

1 flexibility of the ground, the foundation of buildings incorporating the dynamic SSI
2 has two more degrees of freedom: horizontal movement and rotation. The increase of
3 degrees of freedom can reduce the stiffness of soil-structure system, and then lengthen
4 the natural period. Secondly, the foundation input motion is different from the free
5 field motion due to the kinematic interaction and inertial interaction effects (Anand
6 and Satish Kumar 2018). According to the response spectrum theory, the increase of
7 the natural period and the change of the input motion can modify the seismic demand
8 of the structure (Seed et al. 1976). In addition, when the flexible-base structure
9 (considering the SSI) is in vibration, a considerable part of the energy is dissipated
10 into the soil due to the radiation damping generated by wave propagation and the
11 hysteresis damping of soil material, which makes the effective damping ratio of the
12 structures resting on soft soils is generally greater than that of the rigidly supported
13 structure (Wolf 1985).

14 Because SSI lengthens the natural period and increases the damping of system,
15 Veletsos and Meek (1974) concluded that influence of SSI seemed to be beneficial to
16 the structural system under the act of earthquake records. Besides, provisions that
17 allow reduction of base shear when SSI is considered can be found in many current
18 codes (NZS1170.5, 2007; IBC, 2012; NBCC, 2010; GB 50011-2010). However, this
19 conclusion is misleading. Horizontal movement and rotation of the foundation may
20 amplify the lateral deformations and inter-storey drifts of the superstructure (Guin and
21 Banerjee 1998; Tabatabaiefar 2012; Tabatabaiefar & Clifton 2016; Far 2019a), which
22 are the most commonly used damage parameters in performance-based design
23 (Monavari and Massumi 2012; Mohammadi et al. 2015; Samimifar et al. 2019). As a
24 result, considering SSI can alter the performance level of buildings and is notably
25 crucial for high-rise and slender buildings (Kramer 1996; Fatahi et al. 2011; Samali et
26 al. 2011; Tabatabaiefar et al. 2012).

27 Therefore, it can be observed that there are some contradictory opinions about
28 the influence of SSI on the seismic responses of structures (Mylonakis and Gazetas
29 2000; Far & Flint 2017). The complexity of SSI and inconclusiveness of the literature
30 provide sufficient motivation to critically investigate the effects of SSI (Sharma et al.

1 2018).

2 Based on commonly used structure types in reality, it is essential to study the
3 impacts of SSI on various kinds of structural systems to get a complete understanding
4 of their seismic response. When subjected to seismic loading, the behaviour of any
5 structure-soil system results from different factors, such as the input motion, type and
6 depth of foundation, soil modulus and shear wave velocity, geometry and natural
7 period of the superstructure (Anand and Satish Kumar 2018; Saleh et al 2018). In the
8 past decades, many scholars have investigated the influence of SSI on moment
9 resisting frames with low and mid-rise buildings (Tabatabaiefar et al. 2013;
10 Tabatabaiefar and Fatahi 2014; Ghandil and Behnamfar 2017; Far 2019b; Yang et al.
11 2020). Generally, compared with rigidly supported model, the base shear of the model
12 considering SSI decreases whereas the lateral deformations and inter-storey drifts
13 increase relatively, and the SSI effects is more significant with diminishing the shear
14 wave velocity of ground soils. Therefore, ignoring SSI may lead to changes in the
15 performance level of buildings and unsafe results of structures subjected to seismic
16 loading.

17 Studies in terms of high-rise buildings and frame-shear wall structures have been
18 substantially insufficient. Han (2002) examined a 20-storey building resting on pile
19 foundation. The influences of pile foundation displacements on the performance of
20 high-rise buildings were studied, and the results was compared with the response of
21 buildings founded on shallow foundation. The author found that the raft or mat
22 foundation usually induce longer natural period and much significant displacement
23 amplitudes in superstructures and foundations. The conventional design procedure
24 without the influence of SSI cannot fully represent the real behaviour of high rise
25 buildings. In the study of Galal and Naimi (2008), the influences of SSI on the
26 seismic response of 20-storey moment resisting frames were evaluated by comparing
27 the flexible-base model to the rigid-base model under the act of 13 different
28 earthquake records on four types of soils. It is concluded that the fixed-base
29 assumption cannot represent the true seismic behaviour. Besides, the author
30 recommended that designers and researchers should pay more attention to determine

1 whether considering SSI effects is beneficial or detrimental of a structure's seismic
2 response. In the research of Bilotta et al. (2015), seismic response of a 100 m
3 high-rise building in Naples (Italy) resting on a piled raft floating foundation was
4 evaluated. In this model, the site amplification effects and SSI were adequately
5 considered. The results indicated that the peat layer acted as a natural damper in the
6 system and thus, it is beneficial in the free-field seismic behaviour. In addition, the
7 influences of inertial interaction were assessed considering soil-foundation
8 compliance and it leads to an increase of the natural period of the system. The
9 prolonged natural period results in further decrease of spectral acceleration. Bagheri et
10 al. (2018) carried out numerical simulations on 15-storey and 30-storey frame
11 structures rested on six types of pile foundations to study the seismic behaviour of the
12 structures. The numerical results indicated that the performance level of the structures
13 is related to the pile configuration, ground motion features and height-width ratios of
14 the superstructures. Moreover, SSI was founded to have remarkable influences on the
15 distribution way of the shear forces along the superstructures. Scarfone et al. (2020)
16 investigated the impacts of SSI on a 20-storey frame-shear wall building using 3D
17 nonlinear dynamic time history analyses. Particularly, three foundation types (shallow
18 foundation, compensated foundation and pile foundation) were evaluated. Results
19 showed that the increase in foundation flexibility can result in a decrease of seismic
20 demand and a more obvious foundation rotation. Arboleda-Monsalve et al. (2020)
21 investigated the seismic performance of 40-storey buildings considering SSI. Four
22 scenarios of modeling and building configuration were considered: fixed-base model,
23 model considering SSI, fixed-base model with shear walls, and model including shear
24 walls and considering SSI. Taking into account SSI effects in the model altered the
25 seismic demands of the high-rise buildings in regard to the inter-story drifts, storey
26 horizontal accelerations, and seismic-induced settlements. Direct economic losses for
27 the strong earthquakes increased 33% considering SSI effects compared with the
28 fixed-base model. In the research of Qaftan et al. (2020), the scaling process and
29 design of a scaled 15-storey RC frame-shear wall structure with a scale factor of 1:50
30 are addressed. The results indicated the structure built on pile raft foundation

1 experienced on average 30% less rocking in comparison to the structure supported by
2 the raft foundation.

3 From these studies in terms of moment resisting frames and frame-shear wall
4 structures, it has been observed that the current research are mainly focusing on low
5 and mid-rise frame structures. However, the seismic response of low-rise and
6 high-rise structures are different. In the same way, the seismic response of frame
7 structures is different from that of the frame-shear wall structures in which foundation
8 rotation is significant (Sharma et al. 2018; Walsh 2018). Therefore, it is imperative to
9 investigate the behaviour of high-rise wall-frame buildings with different
10 superstructure geometry considering SSI.

11 The aim of this study is to assess the seismic performance of a generally used
12 structural system of high-rise buildings: frame-core tube structure. Firstly, a new 3D
13 numerical soil-structure model is built and verified as a qualified model that can be
14 used for further SSI numerical simulations. After that, seismic behaviour of high-rise
15 frame-core tube structures founded on soft soil deposit with different heights
16 (20-storey, 30-storey, and 40-storey) is investigated.

18 **2 Development and validation of 3D numerical model**

19 In this section, a novel and enhanced soil-structure numerical model is
20 established in finite element software ABAQUS 6.14, and the direct method (Wolf
21 1998) is used to accurately simulate the previous shaking table tests of a frame
22 structure in the authors' research group. The accuracy of the 3D numerical model is
23 validated by comparing the results of numerical simulation and experimental shaking
24 table tests. In the next section, this validated 3D numerical model will be employed to
25 simulate the high-rise frame-core tube structure.

27 **2.1 Structural model**

28 The prototype of structure in shaking table tests is a 15-story frame structure
29 founded on shallow foundation with a natural frequency of 0.384 Hz and a total mass
30 of 953 tonnes. The aim of the scaling procedure for this test was to achieve “dynamic

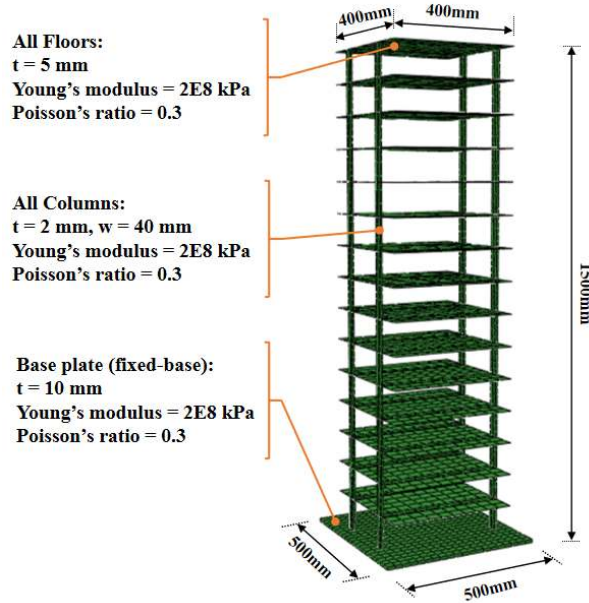
1 similarity”, where model and prototype experience homologous forces. Therefore, the
2 methodology of Meymand (1998) was adopted. The first condition is that the shaking
3 table test is carried out in a 1-g environment, which defines model and prototype
4 accelerations to be equal. Secondly, a model with similar density to the prototype is
5 desired. By defining scaling conditions for density and acceleration, a complete set of
6 dimensionally correct scaling relations can be derived for all variables being studied.
7 By comparing shaking table specifications and dimension and weight of scaled model
8 considering different scaling factors, geometric scaling factor (λ) of 1:30 provides the
9 largest achievable scale model with rational scales, maximum payload, and
10 overturning moment which meet the facility limitations. Thus, scaling factor of 1:30 is
11 adopted for experimental shaking table tests in this study.

12 The scaled frame structure is made of steel, with a height of 1.5 meters and width
13 of 0.4 meters. According to Meymand (1998), in order to achieve dynamic similarity,
14 in addition to geometric dimensions, the natural frequency of the prototype should be
15 scaled by $\lambda^{-1/2}$ (5.480). Therefore, the required natural frequency of the scaled model
16 is 2.11Hz. The scaled model was then designed and assembled. According to the Sin
17 Sweep test, the natural vibration frequency of the constructed structural model is
18 2.19Hz. More details about the shaking table tests can be found in Tabatabaiefar et al.
19 (2014), Fatahi et al. (2015), Tabatabaiefar (2016) and Tabatabaiefar and Mansoury
20 (2016).

21 The dimension, physical and mechanical properties of the numerical frame
22 structure model and ground soil as well as four scaled earthquake records are the same
23 as those selected in the experimental shaking table tests. Shell elements in ABAQUS
24 were selected to simulate the steel columns and slabs of the frame structure and solid
25 elements were selected to simulate the foundation. The natural frequency of the
26 numerical model is 2.21Hz, which is very close to the natural frequency of the
27 shaking table test model (2.19Hz). Under the action of four scaled earthquake records,
28 all structural elements adopt elastic-perfectly plastic behaviour.

29 According to the shaking table tests, the damping ratio of the scaled frame
30 structure is about 1.1%. Therefore, based on the structure’s first and second vibration

1 frequencies, two damping coefficients α and β can be obtained as 2.297 and 0.0004.
2 Fig. 1 illustrates the 3D numerical model of this scaled frame structure.



3

4

Fig. 1 3D numerical model of this scaled fixed-base frame structure in ABAQUS

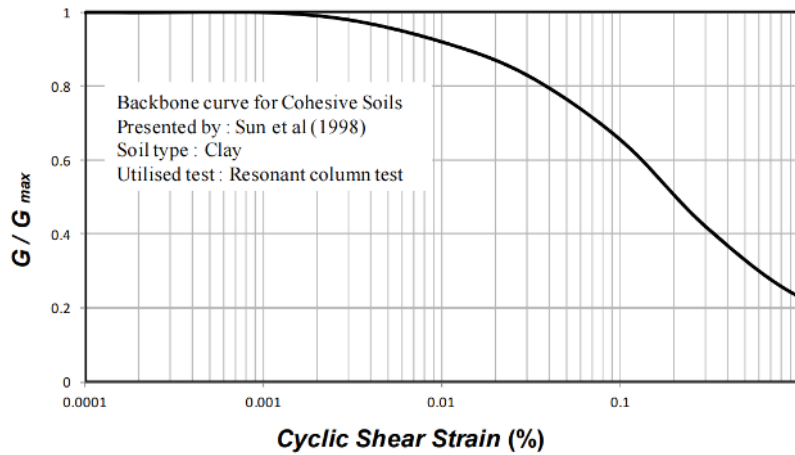
5

6 2.2 Soil model

7 The soil element is simulated by 3D eight-node reduced integration element
8 C3D8R and the Mohr-Coulomb model is adopted. In ABAQUS, cohesion and of
9 internal friction angle (the same values as used in the shaking table tests) and the
10 tension cut off option are specified.

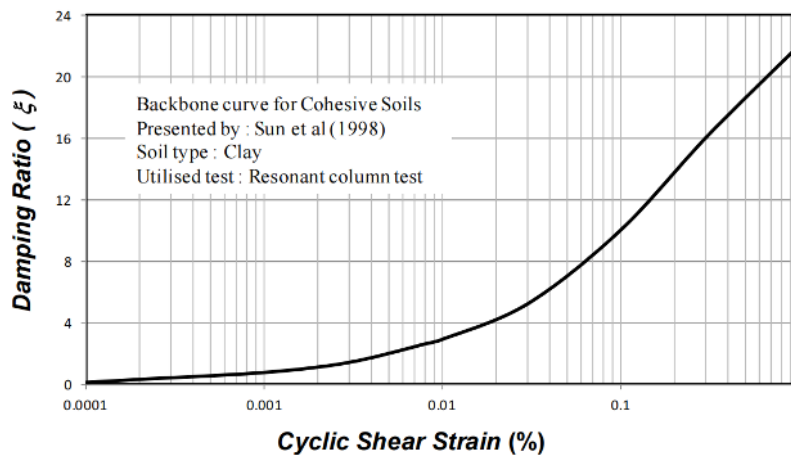
11 When calculating wave propagation or seismic response in soil and rock medium,
12 the fully nonlinear method is a common method to capture the soil nonlinearity. In
13 this method, the nonlinear backbone curves and trial and error were used to obtain the
14 strain-compatible values of soil damping and shear modulus. The detailed steps of this
15 method can be found in Tabatabaiefar et al. (2013), Fatahi and Tabatabaiefar (2014)
16 and Tabatabaiefar et al (2017).

17 Sun et al. (1998) investigated cyclic shear strain (γ_c) depended shear modulus
18 and damping ratio (ξ) of cohesive soils and provided backbone curves recommended
19 for seismic site-response assessments. These curves, including relations between
20 shear modulus ratio (G/G_{max}) versus γ_c (Fig. 2) and ξ versus γ_c (Fig. 3) for cohesive
21 soils, are adopted in this research.



1
2

Fig. 2 Relations between G/G_{max} versus γ_c for cohesive soils (after Sun et al., 1998)



3
4

Fig. 3 Relations between ξ versus γ_c for cohesive soils (after Sun et al., 1998)

5 According to the fully nonlinear method and backbone curve introduced
6 previously, the soil strain-compatible parameters under different seismic excitation
7 can be calculated (Table 1). The shear strain amplitude caused by each seismic record
8 is different, thus the final values of G/G_{max} and ξ change with the change of seismic
9 records.

10 Rayleigh damping is employed to capture the energy losses in the soil medium
11 during seismic events. The calculation of damping coefficients α and β depends on the
12 frequencies selected. Therefore, it is very crucial to select soil frequencies/modes
13 (Park and Hashash 2004). This study adopts the suggestion of Park and Hashash
14 (2004) that the selection of frequencies should partially cover the main frequency
15 range of the input motion. The calculated Rayleigh damping parameters under
16 different earthquakes are shown in Table 1.

17

1 Table 1 Adopted strain-compatible parameters and damping parameters for the soil-frame
 2 structure system

Earthquake records	G/G_{max}	ξ	Damping coefficients
Scaled El-Centro	0.48	14.0%	$\alpha=5.245$ $\beta=0.0028$
Scaled Hachinohe	0.36	17.1%	$\alpha=2.540$ $\beta=0.0071$
Scaled Kobe	0.33	17.5%	$\alpha=5.885$ $\beta=0.0039$
Scaled Northridge	0.25	19.8%	$\alpha=6.534$ $\beta=0.0044$

3

4 **2.3 Contact surface of foundation and soil**

5 Surface to surface contact is selected to simulate the interaction between the
 6 bottom surface of foundation (master surface) and the top surface of soil medium
 7 (slave surface) during seismic loading. In addition, finite sliding formulation and the
 8 surface-to-surface discretisation method are used.

9 In numerical simulation, mechanical behaviour of the contact surfaces are
 10 divided into normal behaviour and tangential behaviour. Normal behaviour adopts
 11 hard contact which uses Lagrange Multipliers to enforce contact constraints. (Nguyen
 12 et al. 2017) In tangential behavior, penalty friction formulation is adopted and
 13 contact-pressure-dependent data is used to simulate the Mohr-Coulomb failure
 14 criterion between the contact surfaces of foundation and soil.

15

16 **2.4 Boundary conditions and seismic motion input method**

17 When using numerical methods to solve the dynamic soil-structure interaction
 18 problem, it is generally inevitable to extract a finite calculation area from the infinite
 19 medium. The simulation of the infinite foundation can be achieved by introducing
 20 artificial boundary condition (ABC) on boundaries of this area to ensure that
 21 boundary can efficiently absorb the scattering waves energy (Liu et al. 2006).

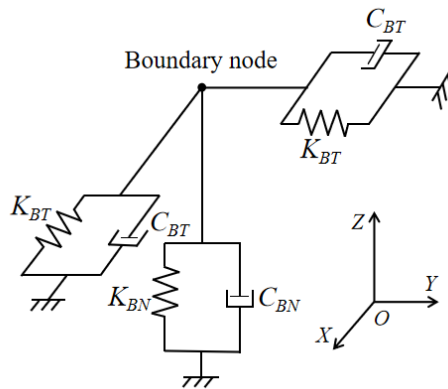
1 Compared with viscous boundary, viscous-spring boundary can absorb the
 2 energy of the scattering waves on the boundary and capture the recovery ability of a
 3 semi-infinite ground without the problems of high frequency instability and low
 4 frequency drift (Gu et al. 2007). Therefore, the viscous-spring boundary is adopted.

5 To realise the viscous-spring boundary, the general approach is to set parallel
 6 springs and dampers in one normal and two tangential directions of the boundary
 7 nodes (Gu et al. 2007) (Fig. 4). The mechanical coefficients of the springs and
 8 dampers are calculated by the property of the surrounding soil medium and the
 9 specific calculation formula is as follows:

$$10 \quad K_{BT} = \alpha_T G/R, C_{BT} = \rho c_s \quad (1)$$

$$11 \quad K_{BN} = \alpha_N G/R, C_{BN} = \rho c_p \quad (2)$$

12 Where K_{BT} , K_{BN} are the stiffness coefficients of springs in tangential and normal
 13 directions, respectively; C_{BT} , C_{BN} are the damping coefficients of dampers in
 14 tangential and normal directions, respectively; α_T , α_N are modified coefficients and
 15 their value referred to the study of Liu (2006), $\alpha_T=0.67$, $\alpha_N=1.33$; R is the distance
 16 from the wave source to boundary nodes; ρ and G are the density and shear modulus
 17 of the soil deposit, respectively; c_s and c_p are shear wave velocity and P wave velocity
 18 of the soil deposit, respectively.



19
 20 Fig. 4 viscous-spring boundary

21 When the viscous-spring boundary is used to completely absorb the scattering
 22 wave from the calculated area, the artificial boundary node is subjected to the free
 23 field motion. Thus, the problem of seismic input is transformed into the problem of

1 free field motion acting on boundary nodes, and the free field motion can be
2 transformed into the equivalent node force F_b acting on boundary nodes (Ma 2020).
3 The equivalent node force F_b includes three parts: the first two parts can eliminate the
4 impacts of viscous-spring boundary, and the third part is the stress generated by free
5 field motion on the artificial boundary:

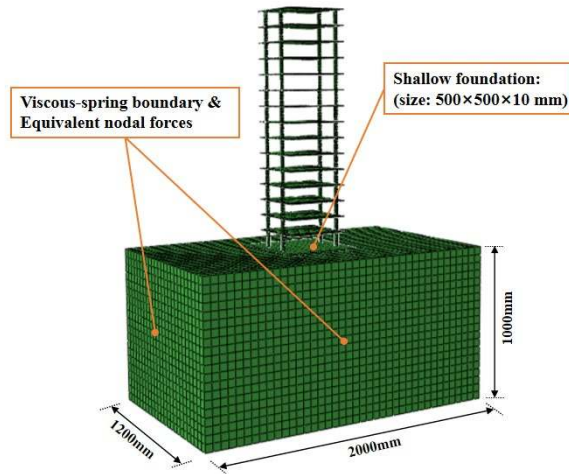
$$6 \quad F_b = (K_b u_b^{ff} + C_b v_b^{ff} + \sigma_b^{ff} \mathbf{n}) A_b \quad (3)$$

7 Where $u_b^{ff} = [u_x \ v_y \ w_z]^T$ is free field displacement vector at artificial boundary nodes,
8 $v_b^{ff} = [v_x \ v_y \ v_z]^T$ is free field velocity vector at artificial boundary nodes, σ_b^{ff} is free
9 field stress tensor, K_b is spring stiffness of viscous-spring boundary, C_b is damping
10 coefficient of viscous-spring boundary. A_b is the influencing area of the boundary
11 node, \mathbf{n} is the cosine vector of the normal direction outside the boundaries. By
12 referring to the method of Ma (2020) and compiling a simple program in MATLAB
13 software, the equivalent nodal force amplitudes in the three directions of each node
14 can be calculated, and then the amplitude file can be read into the inp file of
15 ABAQUS, and thus the seismic equivalent nodal forces can be applied on
16 viscous-spring boundary. After that, the seismic behaviour of the soil-structure model
17 under four scaled earthquake acceleration records can be analysed.

18

19 **2.5 Validation of 3D numerical model**

20 In order to validate the numerical models introduced in this section, the results of
21 fixed-base model (Fig. 1) and the soil-structure model (Fig. 5) under the influence of
22 four scaled earthquake records (Northridge, Kobe, El-Centro, and Hachinohe) were
23 studied and compared with corresponding experimental shaking table test results
24 (Tabatabaiefar et al. 2014; Fatahi et al. 2015; Tabatabaiefar and Mansoury 2016). As
25 mentioned above, the experimental tests and the numerical model have the same
26 model dimensions and physical and mechanical parameters. The maximum lateral
27 deflections (Δ) of fixed-base and flexible-base models under each earthquake records
28 and the average values are compared in Fig. 6 and Fig. 7, respectively.



1

2

Fig. 5 Numerical grid of soil-structure model in ABAQUS

3

4

5

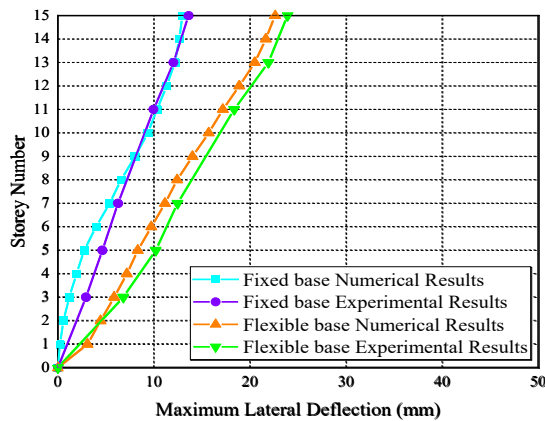
6

7

8

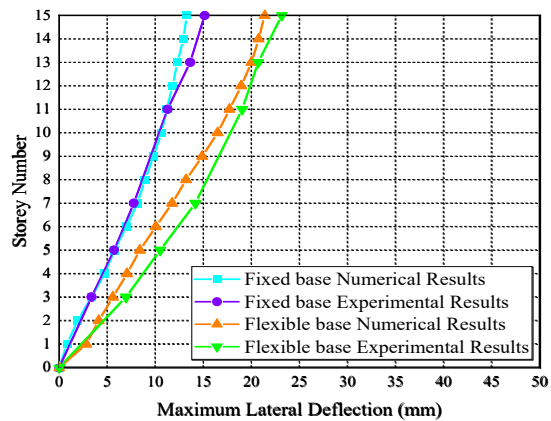
9

As observed in Fig. 6, both the fixed-base and flexible-base numerical models are accurate enough to capture the seismic response of structures under different earthquake records. The trend and the values of the seismic response in numerical simulation are in a good agreement and consistent with the experimental shaking table test results. From Fig. 7, the error of average maximum lateral deflections can be calculated and in fixed-base and flexible-base cases, the errors are only 8.8% and 5.6%, respectively.



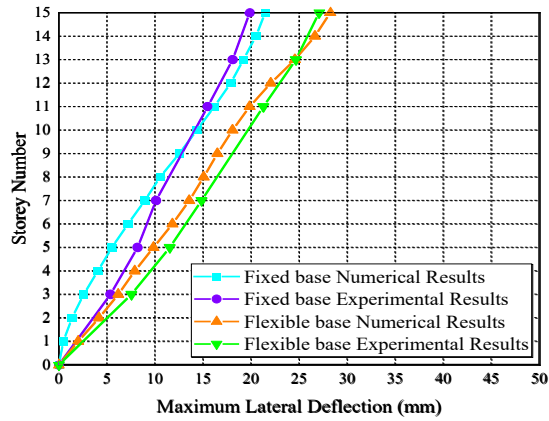
10

(a)



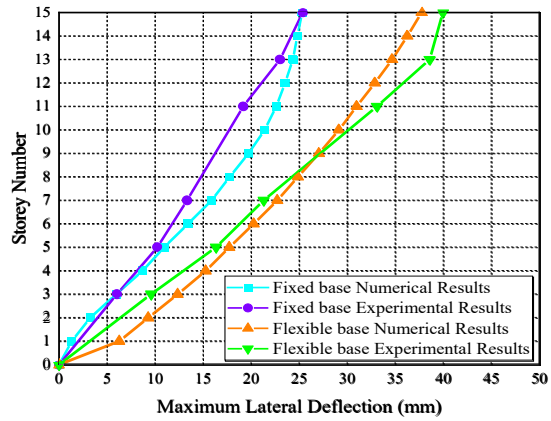
11

(b)



1

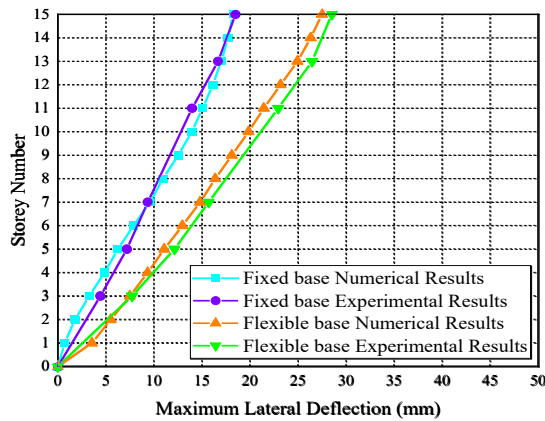
(c)



(d)

2

3 Fig. 6 Numerical and experimental maximum lateral deflections of fixed-base and flexible-base
 4 models under the scaled (a) El Centro (1940) earthquake (b) Hachinohe (1968) earthquake (c)
 5 Kobe (1995) earthquake (d) Northridge (1994) earthquake.



6

7 Fig. 7 Average values of maximum lateral deflections of numerical model and experimental
 8 model.

9

10 3 Overview of the high-rise building numerical model

11 In 1972, the International Conference on Tall Buildings divided tall buildings
 12 into four categories according to height (Mukand et al. 1973): (1) 9~16 stories (up to
 13 50 meters);(2) 17~25 stories (up to 75 meters); (3) 26~40 stories (up to 100 meters);
 14 (4) more than 40 stories (super tall buildings). Thus this paper will study three typical
 15 high-rise building heights: 20 stories (60 meters), 30 stories (90 meters) and 40 stories
 16 (120 meters). The widths of the three different height structures in this study are 10 m,
 17 15 m and 20 m respectively (height-width ratio is 6), with three spans in each

1 direction.

2 **3.1 Characteristics of the adopted high-rise structure**

3 The concrete frame-core tube structure, as a kind of commonly used high-rise
4 building structural system, is selected in this study, the typical plan view of standard
5 floor is shown in Fig. 8 (a).

6 The structural sections were designed according to the relevant standards
7 (AS3600 2018; AS1170.4 2007) in SAP2000 v 20 software. Afterwards nonlinear
8 time history analyses under four earthquake acceleration records (Fig. 9) was
9 performed and the inter-storey drifts were limited in life safe level (<1.5%). Grade 40
10 concrete with characteristic compressive strength (f'_c) of 40 MPa, modulus of
11 elasticity (E_c) of 32.8 GPa and unit weight of 24.5 kN/m³ (AS3600 2018) and Grade
12 D500N steel reinforcing bars with yielding strength (f_{sy}) of 500 MPa (AS4671 2001)
13 were selected. The dimensions of structural sections are shown in Table 2.

14 Table 2 Summary of dimensions of structural beams, columns and thickness of slabs and shear
15 walls (m)

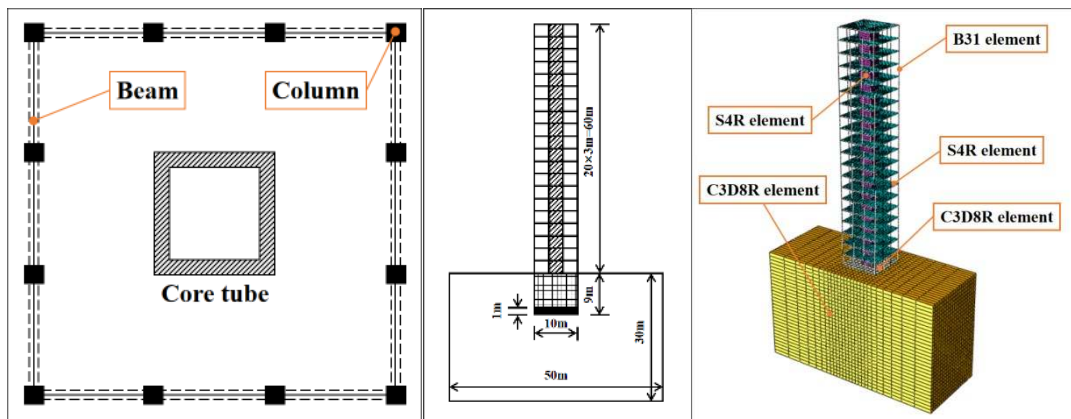
Structures	Levels	Columns	Beams	Shear walls	Slabs
20-storey	1~5	0.55×0.55	0.40×0.40	0.55	0.25
	6~10	0.50×0.50	0.40×0.40	0.50	0.25
	11~15	0.45×0.45	0.40×0.40	0.45	0.25
	16~20	0.40×0.40	0.40×0.40	0.40	0.25
30-storey	1~10	0.70×0.70	0.50×0.50	0.70	0.25
	11~20	0.60×0.60	0.50×0.50	0.60	0.25
	21~30	0.50×0.50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

The superstructures founded on soft clayey soil deposit (Class Ee soil according to AS1170.4 2007) and the bedrock depth is 30 meters. Geotechnical characteristics of the subsoil are summarised in Table 3 (Tabatabaiefar et al. 2014).

Table 3 Geotechnical characteristics of the ground soils in this study

Shear wave velocity (m/s)	Unified classification (USCS)	G_{max} (kPa)	Poisson's ratio	Soil density (kg/m ³)	c' (kPa)	ϕ' (degree)	Plastic Index
150	CL	33,100	0.40	1470	20	12	15

A classical compensated foundation with three basement stories overlying a 1m-thick reinforced concrete base slab is adopted in this study (Fig. 8b), and the total embedment depth is 9 m. The general definition of compensated foundation is that in building design, if the foundation is deep enough so that the weight of the building is approximately equal to the total soil weight removed from the building position, the soil weight can compensate the weight of the building. If the foundation bottom pressure is exactly equal to the soil gravity stress, the additional stress of foundation bottom is zero, theoretically the foundation will not suffer any settlement and shear failure. The reason why compensated foundation is selected is that compared with piled foundation, compensated foundation can produce greater foundation rotation, so that the superstructure can experience more obvious lateral deflection. In this study, the requirements for bearing capacity and maximum settlement are satisfied (Bowles 2001). For 20-, 30- and 40-storey structures, vertical settlements under gravitational (static) loads are 13.4 mm, 24.8mm and 38.8 mm, respectively.



(a)

(b)

(c)

1 Fig. 8 Characteristics of the adopted frame-core tube structure (a) plan view of standard floor (b)
 2 dimensions of 20-storey frame-core tube structure (c) the finite-element model

3 3.2 Numerical analysis

4 In soil-structure numerical model of high-rise building, shell elements S4R were
 5 employed to model shear walls and slabs, beam elements B31 were employed to
 6 model beams and columns, and solid elements C3D8R were employed to model
 7 foundation and soil medium (Fig. 8c). The contact surface properties, boundary
 8 conditions and loading methods of soil-structure numerical model are the same as
 9 those introduced in Section 2.

10 After the numerical model is established for three soil-structure models with
 11 different heights, four bedrock earthquake ground motions (Fig. 9), including two
 12 far-filed earthquakes and two near-filed earthquakes are applied to fixed-base and
 13 flexible-base model respectively. The characteristics of the earthquake ground
 14 motions are summarised in Table 4. The strain-compatible parameters and Rayleigh
 15 damping parameters of subsoil under four earthquake acceleration records are
 16 summarised in Table 5.

17 Table 4 Earthquake ground motions adopted in this study

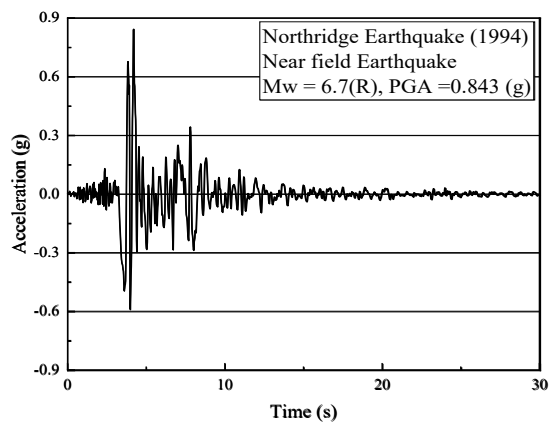
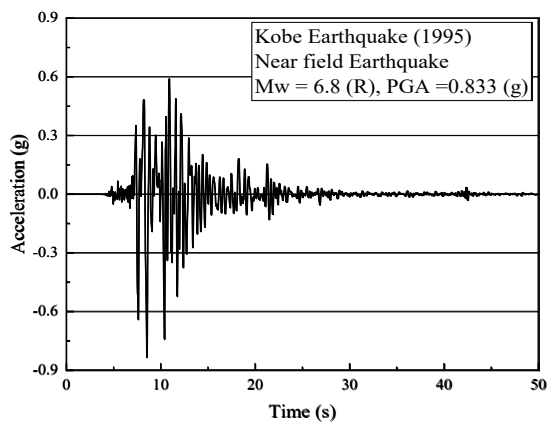
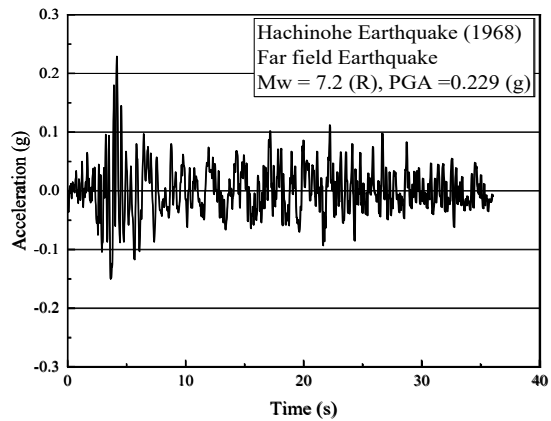
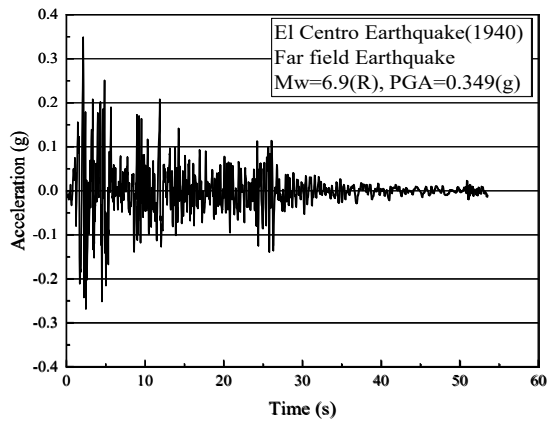
Earthquake	Country	Year	PGA (g)	Mw (R)	T (s) Duration	Type	Hypocentral distance (km)	Record type
El Centro	USA	1940	0.349	6.9	56.5	Far field	15.69	Bedrock record
Hachinohe	Japan	1968	0.229	7.5	36.0	Far field	14.1	Bedrock record
Kobe	Japan	1995	0.833	6.8	50.0	Near field	7.4	Bedrock record
Northridge	USA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

18

19

1 Table 5 Adopted strain-compatible parameters and damping parameters for the soil-high-rise
 2 structure system

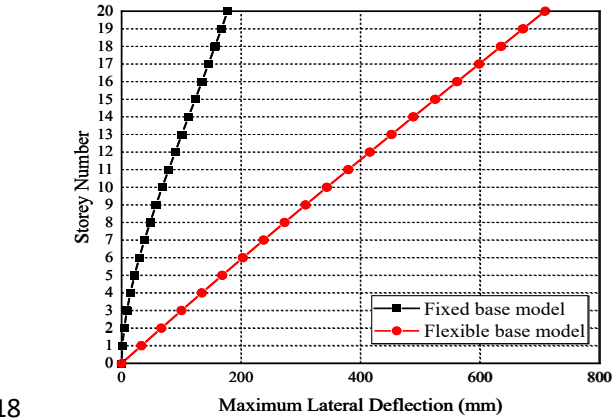
Earthquake records	G/G_{max}	ξ	Damping coefficients
El-Centro	0.57	11.1%	$\alpha=0.769$ $\beta=0.012$
Hachinohe	0.60	10.4%	$\alpha=0.284$ $\beta=0.024$
Kobe	0.35	17.0%	$\alpha=1.043$ $\beta=0.021$
Northridge	0.21	23.5%	$\alpha=1.415$ $\beta=0.029$



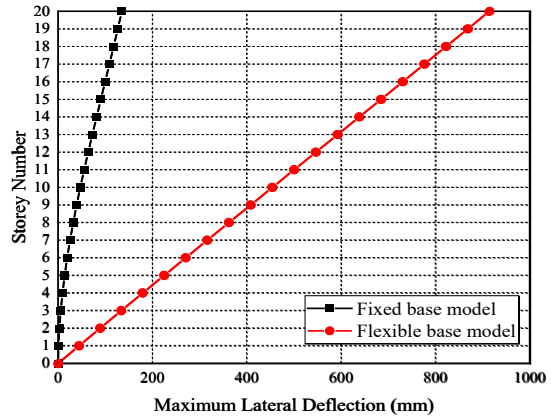
3
 4 (a) (b)
 5 (c) (d)
 6 Fig. 9 Earthquake records: (a) El Centro earthquake (b) Hachinohe earthquake (c) Kobe
 7 earthquake (d) Northridge earthquake
 8
 9

1 **4 Results and discussions**

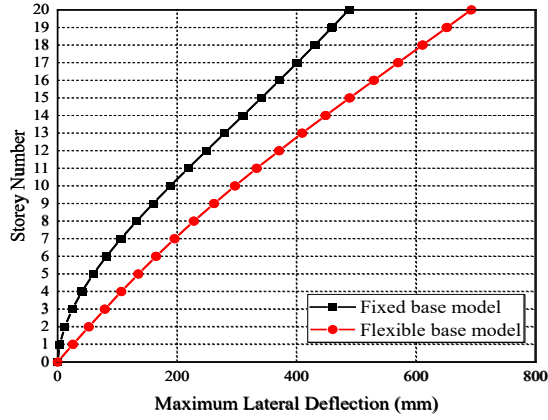
2 Fig. 10, 11 and 12 show the maximum lateral deflections of three frame-core
 3 tube structures with different heights under fixed-base and flexible-base conditions.
 4 As observed, the maximum deflections of almost all cases increased, but there is a
 5 noticeable difference in the deflections between the two far field earthquakes and the
 6 two near field earthquakes. Under the two far field earthquakes, the amplification
 7 effect is very obvious (average increase is 301% for the three flexible-base structure
 8 with different heights), but under the two near field earthquakes, the average
 9 maximum lateral deflections of the structures considering SSI at three heights only
 10 increase by 51%. It should be pointed out that the maximum displacement of the
 11 flexible-base 40-storey structure under the Northridge earthquake is less than that of
 12 the fixed-base structure (from 1184.4 mm to 1042.7 mm). This is due to the fact that
 13 the displacement response spectrum curve usually increases first and then decreases.
 14 The natural period of 40-story structure increases from 2.660 seconds of rigid-base
 15 case to 3.817 seconds of flexible-base case, which is in the descending region of the
 16 displacement response spectrum. Therefore, the displacement of the flexible-base
 17 model decreases rather than increases.



19 (a)

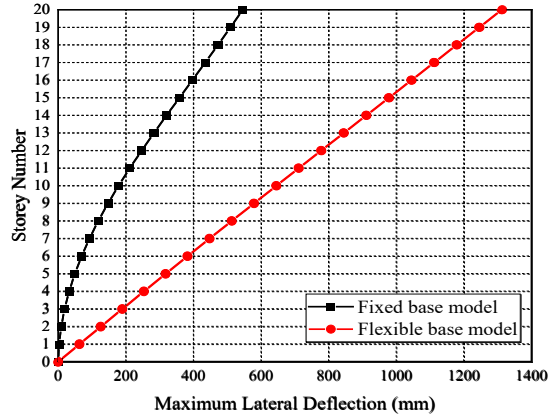


(b)



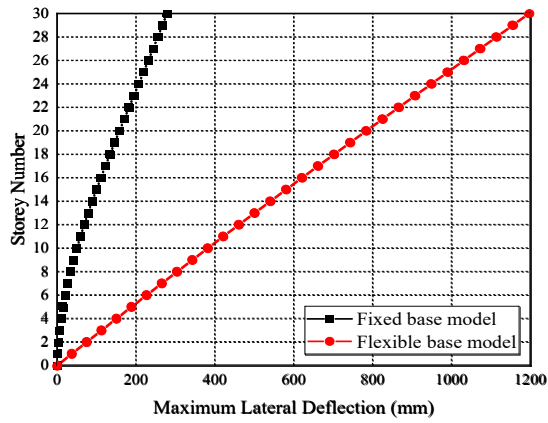
1

(c)



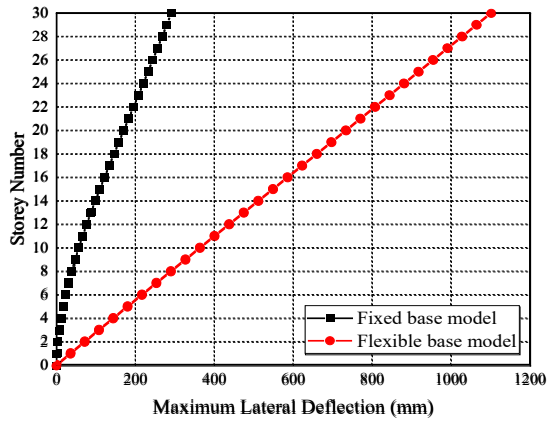
(d)

Fig. 10 Maximum lateral deflections of 20-story structure under following earthquakes: (a) El Centro earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake

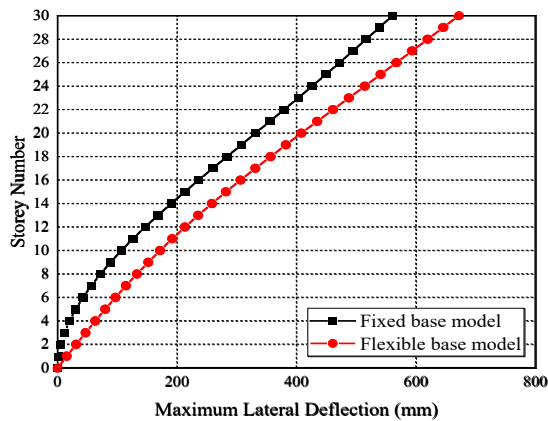


5

(a)

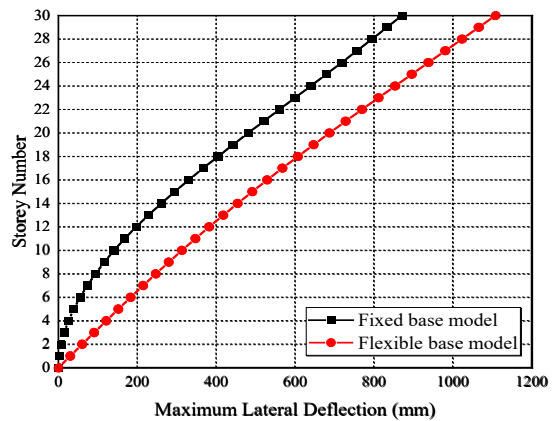


(b)



7

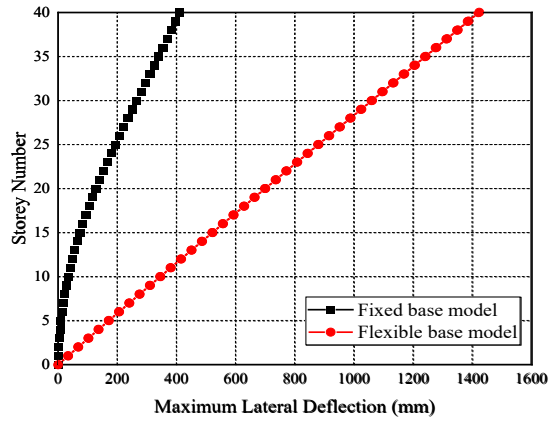
(c)



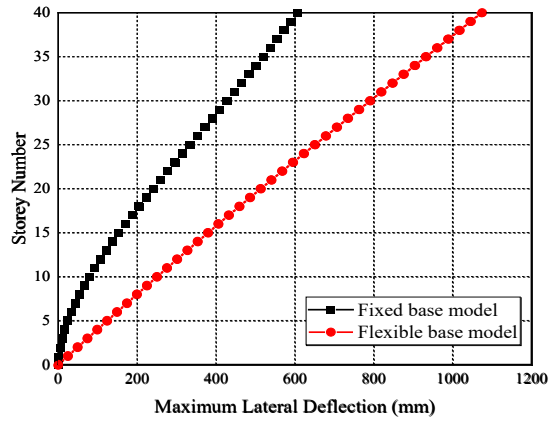
(d)

Fig. 11 Maximum lateral deflections of 30-story structure under the following earthquakes: (a) El Centro earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake

10



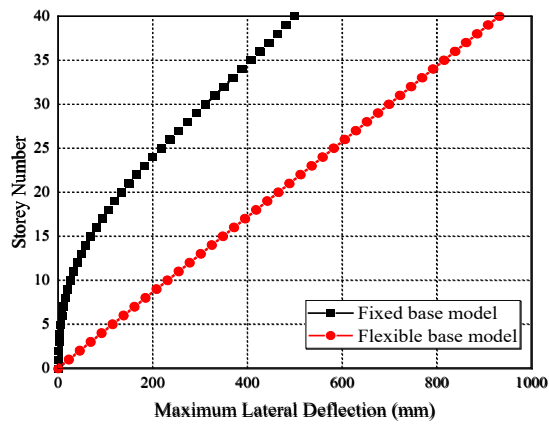
1



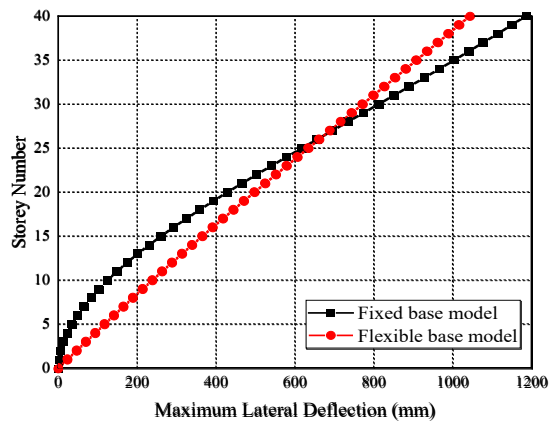
2

(a)

(b)



3



4

(c)

(d)

5 Fig. 12 Maximum lateral deflections of 40-story structure under the following earthquakes: (a) El
6 Centro earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake

7

8 Lateral deflections under the impacts of the SSI includes rocking and distortion
9 components (Kramer 1996). For high-rise buildings, the rocking component plays a
10 significant role in the lateral deflection. Table 6 records the rocking angle of the
11 foundation (θ_f) and the lateral displacement of the top floor caused by foundation
12 rocking (Δ_r) at the moment when structures reach the maximum lateral displacements.
13 For 20-story, 30-story and 40-story structures, 92.01%, 87.28% and 94.31% of the
14 maximum lateral displacements of the roof is due to the rocking components, which is
15 much larger than the proportion of the frame structure. On the one hand, the type of
16 foundation plays a key role in the foundation rocking. In addition, the overall stiffness
17 of the frame-core tube structure is much larger than that of the frame structure, and
18 the structure is more likely to rotate overall rather than deform between adjacent

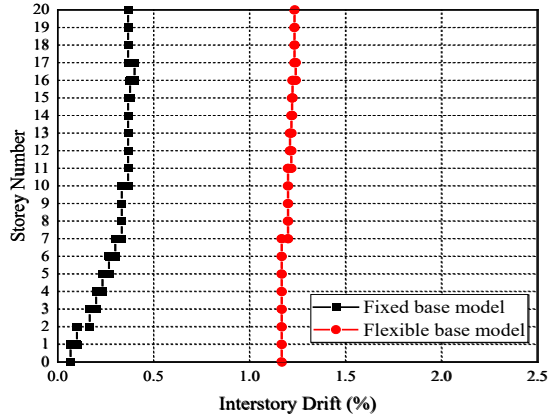
1 stories.

2 Table 6 Foundation rocking angle (θ_f), lateral deflection due to foundation rocking (Δ_r) and the
3 proportion of lateral deflection due to foundation rocking in total lateral deflection (Δ_r/Δ)

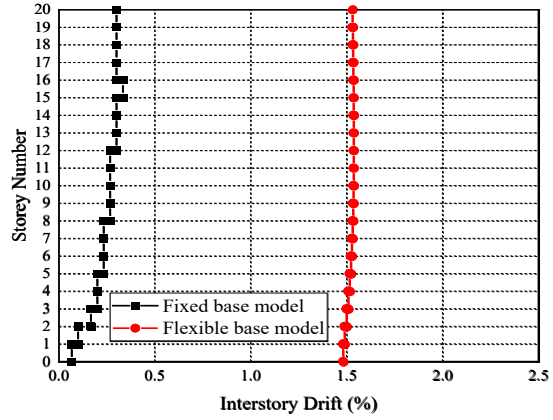
Earthquakes	20-storey structure			30-storey structure			40-storey structure		
	θ_f	Δ_r	Δ_r/Δ	θ_f	Δ_r	Δ_r/Δ	θ_f	Δ_r	Δ_r/Δ
	(°)	(mm)	(%)	(°)	(mm)	(%)	(°)	(mm)	(%)
El Centro	0.64	667.17	94.85	0.71	1120.04	93.66	0.65	1354.30	95.27
Hachinohe	0.84	877.92	96.06	0.68	1060.79	96.36	0.47	990.30	92.27
Kobe	0.54	562.60	81.30	0.32	499.19	74.36	0.43	913.64	98.08
Northridge	1.2	1257.25	95.83	0.60	938.14	84.72	0.46	955.11	91.60

4

5 Studies on the determined inter-story drifts also confirm this viewpoint (Fig. 13,
6 14 and 15). The corresponding inter-story drifts were calculated using the method
7 based on AS1170-4 (2007). As observed, compared with the inter-storey drifts of
8 rigid-base structure, the inter-storey drifts of the structure considering SSI presents an
9 approximately vertical straight line, indicating that the inter-storey drifts does not
10 change much with the height, and the overall deflection curve is approximately linear.
11 The effect of SSI normally increases the inter-story drifts in structures. However,
12 similar to the maximum lateral deflections, the variation of inter-story drifts is also
13 related to the earthquake records. For example, in the far field earthquake cases, the
14 inter-story drifts of the structures considering SSI increase by 194% on average, while
15 in the near filed earthquake cases, the inter-story drifts of the 30-storey structure
16 almost does not increase, and the inter-story drifts of the 40-storey structure even
17 decreases by 5.7% (Kobe earthquake) and 25.4% (Northridge earthquake),
18 respectively. As mentioned above, this is also related to the variation of the natural
19 period of the structure and the shape of the seismic response spectrum curve. In
20 general, SSI may lead to the change of the structural performance level. For instance,
21 the maximum inter-storey drifts of 20-storey flexibly-supported building is more than
22 1.5% under the Hachinohe and Northridge earthquakes, and the performance levels
23 change towards near-collapse level (BSSC 1997).



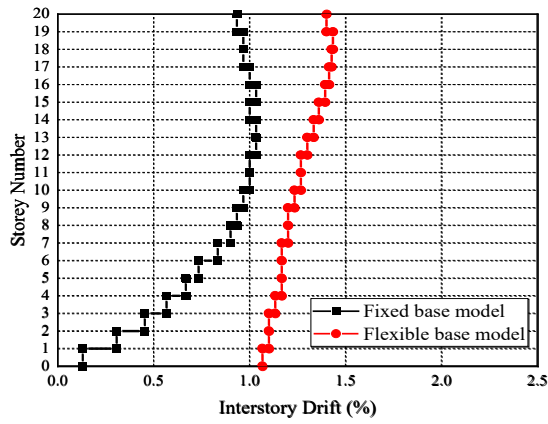
1



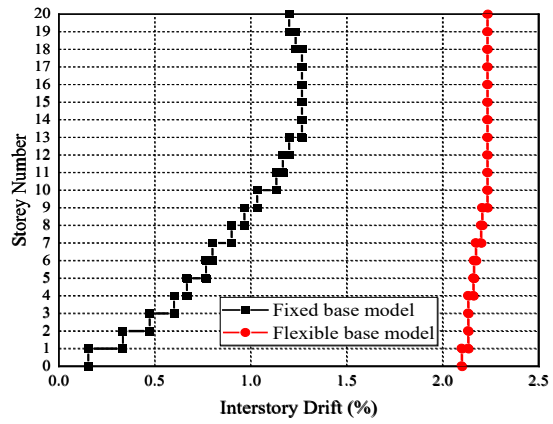
2

(a)

(b)



3

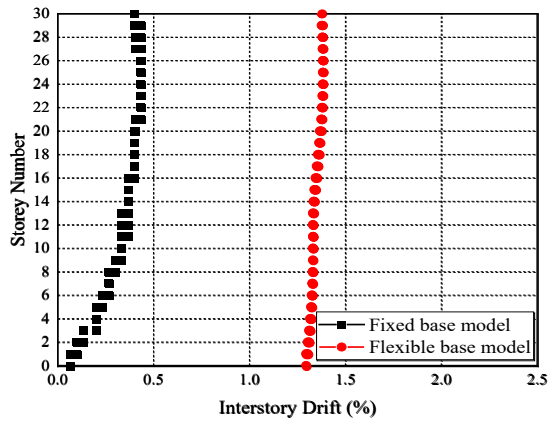


4

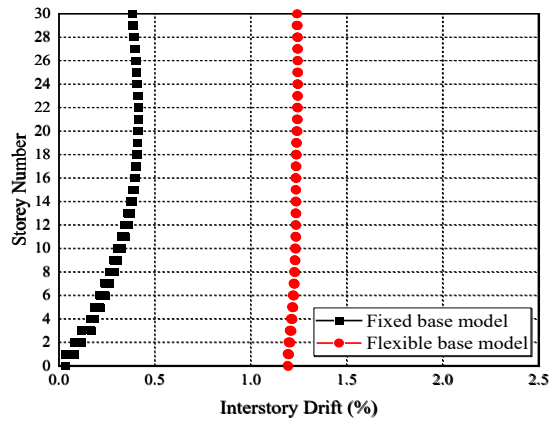
(c)

(d)

5 Fig. 13 Inter-story drifts of 20-story structure under the following earthquakes: (a) El Centro
6 earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake



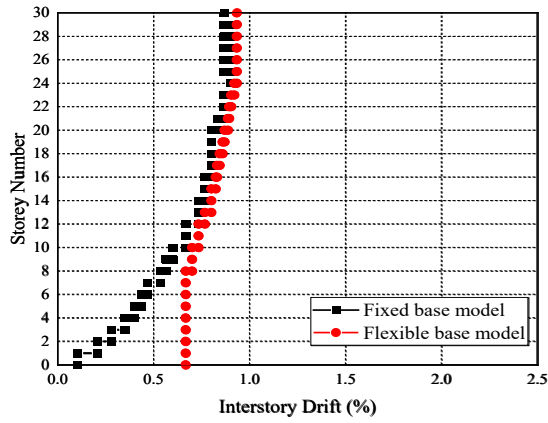
7



8

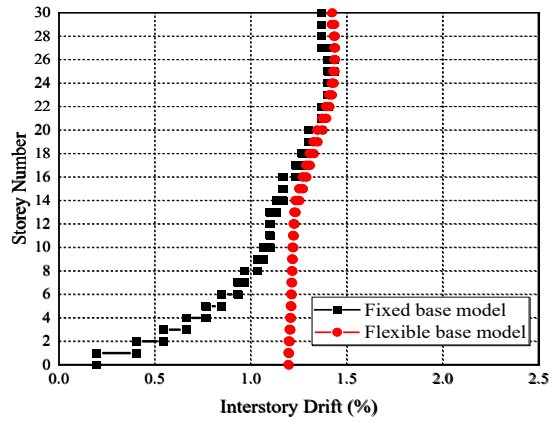
(a)

(b)



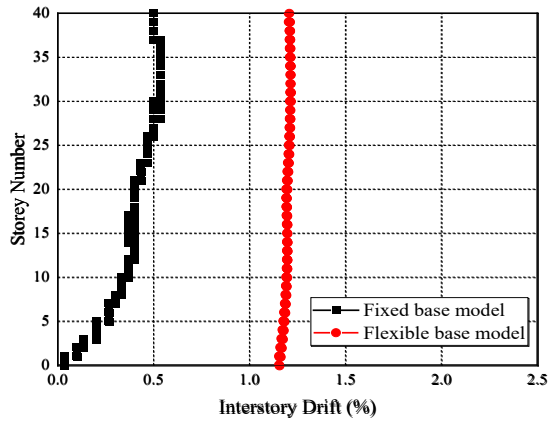
1

(c)



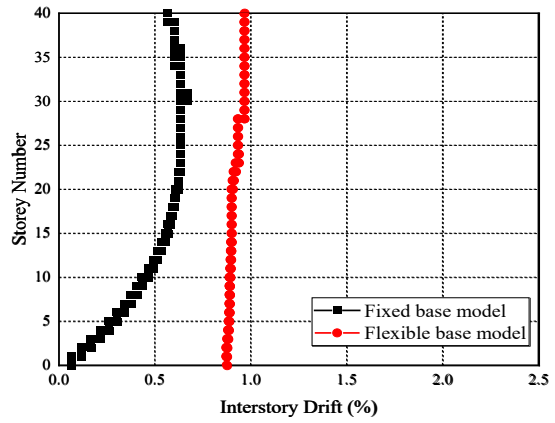
(d)

Fig. 14 Inter-story drifts of 30-story structure under the following earthquakes: (a) El Centro earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake

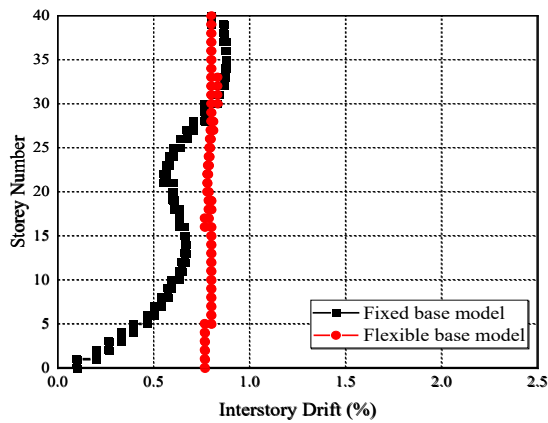


5

(a)

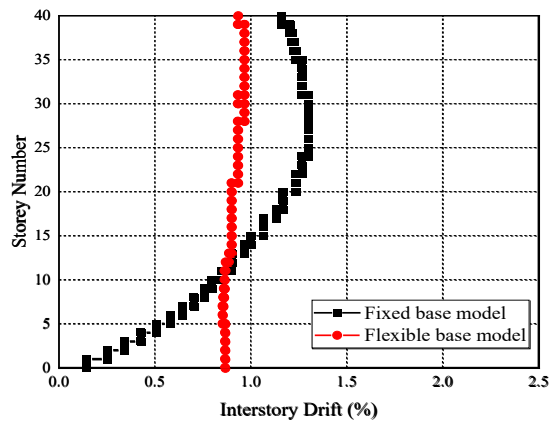


(b)



7

(c)



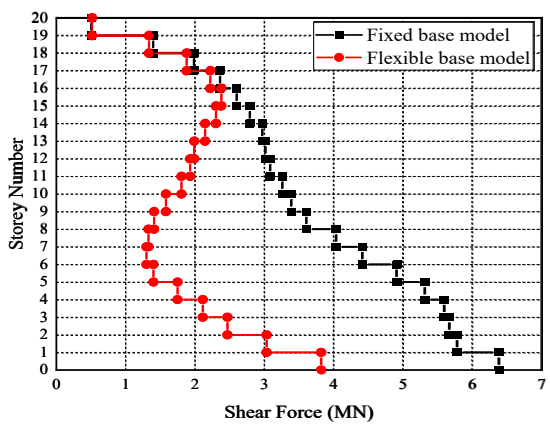
(d)

Fig. 15 Inter-story drifts of 40-story structure under the following earthquakes: (a) El Centro earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake

Fig. 16, 17 and 18 compare the shear forces of three structures with different heights. Similar to the frame structure (Tabatabaiefar et al. 2013), the storey shear

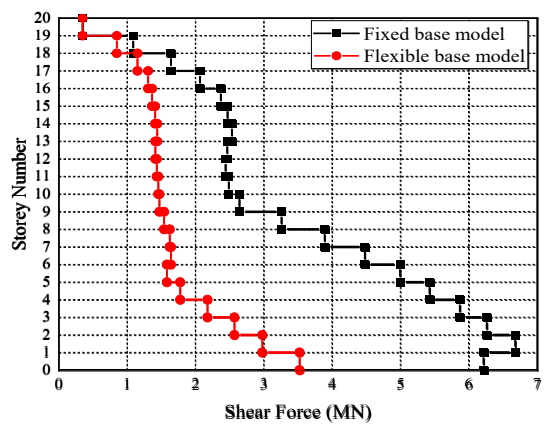
11
12

1 forces of frame-core tube structures decrease to some extent under the influence of the
 2 SSI. For the two far field earthquakes, the average base shear force of flexible-base
 3 structure is about 54.3% of that of fixed-base structure, while the average ratio of the
 4 two near filed earthquakes is 30.8%. This is because the spectral acceleration (S_a)
 5 curves of the four earthquake records basically show monotonous downward trends
 6 within the natural period range of structures. However, it should be pointed out that
 7 not all storey shear forces is reduced. For example, under the Kobe earthquake of
 8 20-storey structure, the shear forces above 16 storey increase after considering SSI.



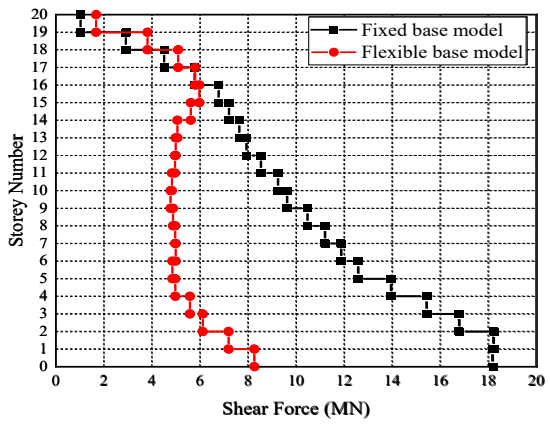
9

(a)



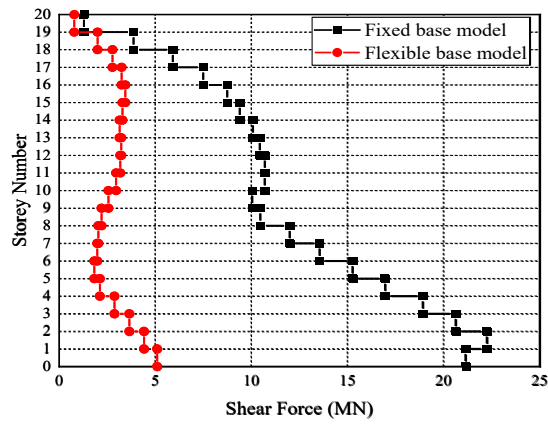
10

(b)



11

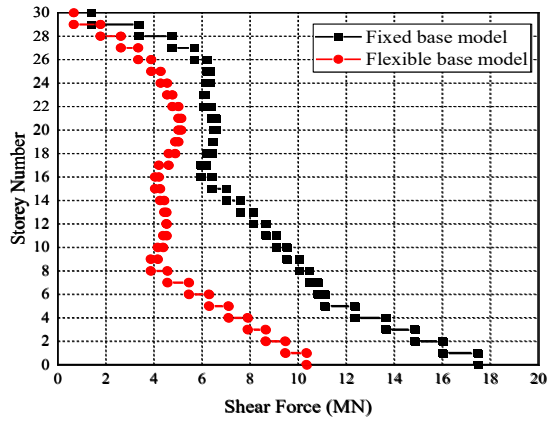
(c)



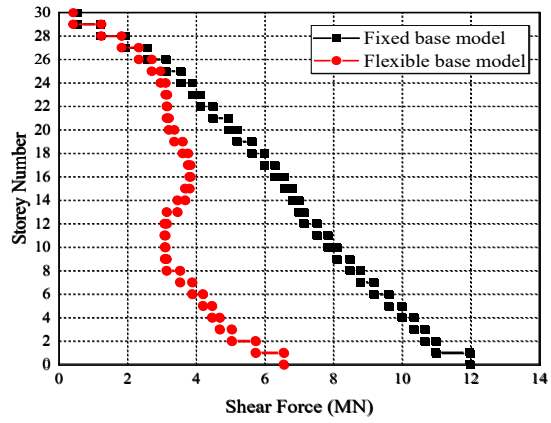
12

(d)

13 Fig. 16 Storey shear forces of 20-storey structure under the following earthquakes: (a) El Centro
 14 earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake



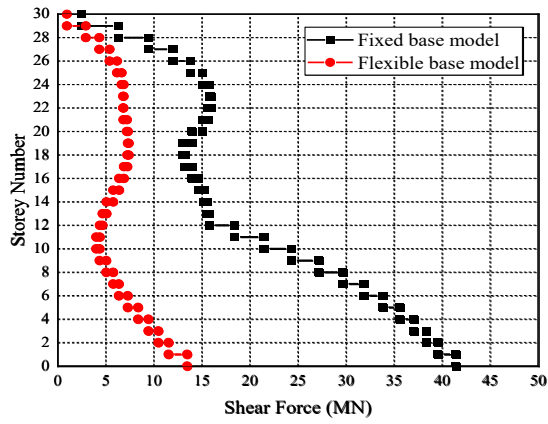
1



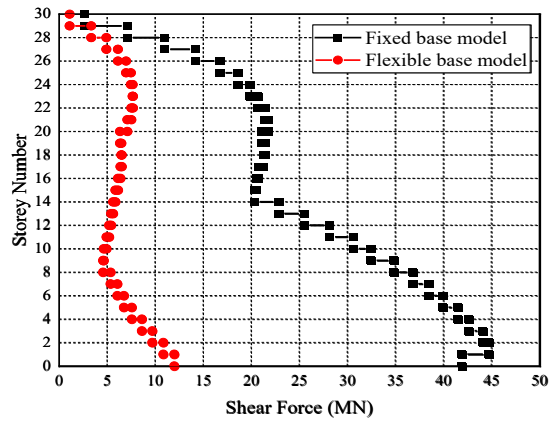
2

(a)

(b)



3

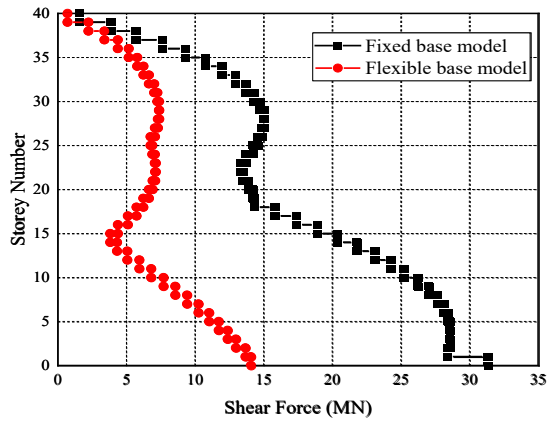


4

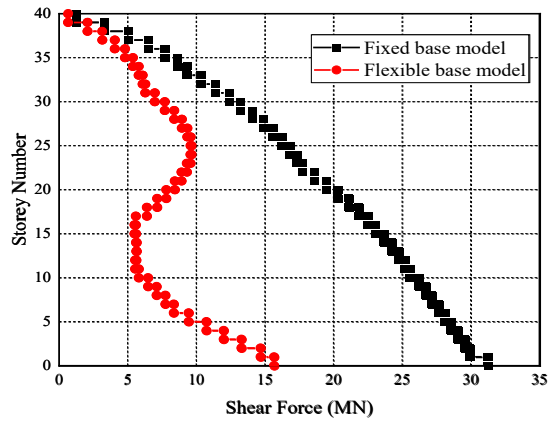
(c)

(d)

5 Fig. 17 Storey shear forces of 30-story structure under the following earthquakes: (a) El Centro
6 earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake



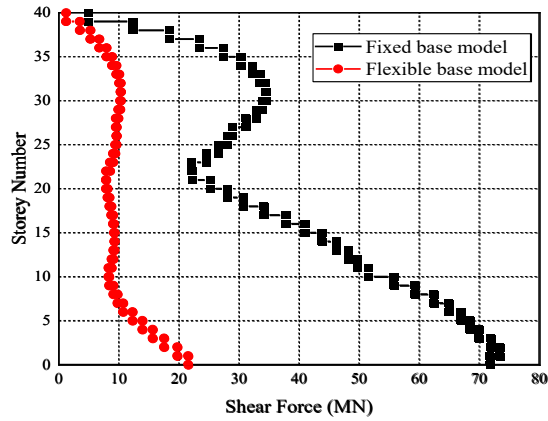
7



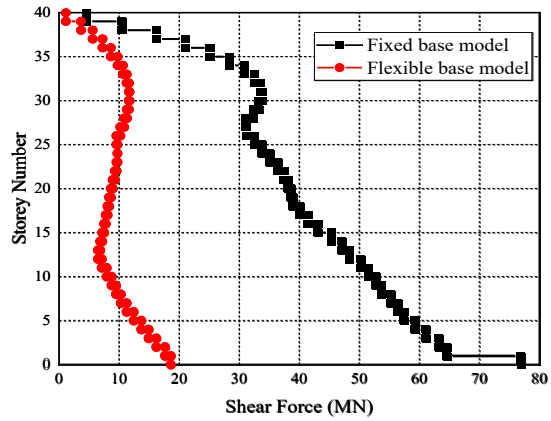
8

(a)

(b)



(c)



(d)

Fig. 18 Storey shear forces of 40-story structure under the following earthquakes: (a) El Centro earthquake (b) Hachinohe earthquake (c) Kobe earthquake (d) Northridge earthquake

5 Conclusions

In this study, a novel and enhanced soil-structure model is developed in finite element software ABAQUS to investigate the influence of SSI on high-rise buildings. Firstly, the accuracy of the numerical model are verified according to the previous shaking table test's results. After that, numerical simulations were conducted on three types of frame-core tube structures: 20, 30, and 40- story buildings. For each case, two near field and two far field earthquakes were applied to rigid-base and flexible-base models, respectively. According to the results of numerical simulation, this study can draw the following conclusions:

Under the two far field earthquake records, the increase of the maximum lateral deflections of structures with different heights considering SSI is much more obvious than that of the structures under the action of two near field earthquakes. In general, SSI can result in an increase in the maximum lateral deflection, but in terms of high-rise structures, its natural period may be in the descending region of displacement response spectrum curve, which may lead to a decrease in the maximum lateral deflection. Therefore, in practical engineering, the effect of SSI does not necessarily amplify the structure lateral deflection as the height of the structure increases. The structural height and the ground motion features should be considered to determine whether the effects of SSI are beneficial or detrimental.

1 Different from frame structures, for frame-core tube structures, the maximum
2 lateral deflection caused by the foundation rocking accounts for a large proportion.
3 The reduction of the foundation rocking can effectively reduce the lateral deflections
4 of buildings. Therefore, the lateral deflection of the structure can be effectively
5 reduced by using pile foundation or increasing the width of foundation.

6 The inter-storey drifts of structures considering SSI do not change much with the
7 height, and the overall deflection curve of the structure is approximately linear, which
8 is also caused by the large proportion of foundation rocking. Generally ignoring SSI is
9 safety threatening because it may change the performance level of buildings from life
10 safe to near collapse.

11 Under different earthquake records, the base shear of frame-core tube structures
12 decrease to different extents after considering SSI. However, it should be pointed out
13 that in most cases in this study, the storey shear forces of the topmost stories are
14 almost not reduced. Hence, in designing high-rise buildings, it is not reasonable to use
15 the same shear force reduction factor for all stories in a structure.

16 In short, SSI has a noticeable influence on the seismic behaviour of high-rise
17 frame-core tube structures. There are also notable differences in the seismic behaviour
18 of soil-structure model under near and far field earthquakes. Therefore, ignoring SSI
19 or considering inappropriate seismic records in real construction projects can
20 misestimate the damage level of structures and cause unsafe consequences.

21

22

1 **Reference**

- 2 Anand V, Satish Kumar SR (2018) Seismic soil-structure interaction: a state-of-the-art
3 review. Structures 16:317-326.
- 4 Arboleda-Monsalve LG, Mercado JA, Terzic V, Mackie KR (2020) Soil-structure
5 interaction effects on seismic performance and earthquake-induced losses in tall
6 buildings. Journal of Geotechnical and Geoenvironmental Engineering
7 146(5):04020028.
- 8 AS1170.4 (2007) Structural Design Actions - Part 4: Earthquake Actions in Australia.
9 Australian Standards, Sydney.
- 10 AS3600 (2018) Concrete Structures. Australian Standards, Sydney.
- 11 AS4671 (2001) Steel reinforcing materials. Standards Australia, Sydney, Australia.
- 12 Bagheri M, Jamkhaneh ME, Samali B (2018) Effect of seismic soil-pile-structure
13 interaction on mid- and high-rise steel buildings resting on a group of pile
14 foundations. International Journal of Geomechanics 18(9):04018103.
- 15 Bilotta E, De Sanctis L, Di Laora R, D'onofrio A, Silvestri F (2015) Importance of
16 seismic site response and soil-structure interaction in dynamic behaviour of a tall
17 building. Géotechnique 65(5): 391-400.
- 18 Bowles JE (2001) Foundation analysis and design. 5th Ed., McGraw-Hill
19 International, New York.
- 20 Building Seismic Safety Council (BSSC) (1997) NEHRP guidelines for the seismic
21 rehabilitation of buildings. 1997 edition, Part 1: Provisions and Part 2:
22 Commentary. FEMA 273/274, FEMA, Washington, DC.
- 23 Far, H., Flint, D. (2017) Significance of using isolated footing technique for
24 residential construction on expansive soils Frontiers of Structural and Civil
25 Engineering, 11(1), pp. 123–129
- 26 Far H (2019a) Advanced computation methods for soil structure interaction analysis
27 of structures resting on soft soils. International Journal of Geotechnical
28 Engineering 13(4):352-359.
- 29 Far H (2019b) Dynamic behaviour of unbraced steel frames resting on soft ground.

1 Steel Construction 12(2):135-140.

2 Fatahi B, Tabatabaiefar HR, Samali B (2011) Performance based assessment of
3 dynamic soil-structure interaction effects on seismic response of building frames.
4 In C. H. Juang, K. K. Phoon, A. J. Puppala, R. A. Green, & G. A. Fenton (Eds.),
5 PROCEEDINGS OF GEORISK 2011-Geotechnical Risk Assessment &
6 Management (Geotechnical Special Publication No. 224) (pp. 344-351). USA:
7 American Society of Civil Engineers (ASCE).

8 Fatahi B, Tabatabaiefar HR (2014) Effects of soil plasticity on seismic performance of
9 mid-rise building frames resting on soft soils. *Advances in Structural
10 Engineering, An International Journal* 17(10):1387-1402.

11 Fatahi B, Tabatabaiefar HR, Ghabraie K, Zhou W H (2015) Evaluation of numerical
12 procedures to determine seismic response of structures under influence of
13 soil-structure interaction. *Structural Engineering and Mechanics* 56(1):27-47.

14 Galal K, Naimi M (2008) Effect of soil conditions on the response of reinforced
15 concrete tall structures to near-fault earthquakes. *The Structural Design of Tall
16 and Special Buildings* 17(3):541-562.

17 GB50011 (2010) Code for Seismic Design of Buildings. China Architecture &
18 Building Press, Beijing.

19 Ghandil M, Behnamfar F (2017) Ductility demands of MRF structures on soft soils
20 considering soil-structure interaction. *Soil Dynamics and Earthquake
21 Engineering* 92:203-214.

22 Gu Y, Liu JB, Du YX (2007) 3D consistent viscous-spring artificial boundary and
23 viscous-spring boundary element. *Engineering Mechanics* 24(12):31-37.

24 Guin J, Banerjee PK (1998) Coupled soil-pile-structure interaction analysis under
25 seismic excitation. *Journal of Structural Engineering* 124:434-444.

26 Han Y (2002) Seismic response of tall building considering soil-pile-structure
27 interaction. *Earthquake Engineering and Engineering Vibration* 1(1): 57-64.

28 IBC (2012) International Building Code. International Code Council (ICC).

29 Kramer SL (1996) Geotechnical earthquake engineering. Prentice Hall, Upper Saddle
30 River, NJ.

- 1 Liu JB, Du YX, Du XL, Wang ZY, Wu J (2006) 3D viscous-spring artificial boundary
2 in time domain. *Earthquake Engineering and Engineering Vibration* 5(1):93-102.
- 3 Ma SJ, Chi MJ, Chen HJ, Chen S (2020) Implementation of viscous-spring boundary
4 in ABAQUS and comparative study on seismic motion input methods. *Chinese*
5 *Journal of Rock Mechanics and Engineering* 39(7):1445-1457.
- 6 Meymand PJ (1998) Shaking table scale model tests of nonlinear soil-pile
7 superstructure interaction in soft clay. PhD thesis in Civil Engineering,
8 University of California, Berkeley.
- 9 Mohammadia R, Massumi A, Meshkat-Dinib A (2015) Structural reliability index
10 versus behavior factor in RC frames with equal lateral resistance. *Earthquakes*
11 *and Structures* 8(5):995-1016.
- 12 Monavari B, Massumi A (2012) Estimating displacement demand in reinforced
13 concrete frames using some failure criteria. *International Journal of Advanced*
14 *Structural Engineering* 4(1):4.
- 15 Mukand J, Padmanabhan P, Arya AS, et al. National conference on tall buildings. New
16 Delhi 1973.
- 17 Mylonakis G, Gazetas G (2000) Seismic soil-structure interaction: beneficial or
18 detrimental? *Journal of Earthquake Engineering* 4(3):277-301.
- 19 National Building Code of Canada (NBCC) (2010) NRC Institute for Research in
20 Construction, Canada.
- 21 Nguyen QV, Fatahi B, Hokmabadi AS (2017) Influence of size and load-bearing
22 mechanism of piles on seismic performance of buildings considering
23 soil-pile-structure interaction. *Int J Geomech* 17(7):04017007.
- 24 NZS1170.5-2007 Structural Design Actions-Part 5: Earthquake Actions-New Zealand,
25 New Zealand Standards, Wellington.
- 26 Park D, Hashash YMA (2003). Soil damping formulation in nonlinear time domain
27 site response analysis. *J Earthquake Eng*, 8(2):249-274.
- 28 Qaftan OS, Toma-Sabbagh T, Weekes L, Augusthus-Nelson L (2020) Validation of a
29 finite element modelling approach on soil-foundation-structure interaction of a
30 multi-storey wall-frame structure under dynamic loadings. *Soil Dynamics and*

- 1 Earthquake Engineering 131:106041.
- 2 Samali, B., Fatahi, B. & Far, H. (2011) Seismic behaviour of Concrete Moment
3 Resisting Buildings on Soft Soil Considering Soil-Structure Interaction,
4 Proceedings of the 21st Australasian Conference on the Mechanics of Structures
5 and Materials (ACMSM21), pp. 407-412.
- 6 Samimifar M, Massumi A, Moghadam AS (2019) A new practical equivalent linear
7 model for estimating seismic hysteretic energy demand of bilinear systems.
8 Structural Engineering and Mechanics 70(3):289-301.
- 9 Saleh, A., Far, H., Mok, L. (2018) Effects of different support conditions on
10 experimental bending strength of thin walled cold formed steel storage upright
11 frames Journal of Constructional Steel Research, 150, pp. 1–6.
- 12 Scarfone R, Morigi M, Conti R (2020) Assessment of dynamic soil-structure
13 interaction effects for tall buildings: A 3D numerical approach. Soil Dynamics
14 and Earthquake Engineering 128:105864.
- 15 Seed HB, Murarka R, Lysmer J, Idriss IM (1976) Relationships of maximum
16 acceleration, maximum velocity, distance from source, and local site conditions
17 for moderately strong earthquakes. Bulletin of the Seismological Society of
18 America 66(4):1323-1342.
- 19 Sharma N, Dasgupta K, Dey A (2018) A state-of-the-art review on seismic SSI studies
20 on building structures. Innovative Infrastructure Solutions 3(22):1-16.
- 21 Sun JI, Goleorkhi R, Seed B (1998) Dynamic module and damping ratios for
22 cohesive soils. Earthquake Engineering Research Centre, Report No.
23 UCB/EERC-88/15, University of California, Berkeley.
- 24 Tabatabaiefar HR (2012) Determining seismic response of mid-rise building frames
25 considering dynamic soil-structure interaction. University of Technology Sydney.
- 26 Tabatabaiefar HR, Fatahi B, Samali B (2012) Finite difference modelling of
27 soil-structure interaction for seismic design of moment resisting building frames.
28 Australian Geomechanics Journal 47(3):113-119.
- 29 Tabatabaiefar HR, Fatahi B, Samali B (2013) Seismic behaviour of building frames

1 considering dynamic soil-structure interaction. *International Journal of*
2 *Geomechanics* 13(4):409-420.

3 Tabatabaiefar HR, Fatahi B (2014) Idealisation of soil-structure system to determine
4 inelastic seismic response of mid-rise building frames. *Soil Dynamics and*
5 *Earthquake Engineering* 66(11):339-351.

6 Tabatabaiefar HR, Fatahi B, Samali B (2014) Numerical and experimental
7 investigations on seismic response of building frames under influence of
8 soil-structure interaction. *Advances in Structural Engineering, An International*
9 *Journal* 17(1):109-130.

10 Tabatabaiefar, H.R. (2016) Detail design and construction procedure of laminar soil
11 containers for experimental shaking table tests *International Journal of*
12 *Geotechnical Engineering*, 10(4), pp. 328–336

13 Tabatabaiefar HR, Clifton T (2016) Significance of considering soil-structure
14 interaction effects on seismic design of unbraced building frames resting on soft
15 soils. *Australian Geomechanics Journal* 51(1):55-64.

16 Tabatabaiefar HR, Mansoury B (2016) Detail design, building and commissioning of
17 tall building structural models for experimental shaking table tests. *The*
18 *Structural Design of Tall and Special Buildings* 25(8):357-374.

19 Tabatabaiefar, H.R., Mansoury, B., Khadivi Zand, M.J., Potter, D. (2017) Mechanical
20 properties of sandwich panels constructed from polystyrene/cement mixed cores
21 and thin cement sheet facings, *Journal of Sandwich Structures and*
22 *Materials*, 19(4), pp. 456–481.

23 Veletsos AS, Meek JW (1974) Dynamic behaviour of building-foundation systems.
24 *Earthquake Engineering And Structural Dynamics* 3:121-138.

25 Walsh, P., Saleh, A., Far, H. (2018) Evaluation of structural systems in slender
26 high-rise buildings *Australian Journal of Structural Engineering*, 19(2), pp. 105–
27 117.

28 Wolf JP (1985) *Dynamic soil-structure interaction*. Prentice-Hall, Englewood Cliffs,
29 New Jersey.

- 1 Wolf JP (1998) Soil–structure interaction analysis in time domain. Upper Saddle
- 2 River, NJ: Prentice Hall Co.
- 3 Yang J, Lu Z, Li P (2020) Large-scale shaking table test on tall buildings with viscous
- 4 dampers considering pile-soil-structure interaction. Engineering Structures
- 5 220:110960.