Seismic Response of High-rise Frame-shear Wall Buildings under the Influence of Dynamic Soil-Structure Interaction

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Abstract

Frame-shear wall buildings with multiple basements are the most commonly used structural form of high-rise buildings in the world today. In the traditional design method, structures are usually assumed as rigid base structures without considering soil-structure interaction (SSI), since incorporating the dynamic SSI tends to prolong natural periods and increase the damping of the system, which are considered beneficial for the seismic response of structures. However, recent studies exposed a potentially harmful aspect of SSI. In this study, a soil-foundation-structure model developed in finite element software and verified by shaking table tests is used to critically investigate the influence of SSI on high-rise frame-shear wall structures with a series of superstructure and substructure parameters. The beneficial and detrimental impacts of SSI are identified and discussed. Numerical simulation results indicate the rise in the stiffness of subsoil can dramatically amplify the base shear of structures. As the foundation rotation increases, inter-storey drifts are increased and base shears are reduced. In general, SSI amplifies the inter-storey drifts showing detrimental effects of SSI. However, as for the base shear, SSI exerts detrimental effects on most piled foundation cases as well as classical compensated foundation structures founded on C_e soil, whereas, for classical compensated foundation structures founded on soil types D_e and E_e, effects of SSI are beneficial since the base shear is reduced. Moreover, regarding structures with different foundation types, minimum base shear ratios considering the SSI reduction

effect are presented.

Keywords

Frame-shear wall buildings; Soil-structure interaction; Inter-storey drifts; Base shear; Finite element analysis; Skyscraper; Seismic response.

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1. Introduction

Coupled frame-shear wall structures combine the advantages of flexible layout and high ductility of frame structures and the advantages of large stiffness and high bearing capacity of shear wall structures (Gao et al. 2005; Saleh et al. 2018). In general, the frame-shear wall buildings can be designed as a dual lateral force resistance system to provide excellent abilities to resist wind loads and earthquake effects (Lu 2005; Son et al. 2017). On the other hand, the development of the economy and the rise in land prices in urban areas require taller buildings and more underground floors than several years ago (Far and Far 2019; Segaline et al. 2022). As a result, the frame-shear wall buildings with multiple basements are the most common structural form of high-rise buildings in the world today (Ayala et al. 2022).

Traditionally, the superstructure system and the basement-foundation system of high-rise buildings are designed separately (Far and Flint 2017). The superstructure is assumed to be rigidly supported at the base and the influence of soil-structure interaction (SSI) is ignored. But actually, when earthquake motions are acting, the response of the superstructure depends on the substructure system and vice versa. By reducing the structure to a classic single degree of freedom (SDOF) system, Wolf (1985) illustrated the fundamental period of the equivalent SDOF system considering SSI is always longer than its fixed-base counterparts. To put this another way, a significant impact of SSI is to decrease structural frequency. In addition, the effective damping of SDOF systems considering SSI is increased compared with fixed-base counterparts. Therefore, it is normally considered that SSI could improve the seismic performance of structures (Veletsos and Meek 1974).

In fact, mechanisms and principles of SSI's influence on the real soil-foundation-structure system are very complicated than the simplified SDOF system since this process involves soil amplification effect, kinematic interaction and inertial interaction. The following is a detailed interpretation of these three mechanisms encompassed by the theory of SSI:

- Firstly, even before the structures are constructed, the free-field motion is different from the bedrock motion, and this modification is called the soil amplification effect. Normally, the motion is amplified based on its frequency content. However, technically the amplification effect is not a part of actual SSI (Anand and Satish Kumar 2018).
- Secondly, excavating and inserting the ideally massless and relatively rigid foundation into the soil further modifies the foundation input motion. This is because the relatively rigid foundation filters out waves with a wavelength shorter than the size of the foundation (Hradilek and Luco 1970), thereby decreasing the energy of the input motion.
- Finally, the inertial forces generated by the superstructure in earthquake events can induce additional deformations in the soil and internal forces at the top of the foundation, which leads to further modification of foundation input motion (Mittal and Samanta 2021). This phenomenon, referred to as inertial interaction, is significant for heavy structures.

Based on the above three SSI principles, a three-step solution to quantify SSI effects, the substructure method, which considers these principles separately, has been developed (Kausel 2010). Compared with the direct method, the substructure method is more computationally efficient and cheaper. As a result, this method can be very suitable and convenient for parametric research (Anand and Satish Kumar 2018). However, it seems likely that taking into account the soil nonlinearity is not easily achievable by this method (Scarfone et al. 2020) because it is based on the superposition principle (Wolf 1998). As a matter of fact, nonlinearities in SSI include a number of aspects: geometric and material nonlinearities in different parts of the model which may result in material yielding, soil liquefaction as well as separation and slippage between foundation elements and surrounding soil. In addition, it is worth pointing out that the stiffness of soil will probably degrade under cyclic loading. It is usually considered by using cyclic strain-dependent shear modulus curves provided by researches such as Kramer (1996) and Darendeli (2001).

In contrast, the soil and superstructure can be analysed in a single step in the direct method. It has been deemed as the most accurate method for SSI simulations because the assumption of superposition is not required and it can better simulate the complexity of dynamic SSI (Borja et al. 1994; Far 2019). Nevertheless, the direct approach will probably require a lot of computational effort, including not only a suitable soil constitutive description, but also an appropriate simulation of the foundation elements, and the contact behaviour between the foundation and subsoil. Fortunately, with the advent of powerful computers, it became possible to analyse large soil-structure models with complex natures, such as irregularly shaped superstructures, embedded or pile foundations, and inhomogeneous and inelasticity of the subsoil.

The parametric study of the structure-soil model with different structural systems and substructure types has always been a hot spot in SSI-related research (Anand and Satish Kumar 2018). Over the past ten years, researchers have used two methods mentioned above to study the impacts of SSI on soil-structure models with different parameters. However, previous studies tended to focus on mid-rise and tall frames buildings (Galal and Naimi 2008; El Ganainy and El Naggar 2009; Fatahi et al. 2011; Tabatabaiefar et al. 2012, 2013; Tabatabaiefar and Fatahi 2014; Tabatabaiefar 2016; Tabatabaiefar and Clifton 2016; Ghandil and Behnamfar 2017; Bagheri et al. 2018; Yang et al. 2020; Radkia et al. 2020; Akbari et al. 2021; Shabani et al. 2021; Kamal et al. 2022; Zhang et al. 2022) or mid-rise frame-shear wall structures (Carbonari et al. 2011, 2012; Choinière et al. 2019; Qaftan et al. 2020; Al Agha et al. 2021), and there are few studies investigated SSI effects on high-rise frame-shear wall structures (Scarfone et al. 2020). It should be pointed out that the seismic behaviour of low-rise and high-rise buildings is different. In the same manner, the seismic behaviour of frame structures and frame-shear wall structures is also different because foundation rocking has an important effect on the latter (Sharma et al. 2018).

Besides, based on the conclusions of previous research, it can be found that due to the different methods, models and parameters adopted, there are many contradictory opinions about the effects of SSI. On the one hand, numerous studies have shown the detrimental effects of SSI. For example, SSI may increase inter-storey drifts (Tabatabaiefar and Fatahi 2014; Ghandil and Behnamfar 2017), lateral displacements (El Ganainy and El Naggar 2009; Radkia et al. 2020), plastic hinge rotation (Shirzadi et al. 2020) and the base shear (Van Nguyen et al. 2017). On the other hand, many researchers have come to opposite conclusions, including the decrease in deformation (Yang et al. 2020) and shear forces (Galal and Naimi 2008; Bagheri et al. 2018; Ansari et al. 2021). Because of the complexity of SSI and the lack of consensus among researchers in terms of the effect of SSI on the seismic performance of structures, very few structure design codes provide guidelines related to SSI. As a result, considering SSI in high-rise frame-shear wall building design practice has been a rarity (Anand and Satish Kumar 2018).

Based on the above discussion, it is necessary to critically investigate the seismic performance of high-rise frame-shear wall buildings with various parameters considering SSI. The question of how to identify the beneficial and harmful scenarios in SSI analysis also remains to be solved. Moreover, it is necessary to develop a simple but fairly accurate design program to take into account SSI in the structural design practice. In order to solve the above problems, a numerical soil-foundation-structure model is developed adopting the direct method to investigate the seismic responses of high-rise frame-shear wall buildings. Firstly, by comparing the results of numerical model and shaking table tests, the numerical simulation technique developed in this study is proved to be a rational and appropriate tool for SSI analyses. After that, the influence of superstructure and substructure parameters, such as height-width ratios (HWR), foundation types and soil types are discussed. The beneficial and detrimental effects of SSI on high-rise frame-shear wall structures are identified and summarised. Finally, the minimum base shear reduction factors regarding different foundation types are proposed.

2. The Parameters of the Structure and the Subsoil

Frame-shear wall buildings with three different structural heights: 60m (20-storey), 90m (30-storey) and 120m (40-storey) are considered in this study. For each structural height, three HWRs of 4, 5, 6 are considered. The arrangement of shear walls and columns is shown in Fig. 1 (a). According to AS 1170.4 2007, three different soil types: C_e, D_e, and E_e soil are adopted and their geotechnical properties are presented in Table 1 (Tabatabaiefar and Fatahi

2014). Additionally, end bearing piled foundation and classical compensated foundation (Fig. 1 b and c), are taken into account. The arrangement and dimensions of piles for buildings of different heights and HWRs have shown in Fig. 2 and Table 2. For each model, two far-field and two near-field earthquake motions (Fig. 3) are applied. The characteristics of the earthquake motions are summarised in Table 3. As a result, 36 rigid-base structures excluding SSI effects and 216 flexible-base structures including SSI effects are calculated in this study.

The structural sections are designed using SAP2000 V 20 software according to AS3600 (2018) and AS1170.4 (2007). The specified compressive strength, modulus of elasticity and unit weight of concrete are 40MPa, 32.8 GPa and 24.5kN/m^3, respectively. Nonlinear time-history analyses were then conducted under earthquakes records in Fig. 3 to ensure the maximum inter-storey drifts of all rigid-base models are less than 1.5%. The dimensions of shear walls, columns, beams and slabs of structures with different heights are shown in Table 4. In order to compare the results conveniently, the same component dimensions are adopted for the frame-shear wall structures with different HWRs.

3. Numerical Simulation Procedure

3.1 Numerical model of the superstructure

Abaqus 6.14 software (Dassault Systèmes SIMULIA 2012) is adopted to model the soil-structure system. In order to reduce the calculation time of a single model under the premise of ensuring accuracy, 4-node general-purpose shell elements, with reduced integration and hourglass control, are selected to simulate slabs and shear walls. Three dimensional 2-node linear beam elements are selected to simulate beams and columns. The basement, piles and soil domain are modelled by 8-node solid elements, with reduced integration and hourglass control. Fig. 1 (d) presents the mesh of the finite element model. Moreover, elastic-perfectly plastic behaviour is adopted in superstructure elements. Because the superstructures in this study are all reinforced concrete structures, the damping ratio is taken as 5%.

3.2 Numerical model of the subsoil

Rayhani and Naggar (2008) suggests the horizontal dimension of the soil domain should be at least five times the width of the superstructure. Because the superstructure height and HWR adopted in this study are variable, the width of the superstructure is also variable (from 10m to 30m). Therefore, the horizontal dimensions of the soil domain vary between 50m and 150m (five times the width of the superstructure). Moreover, as shown in Fig. 1, the bedrock depth is assumed to be 30 m since the most amplification effects occur within the top 30 m of the subsoil (Rayhani and Naggar 2008).

When meshing the ground soil, the guideline proposed by Gazetas (1983) is employed. The height of the soil element should be (1/5~1/8) V_s/f_{max}, where f_{max} is the highest wave frequency considered. In this study, seismic records are filtered to prevent frequencies higher than 25 Hz so as to limit the dimension of soil elements without affecting the accuracy of results.

To consider the nonlinearity of ground soil, the Mohr-Coulomb failure criterion is adopted and the shear modulus reduction curves (Fig. 4) and damping ratio curves (Fig. 5) developed by Sun et al. (1998) and Seed et al. (1986) are used in this study. The strain-compatible values of soil damping and shear modulus are iteratively determined under the action of different earthquakes. The detailed steps of this process are introduced in Fatahi and Tabatabaiefar (2014) and Van Nguyen et al. (2017). Rayleigh damping is adopted to take into account the energy loss in the subsoil in an earthquake event. When calculating damping coefficients, two soil frequencies covering the range with a significant amount of input motion are adopted (Park and Hashash 2004).

3.3 Boundary conditions

In this study, viscous-spring boundary was applied on lateral and bottom surfaces of the subsoil domain to avoid the reflection of outward propagating waves, independent springs and dampers in three directions are specified on the boundary nodes (Fig. 6). The coefficients of the springs (K_T and K_N) and dampers (C_T and C_N) can be obtained as follows (Gu et al. 2007):

$$K_T=\alpha_\mathrm{T}\, G/R,\ C_T=\rho\ V_s \tag{1}$$

$$K_N=\alpha_\mathrm{N}\, G/R,\ C_N=\rho\ V_p \tag{2}$$

Where α_T, α_N are modified coefficients, and their values are suggested by Liu et al. (2006). Subscripts T and N indicate tangential and normal directions. R is the distance between the wave source and boundary nodes; ρ, G and V_p are the density, shear modulus and P wave velocity of the ground soil, respectively.

3.4 Input of earthquake motions

During dynamic time-history analyses, the motion of boundary nodes is supposed to conform to the free-field motion to supply conditions identical to the infinite model. To achieve this goal, the equivalent node force method is adopted. In this method, first of all, the free-field strain of the boundary is obtained from the geometric equation, and then the stress on the boundary is obtained by stress-strain relationship. After that, the boundary node balance relationship is used to calculate the equivalent earthquake load on the boundary node, that is, the equivalent node force ($\boldsymbol{F_b}$). Next $\boldsymbol{F_b}$ is applied on boundary nodes of the soil domain in the form of concentrated forces to realize the seismic wave input.

$\boldsymbol{F_b}$ consists of three terms: the first two terms are employed to compensate for the impacts of springs and dashpots on the boundary nodes, and the third term is the free field stress on the boundary (Ma et al. 2020):

$$\boldsymbol{F_b} = (\boldsymbol{K_b u_b^{ff}} + \boldsymbol{C_b v_b^{ff}} + \boldsymbol{\sigma_b^{ff}}\,\boldsymbol{n})A_b \tag{3}$$

Where $\boldsymbol{u_b^{ff}}$ and $\boldsymbol{v_b^{ff}}$ are free-field displacement vectors and velocity vectors; $\boldsymbol{K_b}$ and $\boldsymbol{C_b}$ are springs and dashpots coefficient vectors. A_b is the influencing area of boundary nodes and $\boldsymbol{n}$ is the cosine vector of the normal direction outside the boundary. $\boldsymbol{\sigma_b^{ff}}$ is the free-field stress tensor which can be derived from the geometric equation and linear elastic material stress-strain relationship.

According to the seismic motion input approach described above, a MATLAB program was compiled by the authors to calculate the amplitudes of $\boldsymbol{F_b}$, and then $\boldsymbol{F_b}$ was applied in three directions of each boundary node.

4. Experimental Shaking Table Test Program

To check the accuracy of the adopted numerical modelling technique, shaking table tests were carried out on a 15-storey frame structure, and then the results of numerical simulation are compared with the experimental results.

The natural frequency and mass of the prototype structure are 0.384 Hz and 953 tonnes respectively, and the structure is assumed to be built on clayey soil with the V_s of 200 m/s and mass density of 1470 kg/m^3. In the scaling process, the objective is to achieve “dynamic similarity", in which same or similar accelerations and density of shaking table test model and prototype are desired (Meymand 1998). After determining the scaling condition of the acceleration and density, the scaling relations for other variables, such as mass, time, length, etc., can also be derived and expressed in terms of geometric scaling factor (λ). By comparing specifications of the shaking table and the characteristics of models with different λ, λ of 1:30 provides the largest achievable model with maximum dimensions, payload, and overturning moment which meet the limitations of the shaking table. Thus, λ=1:30 is adopted for experimental shaking table tests in this study. Therefore, the height, length, and width of the scaled structural model in shaking table tests can be easily calculated to achieve geometric similarity. Moreover, the natural frequency and mass of the scaled model are determined to be 2.11 Hz and 106 kg respectively to achieve the dynamic similarity.

The scaled model was then designed and assembled, and steel plates were used for the manufacturing of columns (500×40×2 mm), slabs (400×400×5 mm), and the foundation (500×500×10 mm), as shown in Fig. 7. The mass of the assembled structure is 104 kg and the natural frequency is 2.19Hz, which are very close to the calculated value above. Similarly, the V_s and mass density of soil in shaking table tests can be determined to be 36 m/s and 1470 kg/m^3. The scaled soil-structure model is shown in Fig. 8. Moreover, for seismic records shown in Fig. 3, it is required to reduce the time steps of original records by a factor of $\lambda^{-1/2}$ = 5.48. More details about the shaking table tests can be found in Tabatabaiefar et al. (2014a), Fatahi et al. (2015), Tabatabaiefar and Mansoury (2016) and Tabatabaiefar (2016).

The identical fixed-base and flexible-base frame structure models (Fig. 9 and 10) were also established in *Abaqus* software using the modelling technique introduced in Section 3.

After that, numerical time-history analyses and shaking table tests were conducted under the action of four scaled seismic records. The results in terms of the maximum lateral deflection of frame structure obtained from these two approaches are compared in Fig. 11 and 12.

As shown in Fig. 11, by comparing the values and trends of numerical calculations and experimental results, it can be found that the numerical model proposed in Section 3 is accurate enough to capture the seismic behaviour of buildings. Fig. 12 indicates that errors of average maximum lateral deflections of fixed-base and flexible-base models are only 8.8% and 5.6%, respectively. Therefore, the numerical simulation technique developed in this study is a rational and appropriate tool for SSI analyses.

5. Results and Discussions

Fig. 13, 14, 15, 16, 17, 18, 19, 20 and 21 recorded the storey lateral deflections of frame-shear wall structures with different structural heights, HWRs, foundation types and soil types when the maximum lateral deflection of the top storey ($\varDelta$) occurs. It can be seen that almost all the $\varDelta$ of flexible-base structures is magnified in comparison with rigid-base structures. On average, compared with rigid-base structures, the $\varDelta$ of piled foundation structures and classical compensated foundation structures has increased by 90.5% and 129.2%, respectively. This is because fundamental periods of the soil-structure system are elongated, and the displacement response spectrum curve basically increases with the increase of the fundamental period of the structure.

Moreover, the $\varDelta$ values of piled foundation models do not increase or decrease dramatically with the variations in the soil types. By contrast, the variations of $\varDelta$ for classical compensated foundation models are remarkable, especially under far-field earthquakes. It can also be found that under near-field earthquakes, the Δ of piled foundation models is not significantly reduced compared to the classical compensation foundation model, and even increases in some cases. Therefore, the presence of the piled foundation does not guarantee reduction in Δ for structures. This is because the piled foundation can absorb a larger amount of inertial energy while reducing the foundation rotation (Van Nguyen et al. 2017).

To analyse and compare the beneficial and detrimental effects of SSI on high-rise frame shear wall buildings more clearly and comprehensively, two commonly used parameters calculated from numerical soil-structure models, base shears (V_{fle}) and maximum inter-storey drifts (δ_{fle}), are normalised by those obtained from conventional fixed-base models (V_{fix} and δ_{fix}). Besides, values of base shear ratio (V_{fle}/V_{fix}) and inter-storey drifts ratio ($\delta_{fle}/\delta_{fix}$) under the action of four seismic records (Fig. 3) are averaged in this study so as to clearly demonstrate the impacts of different parameters. Therefore, if the value of V_{fle}/V_{fix} or $\delta_{fle}/\delta_{fix}$ is greater than 1, it means that the SSI amplifies the base shear or maximum inter-storey drifts and thus, its effect is detrimental. Firstly, the value of V_{fle}/V_{fix} of classical compensated foundation structure and piled foundation structure with different superstructure and substructure parameters are shown in Fig. 22 and 23.

The values of V_{fle}/V_{fix} increase significantly with the increase of the stiffness of soil. For piled foundation structures, the V_{fle}/V_{fix} of models resting on D_e soil and C_e soil increased by 29.0% and 89.8% respectively in comparison with E_e soil supported model. For classical compensation foundation structures, these two values are 106.3% and 366.4%, respectively. The variation of HWR can also slightly change this ratio, but its influence is far less than that of the soil type. For the classical compensated foundation structures, the values of V_{fle}/V_{fix} of structures with the C_e soil are always greater than 1, indicating that the stiff soil can increase the base shear of frame-shear wall structures when SSI is considered. In contrast, V_{fle}/V_{fix} of structures built on D_e and E_e soil types are both less than 1, indicating that base shear can be reduced when structures are built on medium or soft soils.

When it comes to piled foundation buildings, more seismic energy can be absorbed during an earthquake event. As a result, the values of V_{fle}/V_{fix} are greater than 1 in almost all piled foundation cases, which suggests that the seismic demand of high-rise buildings founded on the piled foundation is amplified after considering SSI.

The values of $\delta_{fle}/\delta_{fix}$ are shown in Fig. 24 and 25. It can be found that SSI always increases the maximum inter-storey drifts because $\delta_{fle}/\delta_{fix}$ is greater than 1 in almost all cases. Therefore, SSI may alter the performance level of high-rise buildings. In addition, when superstructure and substructure parameters are changed, the influences of SSI on the values of $\delta_{fle}/\delta_{fix}$ shows different trends. This is because the HWR and substructure stiffness have complex effects on the deformation of the building. On the one hand, as mentioned above, stiffer ground soil and wider structure can limit the foundation rocking, and thereby reduce the deformation of the superstructure; on the other hand, the increased structural weight and stiffness of the substructure system can attract more seismic energy to deform the superstructure. Thus, when studying SSI effects on the deformation of the superstructure, the base shear, inter-storey drift and foundation rocking should be considered comprehensively.

According to Wolf (1985) and Kramer (1996), Δ consists of rocking (Δ_θ) and distortion components (Δ_d). In this study, proportions of lateral deflection caused by foundation rocking (Δ_θ/Δ) are adopted to reflect the significance of the foundation rocking under seismic events.

In order to obtain this value under different parameters, firstly, the moment when the Δ occurred is recorded (Hokmabadi et al. 2012). After that, the Δ_θ is calculated by multiplying the height of the structure by the foundation rocking angle at this moment. Finally, the ratio Δ_θ/Δ can be calculated.

Fig. 26 illustrates the relationship between $\delta_{fle}/\delta_{fix}$, V_{fle}/V_{fix} and Δ_θ/Δ of models with different heights and substructure parameters. When the subsoil is stiff enough and the values of Δ_θ/Δ are small (less than 0.5 for classical compensated foundation structures and less than 0.15 for piled foundation structures), the data points are basically distributed around the $y=x$ line. It indicates that the amplification coefficient for the base shear is almost equal to the amplification coefficient for the inter-storey drifts after considering SSI. Nevertheless, as soil stiffness decreases and the values of Δ_θ/Δ becomes larger, the data points begin to deviate from the $y=x$ line and shift downward to the right, indicating that the increase of the value of Δ_θ/Δ tends to amplify the inter-storey drift and reduce the base shear of high-rise shear wall structures.

It is also worth highlighting that the lateral displacement of the classical compensated foundation model is dominated by Δ_θ, while the proportion of Δ_θ in piled foundation model is relatively small (less than 50%). This is not surprising as the end-bearing piles can effectively restrict the foundation rotation. Moreover, the difference in the SSI effect on seismic responses of structures with different foundation types can be clearly seen in Fig. 26. Almost all data points of the pile foundation model lie in the range of $\delta_{fle}/\delta_{fix}>1$, $V_{fle}/V_{fix}>1$, indicating that no matter how the model parameters change, the SSI has detrimental effects; however, for the classical compensated foundation model, structures with D_e and E_e soil types are below the V_{fle}/V_{fix}=1 line, indicating that SSI has a beneficial effect, it can reduce the base shear of the superstructure, even if the inter-storey drifts are still amplified.

From the above analysis, it has become apparent that SSI has detrimental effects on inter-storey drifts of high-rise frame-shear wall buildings, and its effect on base shears is determined by foundation type and subsoil stiffness. However, as mentioned in the introduction section, previous studies are mainly focusing on the amplification of inter-storey drifts and neglecting the effect of SSI on the shear force, because most of the previous papers concluded SSI can reduce the shear force of the superstructure. For example, Tabatabaiefar et al. (2014b) developed an empirical relationship to predict the value of $\delta_{fle}/\delta_{fix}$ and assess the performance level of buildings. Similarly, this study will summarise the influence of SSI on the base shear of high-rise frame-shear wall buildings and develop a simple and accurate procedure to estimate the value of V_{fle}/V_{fix}.

In *Abaqus*, the modal analysis can be conducted and natural frequency of models can be extracted. The fundamental periods of fixed-base (T_{fix}) and flexible-base model (T_{fle}) are summarised in Table 5, 6 and 7. Fig. 27 shows the relationship between T_{fle}/T_{fix} and V_{fle}/V_{fix}. With the substructure stiffness decreasing and the T_{fle}/T_{fix} increasing, V_{fle}/V_{fix} decreases linearly. But after reaching a certain level, the value of V_{fle}/V_{fix} remains stable and no longer decreases. For the classical compensated foundation structure, the value of V_{fle}/V_{fix} does not decrease after reaching 0.318, while for the piled foundation structure, this value is 0.931. Therefore, current seismic standards may determine whether shear forces should be reduced and specify different minimum values of reduced shear force depending on the type of the foundation.

Moreover, straight line fittings were also performed on the descending sections in Fig. 27. The fitting results of the classical compensated foundation structure and piled foundation structure are as follows, respectively:

$$V_{fle}/V_{fix}=10.356-8.200\ (T_{fle}/T_{fix}) \tag{4}$$

$$V_{fle}/V_{fix}=9.855-7.429\ (T_{fle}/T_{fix}) \tag{5}$$

The linear correlation coefficients r = -0.8731 and -0.8380 respectively. It can be inferred that when the substructure stiffness of soil-structure system is very high ($T_{fle} \approx T_{fix}$), the V_{fle} has been enlarged to more than double the V_{fix}. The slopes in Eqs. 4 and 5 are negative, indicating that with the increase of the SSI effect and the elongation of natural period of the soil-structure system, the value of V_{fle}/V_{fix} shows a downward trend until the T_{fle}/T_{fix} reaches a specific value.

Therefore, in the structural design process, designers can easily obtain the value of V_{fle} for high-rise buildings by calculating the V_{fix} and T_{fle}/T_{fix}, without carrying out time-consuming numerical calculations. In addition, many previous studies have proposed empirical relationships to calculate T_{fle}, which can be found in Balkaya et al. (2012) and Renzi et al. (2013).

Conclusion:

To study the seismic behaviour of the high-rise frame-shear wall buildings considering SSI, numerical simulation of soil-structure systems with different parameters have been conducted in this study using finite element software. The results of fixed- and flexible-base models, such as maximum lateral deflections, foundation rotation, inter-storey drift ratio and base shear ratio are obtained and compared to identify the beneficial and detrimental effects of SSI.

The following conclusions can be drawn:

- After considering SSI, the Δ of almost all models are amplified. The influence of soil types on the deformation of the piled foundation supported structures are not significant. In addition, it is not guaranteed that the deformation of the piled foundation structures is smaller than compensated foundation structures.
- Regardless of the foundation type, the increase of the V_s of subsoil can significantly increase the value of V_{fle}/V_{fix} of structures. In contrast, HWR has little effect on this value.
- The influence of substructure parameters on the value of $\delta_{fle}/\delta_{fix}$ is very complex. In general, the increase of Δ_θ/Δ can amplify the inter-storey drifts and reduce the base shear of high-rise buildings.
- SSI amplifies the value of $\delta_{fle}/\delta_{fix}$ of almost all the cases studied in this paper. Therefore, the effect of SSI is detrimental to the inter-storey drifts of high-rise frame-shear wall buildings.
- SSI amplifies the value of V_{fle}/V_{fix} of piled foundation structures and C_e soil supported classical compensated foundation structures. In terms of classical compensated foundation structures with D_e and E_e soil types, the effects of SSI are beneficial because the value of V_{fle}/V_{fix} is reduced.
- With the increase of the T_{fle}/T_{fix}, the value of V_{fle}/V_{fix} decreases linearly until it reaches 0.318 (classical compensated foundation structures) or 0.931 (piled foundation structures). After that, the value of V_{fle}/V_{fix} remains constant. Therefore, current seismic standards may determine whether shear forces should be reduced and specify different

minimum values of reduced shear force according to the type of foundation.

Current research is focusing on the SSI effects on high-rise frame-shear wall structures. One of the recommendation for future research is to investigate the SSI effects on super high-rise buildings with the tube in tube structure, which is a commonly used structural system for super high-rise buildings. In addition, more complex and practical situations, such as irregular buildings built on layered ground soils, can also be the focus of future study. Moreover, dynamic interaction between soil and structure group is of great importance to the structural design in urban areas.

Data Availability Statement

All data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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Table 1. Geotechnical characteristics of the subsoil

Soil type (AS1170)	Unified classification (USCS)	Shear wave velocity (V_s) (m/s)	G_{max} (kPa)	Soil density (kg/m^3)	Poisson's ratio	c' (kPa)	φ' (degree)	Plastic Index
C_e	GM	600	623,400	1730	0.28	5	40	-
D_e	CL	320	177,300	1730	0.39	20	19	20
E_e	CL	150	33,100	1470	0.40	20	12	15

Table 2. Configuration of piles of structures with different heights and HWRs

Structures	HWRs	Diameter (m)	Centre to centre distance (m)
	4	1.2	4
20-storey	5	1.2	3
	6	1.2	2.6
	4	1.5	6
30-storey	5	1.5	5
	6	1.5	4
	4	2	8
40-storey	5	2	6
	6	2	5

Table 3. Earthquake ground motions adopted in this study

Earthquake	Country	Year	PGA (g)	Mw (R)	T (s) Duration	Type	Hypocentral distance (km)	Record type
El Centro	USA	1940	0.349	6.9	56.5	Far field	15.69	Bedrock record
Hachinohe	Japan	1968	0.229	7.5	36.0	Far field	14.1	Bedrock record
Kobe	Japan	1995	0.833	6.8	50.0	Near field	7.4	Bedrock record
Northridge	USA	1994	0.843	6.7	30.0	Near field	9.2	Bedrock record

Table 4. Dimensions for sections of studied models (m)

Structures	Storey level	Shear walls	Columns	Beams	Slabs
20-storey	1~5	0.55	0.55×0.55	0.40×0.40	0.25
	6~10	0.50	0.50×0.50	0.40×0.40	0.25
	11~15	0.45	0.45×0.45	0.40×0.40	0.25
	16~20	0.40	0.40×0.40	0.40×0.40	0.25
30-storey	1~10	0.70	0.70×0.70	0.50×0.50	0.25
	11~20	0.60	0.60×0.60	0.50×0.50	0.25
	21~30	0.50	0.50×0.50	0.50×0.50	0.25
40-storey	1~10	0.80	1.00×1.00	0.50×0.80	0.25
	11~20	0.70	0.90×0.90	0.50×0.80	0.25
	21~30	0.60	0.80×0.80	0.50×0.80	0.25
	31~40	0.50	0.70×0.70	0.50×0.80	0.25

Table 5. Fundamental periods of 20-storey fixed-base and flexible-base models (s)

HWR	Fixed-base model	Flexible-base model					
		Piled foundation			Classical compensated foundation		
		E_e soil	D_e soil	C_e soil	E_e soil	D_e soil	C_e soil
6	0.9991	1.4098	1.1801	1.1351	1.7434	1.2246	1.1359
5	1.0272	1.4083	1.2001	1.1376	1.6179	1.2142	1.1349
4	1.0047	1.3518	1.1825	1.1321	1.5835	1.2059	1.1205

Table 6. Fundamental periods of 30-storey fixed-base and flexible-base models (s)

HWR	Fixed-base model	Flexible-base model					
		Piled foundation			Classical compensated foundation		
		E_e soil	D_e soil	C_e soil	E_e soil	D_e soil	C_e soil
6	1.7517	2.3216	2.0167	1.9234	2.7370	2.0654	1.9310
5	1.7975	2.2640	2.0163	1.9346	2.7166	2.0679	1.9431
4	1.7682	2.2363	1.9894	1.9174	2.5931	2.0281	1.9234

Table 7. Fundamental periods of 40-storey fixed-base and flexible-base models (s)

HWR	Fixed-base model	Flexible-base model					
		Piled foundation			Classical compensated foundation		
		E_e soil	D_e soil	C_e soil	E_e soil	D_e soil	C_e soil
6	2.4500	3.2517	2.8729	2.6788	3.7733	2.8263	2.6405
5	2.4821	3.1948	2.8754	2.6919	3.7559	2.8670	2.7006
4	2.5123	3.2806	2.8531	2.6825	3.3748	2.8572	2.6850