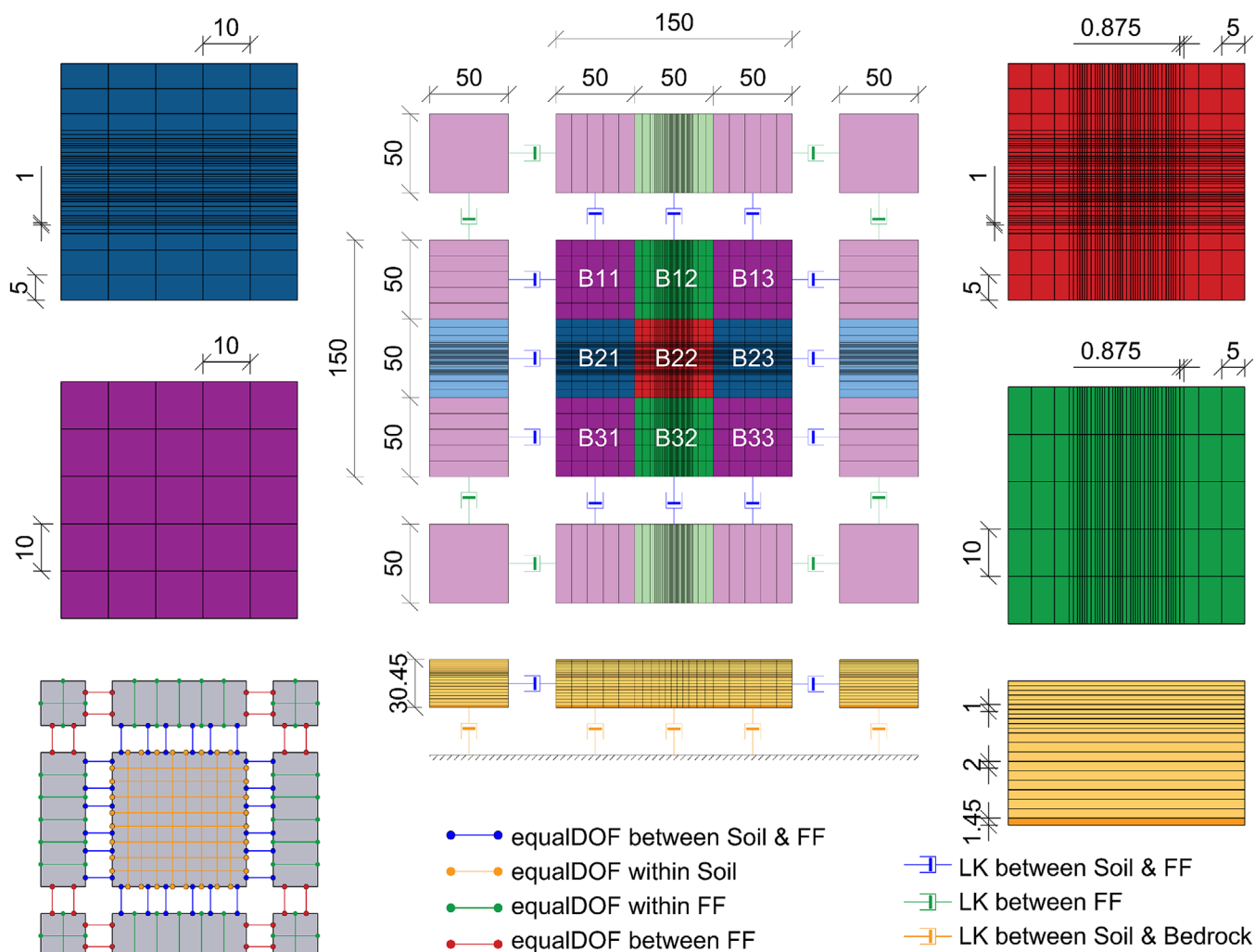


FIGURE 5 Soil modelling approach: Dimensions, constraints and mesh details (all units are in metres).

was used as a benchmark for the simplified model (see Ref. 8 for modelling details). The first three periods of the refined model are 0.71, 0.70 and 0.67 s, while those of the simplified model are 0.74, 0.70 and 0.68 s, respectively.

It must be noted, however, that the reference structure's fundamental period was notably higher (~ 1.8 s) in the associated previous study.⁸ This is because the focus of that study was to validate a simplified methodology for highly irregular structures; thus, the masses were chosen specifically to emphasise the irregularity. In the present study, the focus is not on structural irregularity per se but rather on demonstrating that the simplified modelling approach remains reliable for SSI studies. Consequently, the mass distribution has been adjusted to reflect typical low-rise buildings, resulting in changes to the corresponding periods and a subsequent slight increase in SSI effects with respect to what would have been observed for a structure with the same mass distribution as Blasone et al.⁸

4.3 | Soil model

In this study, a 3D soil model is used with heterogeneous properties across both the horizontal and vertical directions. The numerical model consists of $3 \times 3 = 9$ soil blocks, each measuring $50 \times 50 \text{ m}^2$ in the horizontal directions ($150 \times 150 \text{ m}^2$ in total), as shown in Figure 5. It is noted that the soil model width is more than five times the total width of the building, thus ensuring negligible boundary effects. In the vertical direction, the 30 m of soil is simulated using 20 element layers (11 $\times 1$ m layers and 9 $\times 2$ m layers) as well as one layer representing the elastic bedrock. The latter is included to facilitate the simulation of wave reflections, as well as radiation damping. The choice of different vertical layering was based on the variability of the shear wave profiles, as shown in Figure 3.

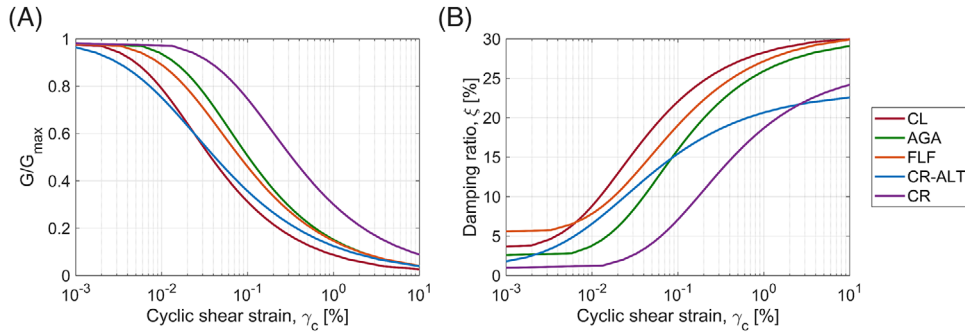


FIGURE 6 Material properties: (A) shear modulus degradation and (B) damping versus shear strain curves.

To effectively capture the wave propagation and energy dissipation mechanisms of the infinite media in the lateral directions, the model integrates free-field (FF) boundaries on all four sides and corners, represented as shear columns (Figure 5). Each element in these columns has the same properties as the adjacent elements of the main blocks. The FF boundaries are connected to the soil boundaries through zero-length (ZL) dashpots, with their properties calculated according to Lysmer and Kuhlemeyer.⁴³ In addition, Lysmer-Kuhlemeyer (LK) boundaries are also applied below the bedrock layer to account for base radiation damping. To ensure consistent deformation along the soil borders, equal degrees-of-freedom (equalDOF) constraints are imposed at the soil boundaries connecting opposite faces of each layer in both horizontal and vertical directions, as illustrated in Figure 5.

Figure 5 provides detailed information on the mesh properties. Soil and FF are modelled with eight-node *stdBrick* elements. In the vertical direction, the discretisation size ensures an adequate number of elements per wavelength, thus, an accurate simulation of wave propagation within the soil medium. Shear wave velocities are small up to a depth of 11 metres, as visible in the corresponding profile (Figure 3). A 1 m-deep meshing is thus applied. Below that level, shear wave velocities increase, thus resulting in larger wavelengths and a coarser discretisation size of 2 m is used. Regarding the horizontal direction, a finer mesh size is employed near the structure to ensure sufficient discretisation below each foundation. Dimensions vary across this area to accommodate the location of the foundation nodes within the defined building gridlines. Further details on the foundation and soil-structure coupling details are discussed in Section 4.4.

To capture the non-linear soil behaviour, the *PressureIndependentMultiYield* (PIMY) material developed by UC San Diego is used.⁴⁴ During gravity analysis, this material exhibits linear-elastic behaviour. However, in dynamic analysis, multi-surface plasticity is activated. Yield surfaces are defined based on the Von Mises yield criterion, with apertures and strain hardening properties computed based on the RO model by Ramberg and Osgood.⁴⁵ The latter was calibrated to reproduce the shear strength degradation $G/G_{\max}$ and damping increase ξ of the soil material with increasing cyclic shear strain amplitude γ , as illustrated in Figure 6A,B and Equations (1) and (2):

$$\gamma = \left[\frac{1 - \frac{G}{G_{\max}}}{C \cdot \left(\frac{G}{G_{\max}} \right)^R} \right]^{\frac{1}{R-1}} \quad (1)$$

$$\xi = \xi_0 + \frac{2}{\pi} \cdot \frac{R-1}{R+1} \cdot \left(1 - \frac{G}{G_{\max}} \right) \quad (2)$$

where ξ_0 is the minimum damping, and C and R are the Ramberg-Osgood parameters provided in Table A1 for each soil type.

The ground profile comprises five distinct types of soils along its depth: coarse debris colluvium (CL), deep Pliocene clay (AGA), fine fluvial lacustrine (FLF), weathered Rissian conglomerate (CR-ALT), and cemented Rissian conglomerate (CR), with different $G/G_{\max}$ and ξ versus γ curves defined for each soil type. Specific locations of these soils are detailed in Table A1. No topographic irregularities are considered in this study since the selected region has minimal elevation variability. Further information, including mechanical properties and the digital elevation model (DEM), is presented by De Risi et al.³⁶

To validate the soil model, a site response analysis is carried out by applying both components of the record with the highest peak ground acceleration (PGA), as shown in Figure 7A. During gravity analysis, the bottom faces are fixed in all

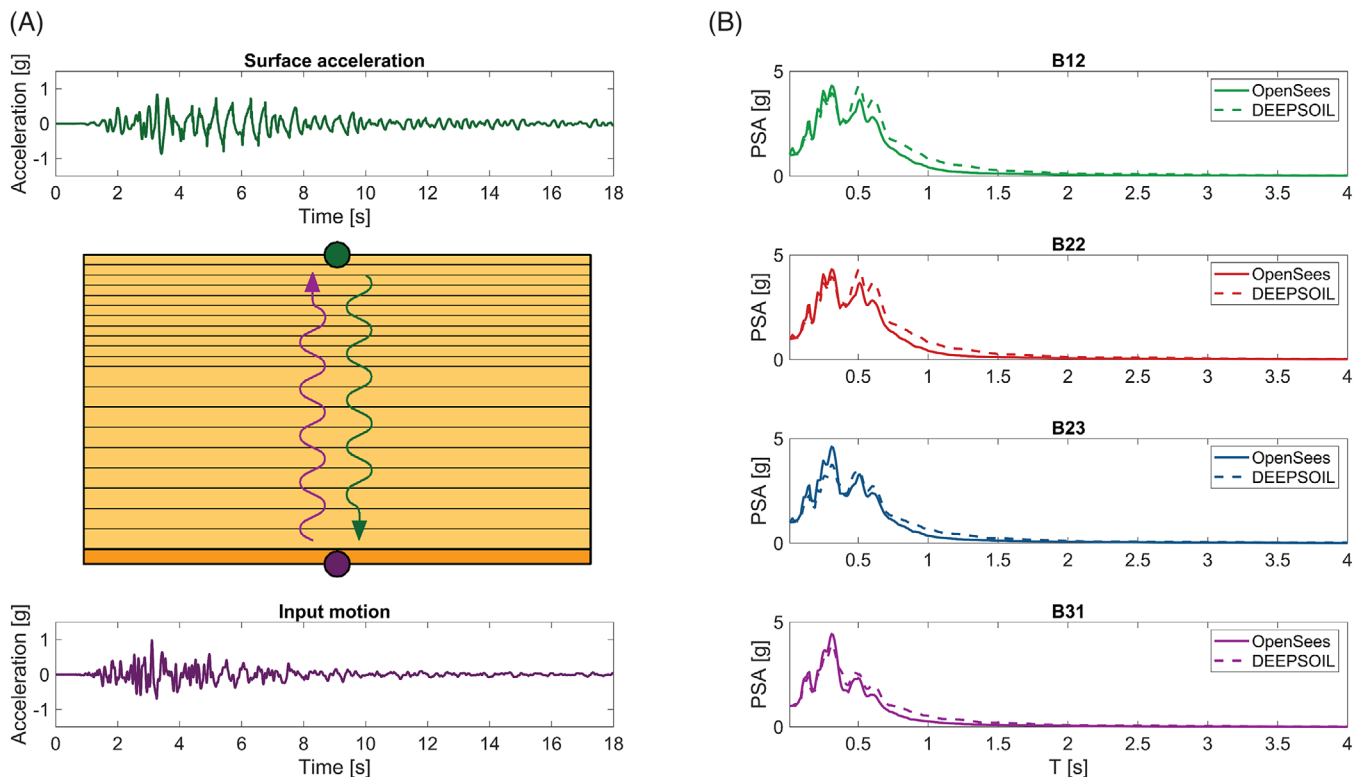


FIGURE 7 (A) Soil amplification and (B) surface acceleration validation versus DEEPSOIL.

DOFs. Then, prior to dynamic analysis, this constraint is released, and a *ForceFromReaction* load pattern is applied to the same DOFs to maintain static equilibrium. Minimum damping is introduced in the form of Rayleigh damping, set to a value of 1%⁴⁶ at the frequencies corresponding to the natural frequencies of the structure (0.6 Hz) and soil (3 Hz). Dynamic loads are then introduced as velocity time histories, accompanied by *DistributedLK3D* constraints to capture the damped soil response. The accuracy of the results is verified through comparison with the predictions of DEEPSOIL,⁴⁷ employing the *Discrete Points* soil model, where the shear modulus reduction and damping increase curves were fitted to the RO curves, as shown in Figure 6. As shown in Figure 7B, a fairly good agreement is obtained, confirming the validity of the proposed model. Additionally, a comparison of results across soil blocks with varying mesh details is carried out to verify that the mesh size is adequate to capture the soil behaviour. Notably, the differences between the results for the block with the coarsest mesh size (B31) and the one with the finest mesh size (B22) are negligible. The slight differences observed in the non-linear response are primarily due to the different material properties in the two codes (Figure 6). However, it must be noted that while the non-linear model accounts for shear modulus degradation and damping increase at small to medium strain levels, it may not accurately represent failure conditions. Given the high factor of safety against bearing capacity (i.e., around 2), sub-soil failure remains highly unlikely in the case examined herein.

4.4 | Soil-structure interaction

Figure 8A illustrates the SSI model via a close-up view of the refined structure foundations. It should be recalled that the column element nodes have both translational and rotational DOFs, while the *stdBrick* soil elements only have translational DOFs. Therefore, a simple connection would not allow for moment loads to be transferred to the foundations. To remedy this issue, the model incorporates *BeamSolidCoupling* constraints at the base of each column, establishing an interface with the underlying soil.¹² These constraints create a rigid link between the base node of the column (the master node) and the top nodes of the *stdBrick* soil elements that correspond to the foundation (the slave nodes), thus allowing for both translational and moment forces to be transferred (Figure 8B).

The above configuration also defines the dimensions of the structure's foundations, which play a significant role in SSI effects. More specifically, 2 × 2 m square footings were assumed, providing reasonable safety factors (i.e., around 2) for

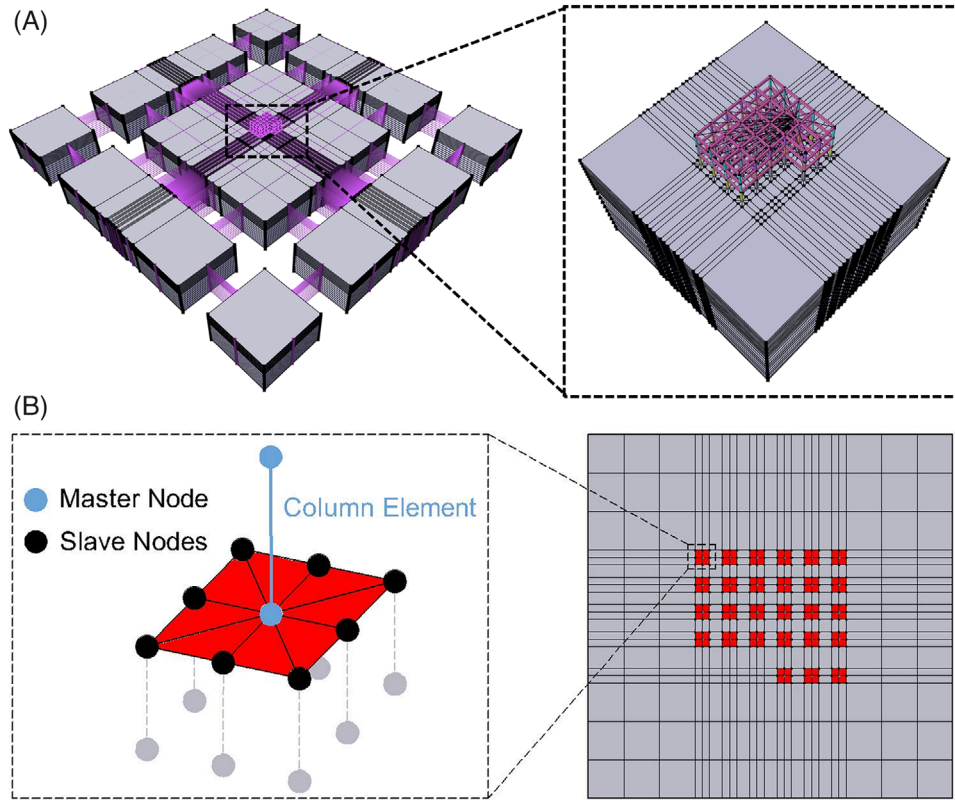


FIGURE 8 (A) Close-up view of the soil block with the structure and (B) foundation details with the *BeamSolidCoupling* element.

the given building loads and soil conditions. The mesh size is adjusted to accommodate these dimensions, as described in Section 4.3. The accuracy of this technique in capturing SSI was later validated by applying vertical (z), horizontal (x or y), rocking (rx or ry) and torsional (t) displacements to the master node of the element constraints, measuring the corresponding reactions and comparing the resulting stiffness against available analytical expressions for the square footings (2B by 2B) on equivalent homogeneous soil deposits.^{15,21,22} The calculations are based on Equations (3)–(6):

$$K_z = \frac{4.54GB}{1 - \nu} \quad (3)$$

$$K_x = K_y = \frac{9GB}{2 - \nu} \quad (4)$$

$$K_{rx} = K_{ry} = \frac{3.6GB^3}{1 - \nu} \quad (5)$$

$$K_t = 8.3GB^3 \quad (6)$$

Here, $\nu = 0.3$ represents the Poisson's ratio for soil, $G = 1.56 \times 10^5$ kPa is the shear modulus of the soil (chosen to ensure the resulting foundation stiffnesses are close to the heterogeneous case), and $B = 1$ m is the half-width of the foundation.

The accuracy of the numerical analysis methodology was then validated for an equivalent homogeneous case, for which well-established analytical formulations are available in the literature. To define a uniform shear modulus for this equivalent homogeneous case, the static foundation impedances of the inhomogeneous model were first numerically evaluated, and a uniform shear modulus that produced similar impedance values was subsequently selected.

To facilitate the comparison between numerically and analytically obtained stiffnesses under different loading directions, the concept of the equivalent foundation half-width, denoted as B_{eq} , is employed. This parameter represents the foundation half-width that would produce the same foundation stiffness as calculated in different loading scenarios. More specifically, the foundation half-widths required to match the stiffnesses were calculated for each mode obtained from numerical analysis with those estimated from the analytical formulas using backward calculation. This process involved

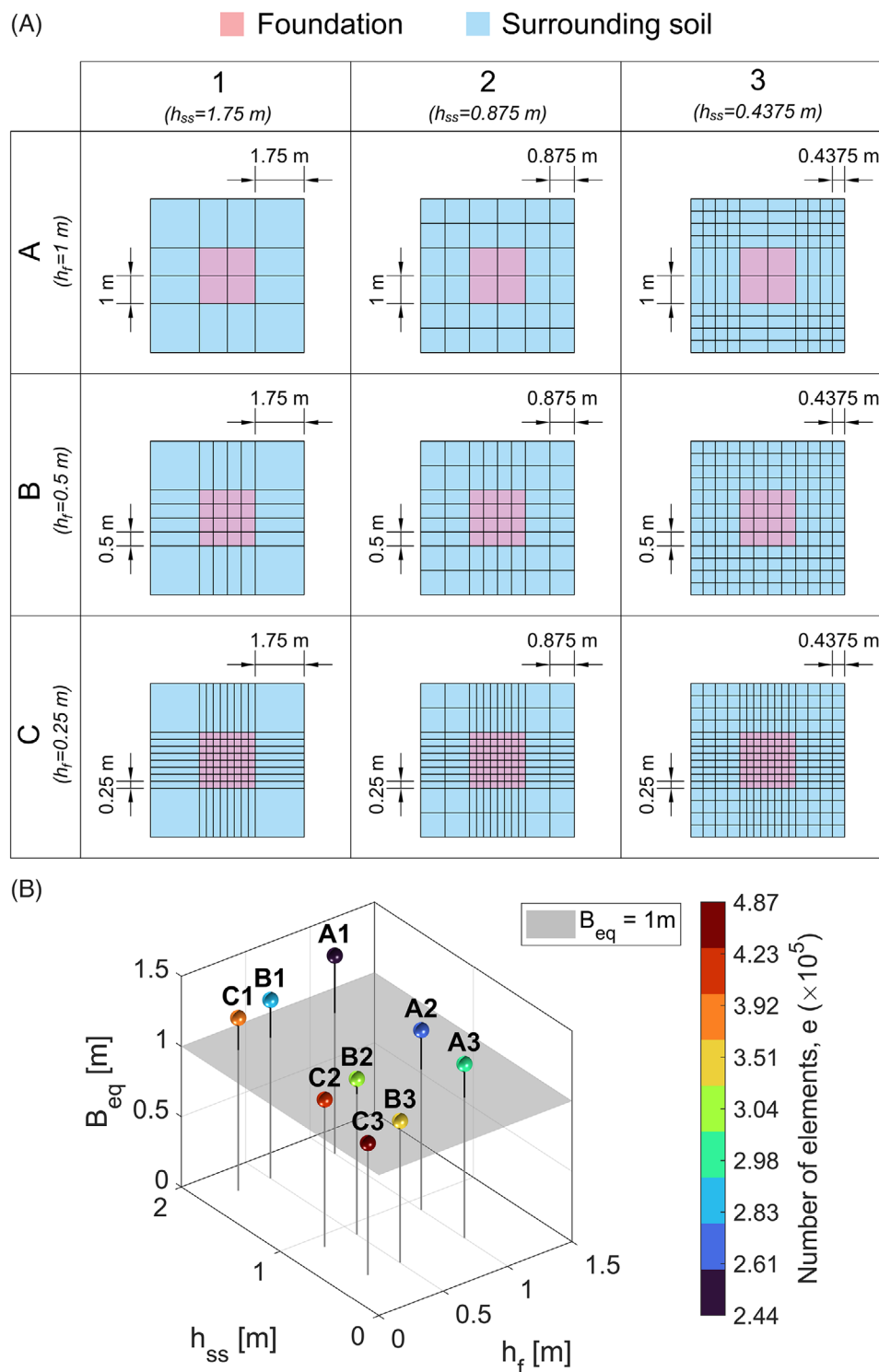


FIGURE 9 Mesh sensitivity analysis: (A) Different mesh configurations and (B) accuracy of varying mesh sizes.

adjusting the numerical model's foundation width until the stiffnesses predicted by the numerical model matched those estimated by the analytical formulas. The equivalent foundation half-width (B_{eq}) represents the average of these four values, providing a single representative value for comparison.

As depicted in Figure 9, decreasing either the surrounding soil mesh size (h_{ss}) or the foundation mesh size (h_f) affects the equivalent foundation half-width corresponding to the analytical predictions (B_{eq}), bringing it closer to the expected analytical value of 1 m. By refining the mesh sizes, the accuracy of the numerical model is improved, achieving a B_{eq} value

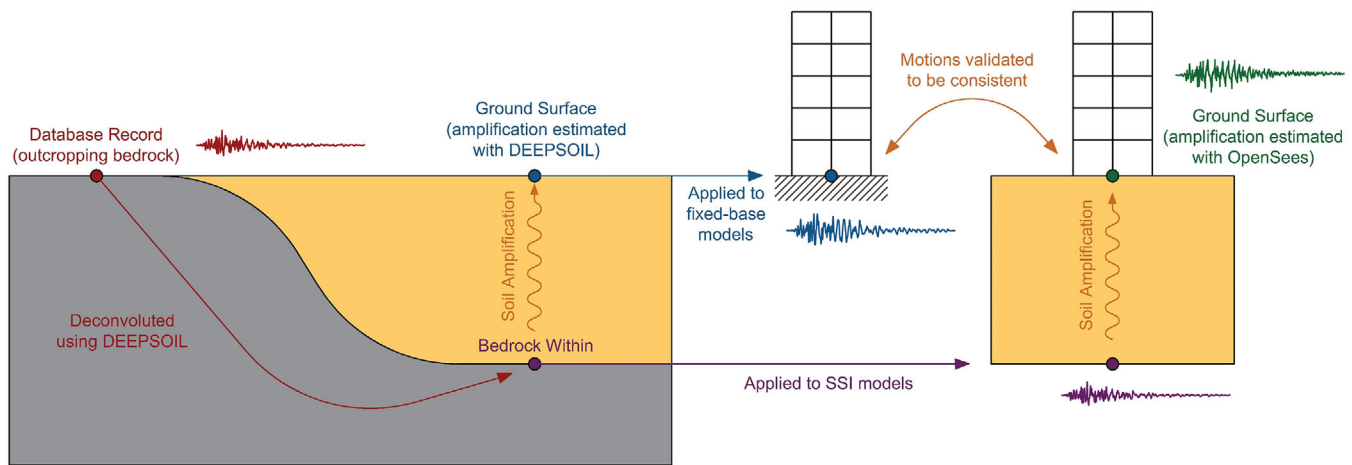


FIGURE 10 Schematic diagram of record processing.

closer to this target. The optimal configuration comprised 3.04×10^5 elements, which yielded a B_{eq} of 1.11 m (denoted as 'B2' in Figure 9), balancing computational efficiency and simulation accuracy. Further reduction of the mesh size would lead to very long computational times while only achieving minimal improvements in simulation accuracy, reinforcing the decision to adopt this optimal solution.

Prior to conducting the time history analysis, a standard push-over analysis with a first-mode distribution shape was performed to assess the changes in lateral load resistance due to SSI. The initial stiffnesses derived from the push-over curves indicate a decrease of up to 17.2%, associated with a change in base shear at which softening occurred. The results suggest that although SSI effects reduce structural stiffness, the reduction is not substantial for the case study at hand. Other studies in the literature have observed more significant stiffness reductions (e.g., Refs. 10, 28). This is typically the case for stiff (e.g., modern and code-conforming) structures on highly deformable soils (e.g., class D soils or soils with low $V_{S,30}$ values within the class C range).

The SSI effects observed in this study are influenced by the specific characteristics of the structure, foundation, soil and seismic excitations. The explored case study, based on realistic data, involves a mix of B and C soil classifications and a three-storey substandard building with a relatively high period (~ 0.7 s). Considering that SSI effects are more pronounced in softer soils and stiffer structures,^{15,48} the examined structure is anticipated to experience limited period elongation. Moreover, given that the examined ground motions feature predominant periods that remain lower than the eigen-period of the structure, SSI is expected to act beneficially rather than detrimentally (Mylonakis and Gazetas, 2000,^{15,49,50} further emphasising the importance of considering the specific case-study characteristics while interpreting the results.

5 | RESULTS

The SSI effects on the refined and simplified models are assessed through a comparison with the results of the corresponding FB models subjected to the same ground motion inputs. Non-linear dynamic analyses are carried out, and cloud analysis is employed to estimate fragility curves of the structures for the different modelling approaches: (i) FB refined, (ii) FB simplified, (iii) SSI refined and (iv) SSI simplified (with [iii] and [iv] including the soil model described in Section 4).

5.1 | Ground motions

The ground motion records are selected from the 150 ITACA outcropping bedrock records used in the Benevento study by De Risi et al.³⁶ The processing of the records is schematically shown in Figure 10. Initially, the database records are deconvoluted to the *bedrock within* motion using DEEPSOIL, with the same ground properties as in the case study. These deconvoluted motions are then (a) directly imposed on the base of the 3D SSI models *iii* and *iv* in OpenSees and (b) applied to the base of a DEEPSOIL analysis with the same soil profile (B22 shown in Section 3.2) to generate amplified records at

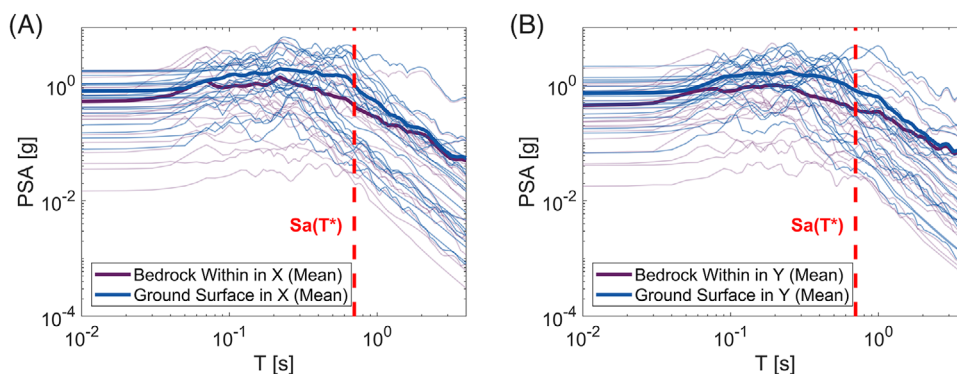


FIGURE 11 Response spectra of the selected records in (A) x and (B) y directions.

the *ground surface*, which are subsequently imposed to the FB models *i* and *ii*. In this way, it is ensured that the buildings in the two types of analyses (i.e., FB and SSI) are subjected to similar excitations, and the corresponding results are directly comparable. The amplified records at the *ground surface*, estimated through DEEPSOIL and OpenSees, are compared and validated for consistency.

The ground motion set consists of 20 records with components applied in both the *x* and *y* directions. These records are selected to cover a wide range of intensity measures at the period of interest (T^*), comply with the minimum value suggested by ASCE/SEI 7-16,⁵¹ and consider record-to-record variability as discussed by Jalayer et al.³ and Jalayer et al.⁵² The mean of the response spectra of these records is also aligned with the design spectrum for a return period of 475 years. The response spectra of the processed records (*bedrock within* and *ground surface*) in the *x* and *y* directions are illustrated in Figure 11A,B, respectively (see Table A2 for record details). The period of interest (T^*) is determined as the mean of the natural period of the refined and simplified structures (0.7 s) in the *x* and *y* directions. Spectral acceleration values at this period serve as an intensity measure (IM) for the subsequent cloud analysis, detailed in Section 5.3. Each model is characterised by a slightly different period, but the selected IM is the same for all structures for the purpose of consistent comparison (see Ref. 53).

5.2 | Structural response

The capability of the simplified FB model to capture the non-linear time-history (NLTH) analysis results of the refined FB model has been validated by Blasone et al.⁸ using 150 records selected from the SIMBAD database. In this study, NLTH analyses are carried out with the new ground motion set discussed in Section 5.1 for all four models (simplified FB, refined FB, simplified SSI and refined SSI). IDRs are recorded for each node in each storey, and the maximum IDR in each storey is determined for both the *x* and *y* directions. The median (η) and standard deviation (β) of the maximum IDR values (IDR_{max}) for each storey are then calculated for every model and compared against each other, as illustrated in Figure 12.

To determine whether the differences are statistically significant, the Aspin-Welch hypothesis test^{54,55} was performed. In this test, the null hypothesis (H_0) states that the median IDR values for both models are equivalent, implying no significant difference (i.e., p -value > 0.05). Conversely, the alternative hypothesis (H_1) suggests that the median IDR values for the two models are significantly different, as indicated by p -values of less than 0.05. For the FB models (Figure 12A,B), the results exhibit good agreement with each other in every storey, consistent with the observations made by Blasone et al.⁸ The highest IDR values are obtained in the first storey, resulting in higher dispersion ($\eta + \beta$ reaching beyond 5%) for both cases. The Aspin-Welch hypothesis test yields p -values of 0.59 and 0.86 in the *x* and *y* directions, respectively, in the first storey, indicating a lack of significant difference. In contrast, for the SSI models (Figure 12C,D), the differences in results are relatively more significant, even though they still show similar trends. Once again, the highest IDRs are obtained in the first storey, where the highest dispersion occurs ($\eta + \beta$ exceeding 5%), and p -values of 0.73 and 0.47 in the *x* and *y* directions, respectively. These p -values suggest that the differences between the refined and simplified models are not statistically significant. Overall, despite the increasing discrepancies, both models can capture the structural behaviour and the changes in the response due to SSI effects. This is further supported by the fragility analyses discussed in the following section.

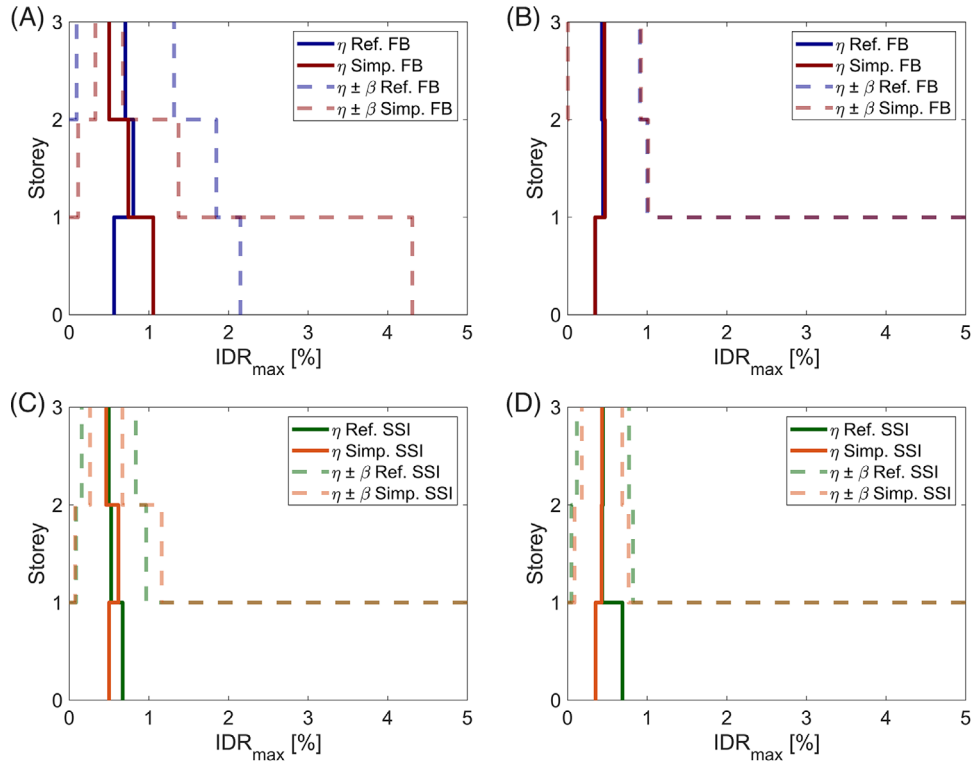


FIGURE 12 Statistical comparison of the inter-storey drift ratios for FB models in (A) the x and (B) the y direction, and SSI models in (C) the x and (D) the y direction.

5.3 | Fragility analysis results

Cloud analysis is performed to evaluate structural fragility, which employs linear regression for the assessment.³ The IDRs are normalised with respect to $IDR_{LS} = 2\%$, which is the threshold IDR for the life safety performance level according to FEMA 356 guidelines⁵⁶ used here as a capacity metric for the structure. The normalised value $Y = IDR_{max}/IDR_{LS}$ represents the demand-to-capacity ratio (DCR) in cloud analysis. For consistency (see Section 5.1) the input ground motion IM is $S_a(T_I^*)$, which is the maximum spectral acceleration at the period of interest in the x and y directions.

Instances of structural collapse and/or dynamic instability leading to very high deformations are often encountered due to the wide range of intensities required for cloud analysis. These *collapse* (C) cases are integrated into the cloud analysis alongside *non-collapse* cases (NoC), employing a 5-parameter fragility model proposed by Jalayer et al.⁵² and used for collapse cases in masonry by Cross et al.⁵⁷ This model combines linear regression with logistic regression to establish the final fragility. Initially, a 2-parameter fragility model is applied to the NoC cases, as shown in Equations (5), (6) and (7). Subsequently, logistic regression is carried out for the C cases, as expressed in Equation (8). The final fragility analysis is then performed using Equation (9). The probability of DCR exceeding unity given collapse ($P(Y > 1 | S_a, C)$) is assumed to be one.

$$\log \eta_{Y|S_a} = \log a + b \log S_a \quad (7)$$

$$\sigma_{\log Y|S_a} = \sqrt{\frac{\sum_{i=1}^n \left(\log Y_i - \log \eta_{Y|S_a,i} \right)^2}{n-2}} \quad (8)$$

$$P(Y > 1 | S_a, NoC) = \Phi \left(\frac{\log \eta_{Y|S_a}}{\sigma_{\log Y|S_a}} \right) \quad (9)$$

$$P(C|S_a) = \frac{1}{1 + e^{-(\alpha_0 + \alpha_1 \cdot \ln(S_a))}} \quad (10)$$

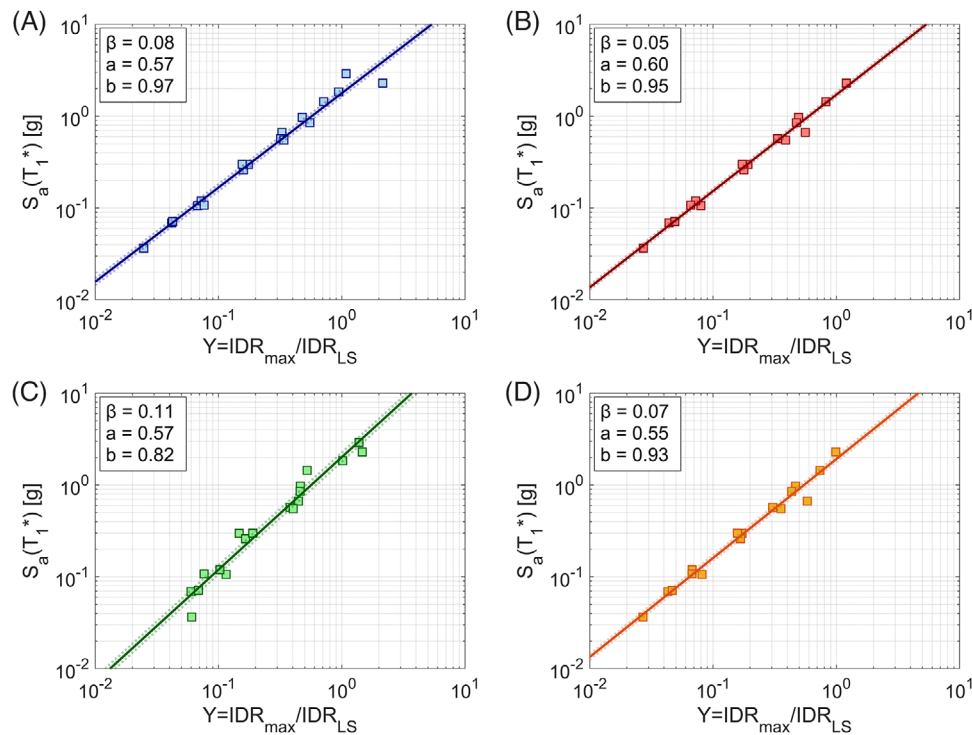


FIGURE 13 Cloud analysis results: linear regression of NoC results for (A) FB refined, (B) FB simplified, (C) SSI refined and (D) SSI simplified cases.

TABLE 1 Final fragility analysis parameters.

Case	No. of collapses	Median, η [g]	Standard deviation, β
Refined FB	2	1.806	0.084
Simplified FB	4	1.729	0.069
Refined SSI	2	1.916	0.112
Simplified SSI	4	1.885	0.077

$$P(Y > 1|S_a) = P(Y > 1|S_a, NoC) \cdot (1 - P(C|S_a)) + P(Y > 1|S_a, C) \cdot P(C|S_a) \quad (11)$$

Here, $\eta_{Y|S_a}$ is the median of Y given S_a , $\sigma_{\log Y|S_a}$ is the logarithmic standard deviation of Y given S_a , a and b are the linear regression parameters, n is the number of NoC cases, and α_0 and α_1 are the logistic regression parameters.

For all the models used in this study, four collapse cases and seven non-collapse cases are observed. The cloud analysis results for NoC cases of (a) FB refined, (b) FB simplified, (c) SSI refined and (d) SSI simplified models are shown in Figure 13. Square markers represent data points, solid lines indicate linear regression fits and dashed lines show the linear regression fit with unit logarithmic standard deviation. The three parameters in these figures are used to generate the corresponding fragility curves for FB and SSI models, as presented in Figure 14. Before incorporating collapse cases, median capacities indicate a 5.0% difference between FB results and a 4.7% difference between SSI results.

The logistic regression fit for the collapse cases is illustrated in Figure 14C, with the record IDs representing a combination of station codes and waveform IDs detailed in Table A2. Following the logistic regression, the final fragility analysis is carried out, as shown in Figure 14D. The parameters of the final fragility curves are provided in Table 1. Median capacities show a 4.2% difference between the FB models and a 1.7% difference between the SSI models. Hence, even when considering SSI effects, the distinction between the results of the simplified and refined modelling approaches remains small and fully acceptable, supporting the validity of the study's hypothesis.

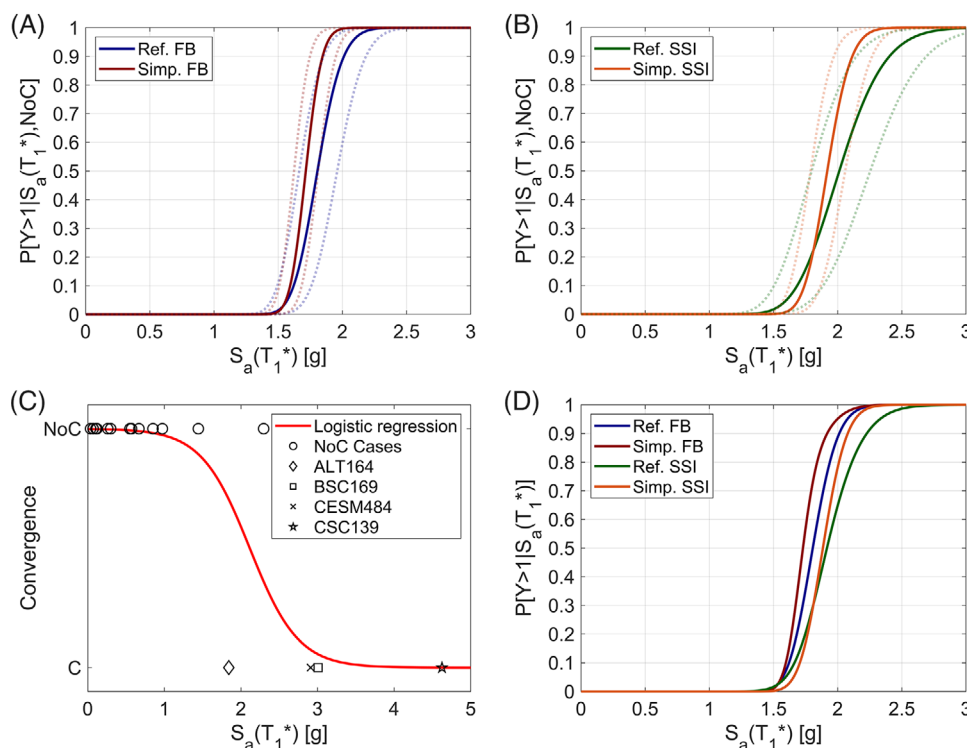


FIGURE 14 Cloud analysis results: fragility curves of (A) FB models and (B) SSI models for NoC cases, (C) logistic regression and (D) final fragility curves for the combined cases (dashed lines in (A) and (B) represent the fragility curves).

6 | CONCLUSIONS

A fragility analysis is presented for an irregular RC structure on a non-homogeneous soil deposit. The case study building is an adaptation of an existing structure already proposed in a previous study. The soil parameters are those of a real site in Benevento, Italy, and relevant excitations are used. The dynamic soil response and the effects of SSI are evaluated using a newly implemented, detailed 3D Finite Element soil model in OpenSees, which accounts for soil non-linearity (shear stiffness degradation and damping increase with cyclic shear strain amplitude), boundary conditions (wave dissipation using a combination of dashpots and free-field elements), and the column-to-soil connection (via appropriate DOF constraints). The superstructure is initially modelled using the innovative simplified approach proposed by Blasone et al.,⁸ and the relevant results are then validated against analyses carried out with a refined building model. The outcomes of the study can be summarised in the following:

- The proposed continuum soil model was validated in terms of soil amplification and sensitivity analysis. The results indicate that the model is a robust numerical tool that can be used for future research applications involving realistic, heterogeneous soil conditions.
- The integration of DOF-coupling constraints in the connection of the building column bases with the soil nodes corresponding to the foundation presents a comprehensive and effective way for implementing SSI effects when modelling the soil as a continuum.
- The stiffness degradation due to SSI does not significantly alter the overall structural behaviour under the conditions studied. Nevertheless, it is important to note that these results are specific to the chosen parameters and models, which include a relatively high-period structure founded on medium-stiffness soil deposit.
- The simplified building model accurately captures the overall structural response, both with and without considering SSI effects. Minor discrepancies are observed in the fixed-base analysis, with p -values of 0.59 and 0.86 of an Aspin-Welch test on medians in the x and y directions, respectively, indicating no significant differences in interstorey drift ratios. In the SSI model, the p -values are 0.73 and 0.47 in the x and y directions, respectively, showing similar, non-significant statistical differences.

- In terms of fragility analysis, the variation between FB and SSI models is rather small and does not exceed 4.2%. This highlights the practical applicability of the simplified modelling approach for assessing the fragility of RC structures, demonstrating its effectiveness across FB and SSI scenarios.

This study further demonstrates the applicability of the considered simplified model for seismic assessment analysis of RC structures, with and without SSI effects, introducing a novel approach for ambitious large-scale seismic risk assessments integrating SSI effects. This study covered an irregular moment-resisting frames, future work should address different structural configurations, soil conditions and refine the proposed model to enhance its adaptability and suitability for wider applications.

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DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from the corresponding author upon reasonable request.

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APPENDIX

TABLE A1 Soil properties.

Block	EC8 soil type	$V_{s,30}$ [m/s]	ρ [ton/m ³]	Material ID*				
				CL	AGA	FLF	CR-ALT	CR
B11	B	367.4	1.73293	0–12 m	12–16 m	16–20 m	20–21 m	21–30 m
B12	C	358.1	1.73293	0–12 m	12–16 m	16–20 m	20–21 m	21–30 m
B13	B	365.7	1.73293	0–9 m	13–16 m	16–20 m	20–21 m	9–13 m, 21–30 m
B21	B	368.0	1.73293	0–8 m, 10–12 m	12–16 m	16–20 m	8–9 m, 20–21 m	9–10 m, 21–30 m
B22	C	320.7	1.73293	0–12 m	12–16 m	16–20 m	20–21 m	21–30 m
B23	B	365.9	1.73293	0–8 m	15–16 m	16–20 m	8–9 m, 20–21 m	9–15 m, 21–30 m
B31	B	383.1	1.73293	0–8 m, 10–12 m	12–16 m	16–20 m	8–9 m, 20–21 m	9–10 m, 21–30 m
B32	B	400.4	1.73293	0–8 m	13–16 m	16–20 m	8–9 m, 20–21 m	9–13 m, 21–30 m
B33	B	398.7	1.73293	0–6 m	15–16 m	16–20 m	6–7 m, 8–9 m, 20–21 m	7–8 m, 9–15 m, 21–30 m
Ramberg-Osgood parameters			ξ_0 [%]	3.0	2.0	5.0	1.0	0.5
			C	552,591	170,899	47,533	10,308	7041
			R	2.54	2.59	2.38	2.09	2.38

TABLE A2 Ground motion set (where M_w indicates moment magnitude and PGA peak ground acceleration).

Waveform ID	Station code	Date	M_w	Bedrock within		Ground surface	
				PGA _x [g]	PGA _y [g]	PGA _x [g]	PGA _y [g]
169	BSC	23/11/1980	6.9	0.93	0.80	1.77	1.26
164	ALT	23/11/1980	6.9	0.51	0.54	1.06	0.95
139	CSC	19/09/1979	5.8	1.35	1.97	1.72	2.14
1175	STB	26/10/2006	5.8	0.26	0.19	0.60	0.54
1201	STB	04/07/2007	5.2	0.26	0.23	0.42	0.49
260	ATN	07/05/1984	5.9	1.05	0.90	1.36	1.06
484	CESM	14/10/1997	5.6	1.77	0.97	1.80	1.19
788	ANT	06/04/2009	6.3	0.29	0.20	0.60	0.45
277	ATR	11/05/1984	5.5	0.41	0.39	1.00	0.77
616	PNS	01/04/2000	4.5	1.64	0.65	1.48	1.12
872	AQM	07/04/2009	5.6	0.30	0.23	0.66	0.65
415	NCM	03/10/1997	5.2	0.65	0.71	0.64	0.73
993	MTR	09/04/2009	5.3	0.04	0.04	0.12	0.15
831	ANT	06/04/2009	5.1	0.03	0.04	0.15	0.18
464	NCM	12/10/1997	5.2	0.10	0.12	0.33	0.32
574	CESM	03/04/1998	5.1	0.14	0.16	0.39	0.40
591	CAG	05/04/1998	4.8	0.33	0.24	0.74	0.68
836	FMG	06/04/2009	5.1	0.01	0.02	0.08	0.07
357	PCB	20/10/1996	4.4	0.45	0.27	1.05	0.79
70	SMG	07/09/1976	4.2	0.07	0.07	0.20	0.27