Numerical simulation of shaking table tests on a soil-structure system

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ABSTRACT
Based on the results of the shaking table model tests, the three-dimensional (3D) numerical simulation and analysis of the shaking table model test of the soil-box foundation-12-story reinforced concrete (RC) frame structure system are carried out using ABAQUS software in this paper. The material nonlinearity, geometric nonlinearity, and nonlinearity of the contact interfaces between the foundation and surrounding soil are considered in the numerical models. By comparing the simulated data with the measured data of the shaking table model tests, the simulated results are well consistent with the shaking table test results. It can be verified that the numerical models are reasonable. The numerical models' key parameters are adjusted, and the influence of these parameters on the dynamic soil-structure interaction (DSSI) is analyzed. The results show that their influence is significant for studying the seismic response of the DSSI system, as the foundation buried depth decreases, the peak contact pressure and the peak inter-story drift at the top of the structure increase 30.69% and 45.24%, respectively, whereas the peak acceleration magnification factors at the top of the structure decreases 32.66%.

1. Introduction
Recently, the influence of DSSI on seismic response of structures has been followed with interest from researchers and engineers. Mylonakis et al. (Mylonakis and Gazetas 2000) reexplored the role of DSSI in the seismic response of structures using recorded motions and theoretical considerations. The results showed that in certain seismic and soil environments, an increase in the fundamental natural period of amodately flexible structure due to DSSI may have detrimental effect on the imposed seismic demand. Drosos et al. (Drosos et al. 2012) experimentally investigated the nonlinear response of asurface foundation on sand and its effect on the seismic performance of an idealized slender single-degree-of-freedom structure. Kim et al. (Kim and Roesset 2004) studied the influence of the soil non-linear model on the superstructure by using the 3D finite element analysis method. It is found that the parameters of the soil constitutive model have agreater influence on the DSSI effect. (Figini, Paolucci, and Chatzigogos 2012) introduced anovel formulation of anmacro-element model for non-linear DSSI analyses of structures lying on shallow foundations. This macro-element allowed one to model soil-footing geometric (uplift) and material (soil plasticity) non-linearities that were coupled through astiffness degradation model. Gazetas et al. (Gazetas et al. 2013) studied the response of surface foundations to large overturning moments under undrained conditions. Pecker et al. (Pecker et al. 2014) provided an overview of recent research work that contributed to clarify the effects of non-linear dynamic interaction on the seismic response of soil-foundation-superstructure systems. Gazetas (Gazetas 2015) proposed anew dynamic approach in performance-based seismic design of soil–foundation–structure systems. It was shown that system collapse could be avoided even under seismic shaking far beyond the design ground motion. Abate et al. (Abate and Massimino 2016) investigated coupled dynamic soil-foundation-superstructure interaction by means of al-g shaking table test. Abate et al. (Abate et al. 2017) provided the results of FEM simulation of dynamic tests recently performed in Thessaloniki on large-scale single-degree-of-freedom structure resting on asoft soil. Numerical and experimental results were extensively compared, and very interesting results were reached above all in terms of the effects of soil-foundation interface behaviour. Massimino et al. (Massimino et al. 2019a) performed FEM modelling for the soil-foundation-superstructure systems and investigated the dynamic response of fully-coupled, soil-structure systems in structures with shallow foundations in the time and frequency domains. Massimino et al. (Massimino et al. 2019b) investigated the dynamic behaviour of accoupled soil-structure system, i.e., aschool building in Catania, characterized by ahigh seismic hazard. Gavras et al. (Gavras et al. 2020) established anew publicly available
database, “FoRDy” (Foundation Rocking– Dynamic), which summarized the results of dynamic physical model tests of single-degree-of-freedom-like structures supported on rocking foundations and contained data from five centrifuge and three 1-g shaking table test series.

Although the existing literature has analyzed the dynamic response of the DSSI system and obtained some laws, but there is a lack of systematic research on the in-deep internal laws of the dynamic response of the DSSI system and the influence of different parameters on DSSI.

In this paper, based on the shaking table model tests, the 3D numerical models of the shaking table model tests of the DSSI system are established using ABAQUS software. The simulation results are compared with the shaking table test data to verify the correctness of the numerical models. The main parameters of the numerical models are adjusted to establish the parametric analysis models for different parameters. The in-deep internal laws of the system dynamic response and the influence of different parameters on DSSI are investigated systematically by the parametric analysis models.

2. Brief description of shaking table model tests

A series of shaking table tests were carried out by Lu et al. (Lu et al. 2003). The focus of this paper is mainly on one of shaking table tests, labeled BS10 hereafter.

The prototype system can be considered as atypical small high-rise building system of Shanghai. The superstructure was a 12-story RC frame structure with a single span that had a foundation RC frame structure with a single span that had a foundation embedded in the soil. The model’s scale was 1/10, which ignored the effect of the gravity similarity model (IM for short). In the test, the layout and reinforcement detail of the BS10 test model is shown in Figure 1, and the geological parameters of each model soil layer are shown

Figure 1. The layout and reinforcement detail of the BS10 test model (unit:mm).
in Table 1. A flexible cylindrical container was used to reduce the box effect.

Accelerometers and strain gauges were used to measure the dynamic response of the superstructure, foundation, and soil. Soil pressure gauges were used to measure the contact pressure between foundation and soil. The unidirectional (X direction of shaking table) excitation was employed in the test, including the records of the El Centro wave and Kobe wave (in 1995), labeled EL and KO hereafter.

The working condition of the BS10 test, labeled ELX1 hereafter, is used to verify the calculation model.

3. Modeling method

3.1. Simulation of the flexible soil box

The shell element was used to mesh the lateral rubber membrane of the flexible soil box in modeling. The relative slip between the soil and the bottom of the soil box is ignored, and the bottom of the soil is approximately regarded as fixed end. The nodes along the circumference of the cylinder with the same height have the same displacement in the ground motion input direction (shaking table X or Y direction), which is realized by the coupling constraint in ABAQUS software. The lateral rubber membrane of the soil box and the surrounding soil are meshed by the common node method to meet the coordination of displacement and force equilibrium. A rigid plate with a thickness of 0.15 m is added at the bottom of the flexible container model to apply the seismic load to simulate the shaking table panel.

3.2. Simulation of material nonlinearity

In this paper, the Drucker-Prager model (D-P model for short) is adopted for the soil. The D-P model is an improvement of the Mohr-Coulomb model, whose flow criterion cannot be changed with the gradual yield of the materials, and the yield strength increases correspondingly with the increase of the confining pressure (hydrostatic pressure). The D-P model considers the volume expansion caused by the material yielding, but does not consider the effect of the temperature change. When the material reaches the yield strength, the stress no longer increases, but the strain keeps increasing all the time, which can accurately reflect the basic characteristics of the soil as friction material. The D-P model parameters are shown in Table 1. The material nonlinearity of the concrete structure is simulated by the Plastic-damage model.

| Soil                  | Thickness (m) | Density (Kg/m³) | Elasticity modulus (MPa) | Poisson's ratio | Cohesion (kPa) | Shear strength angle φ (°) |
|-----------------------|---------------|-----------------|--------------------------|-----------------|----------------|-----------------------------|
| Silty clay            | 0.25          | 1845            | 8                        | 0.35            | 25             | 10                          |
| Sandy silt            | 1.00          | 1800            | 12                       | 0.3             | 15             | 20                          |
| Medium sand           | 0.35          | 2005            | 14                       | 0.25            | 3              | 30                          |

3.3. Simulation of the contact nonlinearity on soil-structure interfaces

In this paper, the contact elements in software ABAQUS are used to simulate the state nonlinearity of the contact surfaces between the foundation and the surrounding soil. The soil surface is regarded as the master surface, and the structure (foundation) surface with greater stiffness as the slave surface. The corresponding master and slave surfaces are defined as a contact pair, in which hard contact is considered as the normal constitutive model, penalty function as the tangential constitutive model. In this simulation, the Coulomb friction coefficient is tan (0.75 φ) used by Fei et al. (Fei and Zhang 2010), and the parameter φ is the shear strength angle of the soil in Table 1.

3.4. Treatment of reinforcement in RC structure

In this paper, the concrete structure is modeled by 3D solid element, it is assumed that the reinforcement is distributed in the whole structural element, and the element is regarded as continuous and uniform material. The contribution of the reinforcement to the whole structure is realized by the equivalent principle of the bending stiffness EI,

\[ EI = E_I I_x + E_I I_y \]  

in which \( E_I \) and \( I_x \) are the equivalent elastic modulus and the section moment of inertia respectively; the subscripts con and r represent the concrete and the reinforcement respectively.

3.5. Damping model

In the DSSI system, soil and concrete belong to different materials, and the damping ratio of the soil is generally greater than that of the concrete structure. The damping ratio of the soil is related to the level of the shear strain under the dynamic load, the dynamic triaxial test results of the soils in Table 1 show that when the level of the shear strain of the soils is \( 10^{-4} \), the damping ratio of various soils is about 0.1. In this paper, the damping ratios \( \xi \) of the soil and concrete
structure are taken as 0.1 and 0.05, respectively. It is assumed that the first two modes of each material have the same damping ratio. In this simulation, Rayleigh damping is applied in ABAQUS, the Rayleigh damping coefficients $\alpha_i$ and $\beta_i$ of each material can be calculated by the following formula:

$$\alpha_i = \frac{2\omega_1\omega_2}{\omega_1 + \omega_2} \zeta_i$$

$$\beta_i = \frac{2}{\omega_1 + \omega_2} \zeta_i$$

in which $\omega_1$ and $\omega_2$ are the first and second-order natural frequencies of the DSSI system, which can be obtained by modal analysis of the software ABAQUS.

3.6. Consideration of gravity

Before the seismic analysis is carried out, it is important to balance the initial stress field caused by the gravity load. The method is as follows:

1. The gravity load is applied to the soil and the boundary conditions in line with the engineering practice are exerted.

4. The stress field under the gravity load is obtained and defined as the initial stress field

3. The initial stress field is applied to the finite element model together with the gravity load to ensure that the initial displacement of each node is approximately zero.

4. In this paper, the initial displacement of each node has reached $10^{-7}$ m, thus, the balance of initial stress is achieved in the numerical models.

4.1. Dealing with uncoordinated degrees of freedom (DOF)

The soil is modeled by 3D solid element, while the lateral rubber membrane of the flexible soil box is simulated by 3D shell element. The 3D shell element at each node have six DOF: three translational DOF and three rotational DOF, while the solid element only have three translational DOF. Hence, the problem of uncoordinated DOF occurs at the boundary of the two elements.

In this study, the method of relaxing the DOF is used to deal with the problem of uncoordinated DOF. In other words, at the interface between 3D shell element and 3D solid element, the three translational DOF of the common nodes at the same location of shell and solid elements are constrained to be the same, and the three rotational DOF are relaxed without any constraints. Since the rotational DOF only correspond to every small part of the kinetic energy of the structure, the influence of this method on the structure is very small.

4.2. Meshing

When meshing the DSSI system, the following three principles should be considered.

1. If the element size is too large, the high-frequency component of the ground motion is difficult to be transmitted.

2. The nodes of the finite element models should correspond to the locations of the measuring points of the tests so that the simulation results can be compared with the experimental results.

3. The finer meshing, the more DOF, and the higher the accuracy, but the longer the calculation time.

4. Efficient reproduction of all the waveforms of the whole frequency range under study (e.g., following the principle that the element size must be 1/6–1/8 of the minimum wavelength).

5. Efficient modelling simulation of the soil close to the structure (therefore, a finer discretization beneath the foundation has been selected).

Therefore, the proper element size should be applied to obtain satisfactory calculation accuracy at an appropriate cost. The meshing of the BS10 test model that meets the above modeling principles is shown in Figure 2.

4.3. Verification of the finite element model

The acceleration time-history curves comparing the simulated data with the measured data of the BS10 test are shown in Figure 3, in which Point A7 (see Figure 1) is at the top of the superstructure, and the soil surface Point S8 (see Figure 1) is 0.9m away from the soil box center.

As can be seen from the Figure 3, the numerical results are in good agreement with the experimental data. It is then verified that the modeling method is rational, and the finite element model is suitable for the numerical analysis of the DSSI system.

(The BS10 test model, Under the excitation of ELX1).

5. Parametric analysis model

Since the design of the BS10 test only uses the ignoring gravity similarity model, and the anti-seismic fortification level of the structure is not high, it cannot meet the analysis requirements of the additional weight of the structure and large input ground motion acceleration peak. Therefore, in order to analyze the influence of the buried depth of the foundation, the stiffness and the additional weight of the structure and the type and amplitude of the input ground motion on DSSI, on the basis of adjusting the design parameters of the BS10 test, three similar models of the ignoring
of the parametric analysis model is shown in Figure 5, which is the basis of other models. Compared with Figure 1, the structure in Figure 4 has improved reinforcement and concrete strength, and the anti-seismic fortification level of the structure has been increased to 0.4 g.

The length $l$, the elastic modulus $E$, and the mass density $\rho$ are taken as the basic quantities, all physical quantities are scaled using similitude formulas from the Buckingham $\pi$ theorem. The similitude factors of the parametric analysis models are listed in Table 2. The same similitude relation is applied to soil, foundation and superstructure.

When studying the influence of different parameters on DSSI, different working conditions are set: Three working conditions are set for the additional weight of the superstructure, which are the no-load (IM), half-load (UM), and full-load (AM), it means the additional weight is zero mass, half the mass of the superstructure, the total mass of the superstructure, respectively. For the buried depth of the foundation, three working conditions are the no-depth (SSND), half-depth (SSHD), and full-depth (SSFD), which means the buried depth of the foundation is equal to zero, 0.2m and 0.4m, respectively. For the stiffness of the superstructure, the working conditions are the C30, C40, and C50, which are the concrete strength level. Due to the difference in spectral characteristics, the original records of the El Centro wave, Kobe wave, Taft wave, and Tianjin wave are selected for the input ground motions, labeled EL, KO, TA, and TJ hereafter, whose acceleration time-history and corresponding Fourier spectrum are shown in Figure 6. The acceleration peak and time interval of the input ground motions are adjusted by the similitude relation in Table 2, the working conditions for the type and amplitude of the input ground motions, called the test loading schedules, are shown in Table 3, which consists of four excitation levels that are labeled form EL1 to EL4, KO1 to KO4, TA1 to TA4, and TJ1 to TJ4. To study the softer foundation soil, the parameter analysis
6. Simulation results

Based on the parametric analysis models in the previous section, some important simulation results are discussed in this section.

6.1. Contact and separation on the bottom surface of the foundation

The contact pressure time-history and the separation displacement time-history of different points on the foundation bottom surface of the parametric analysis models subjected to all kinds of input ground motions are analyzed. It can be found that the situations of the zero-contact pressure and the positive separation displacement occur in each working condition. That is to say, the foundation and surrounding soil are separated at the bottom of the foundation. The contact pressure time-history and the separation displacement time-history of the endpoint along the X-axis (see Figure 7) on the foundation bottom subjected to the El Centro wave with different acceleration peaks are shown in.

As can be seen from Figure 8, for the same parametric model and the same waveform, as the acceleration peak of the input ground motion increases, the contact pressure and the separation displacement of the endpoint along the X-axis on the foundation bottom increase continuously, and the zero-contact pressure case and the positive separation displacement case also increase. The peak contact pressure and the zero-contact pressure case increase 86.65% and 99.48% respectively under EL2 excitation compared with these under EL1 excitation, and 59.48% and 64.83% respectively under EL3 excitation compared with under EL2 excitation. This is mainly because with the increase of the peak acceleration, the soil shows strong non-linearity. Under the excitation of four input ground motions with the same acceleration peak, the contact pressure of the endpoint along the X-axis on the foundation bottom is sequentially from large to small: Tianjin wave, Kobe wave, El Centro wave, and Taft wave, which are mainly related to the spectral characteristics. It is shown that the contact pressure of the endpoint along the X-axis on the foundation bottom is closely related to the intensity and spectral characteristics of the input ground motion. It can also be seen from Figure 8 that the position of the positive separation displacement and the position of the zero-contact pressure correspond to each other.

The contact pressure time-history of the endpoint along the X-axis on the foundation bottom of the parametric models with different buried depth of the foundation under the excitation of the Taft wave is shown in Figure 9(a), whereas the contact pressure time-history of the parametric analysis models with
Figure 5. The layout and reinforcement detail of the parametric analysis model (unit:mm).

Table 2. Dynamic similitude relation of the parametric analysis models.

| Type                     | Physical Quantity | Similitude Formula | IM     | UM     | AM     | Remark             |
|--------------------------|-------------------|--------------------|--------|--------|--------|--------------------|
| Material Parameter       | Stress $\sigma$   | $S_\sigma = S_0\sigma$ | 1/5    | 1/5    | 1/5    |                    |
|                          | Strain $\epsilon$ | $S_\epsilon = S_0\epsilon$ | 1.0    | 1.0    | 1.0    |                    |
|                          | Young’s Modulus $E$ | $S_E = S_0E$ | 1/5    | 1/5    | 1/5    | CP                 |
|                          | Poisson’s ratio $\mu$ | $S_\mu = S_0\mu$ | 1.0    | 1.0    | 1.0    |                    |
|                          | Density $\rho$    | $S_\rho = S_0\rho$ | 1.0    | 1.5    | 2.0    | CP                 |
| Geometrical Parameter    | Length $l$        | $S_l = S_0l$ | 1/10   | 1/10   | 1/10   | CP                 |
|                          | Area $A$          | $S_A = S_0A$ | 1/100  | 1/100  | 1/100  |                    |
|                          | Linear displacement $X$ | $S_X = S_0X$ | 1/10   | 1/10   | 1/10   |                    |
|                          | Angular displacement $\beta$ | $S_\beta = S_0\beta$ | 1.0    | 1.0    | 1.0    |                    |
| Load Parameter           | Concentrated force $F$ | $S_F = S_0F$ | 1/500  | 1/500  | 1/500  |                    |
|                          | Linear load $p$   | $S_p = S_0p$ | 1/50   | 1/50   | 1/50   |                    |
|                          | Area load $q$     | $S_q = S_0q$ | 1/5    | 1/5    | 1/5    |                    |
|                          | Moment $M$        | $S_M = S_0M$ | 1/5000 | 1/5000 | 1/5000 |                    |
| Dynamic Parameter        | Mass $m$          | $S_m = S_0m$ | 1/1000 | 3/2000 | 1/500  |                    |
|                          | Rigidity $k$      | $S_k = S_0k$ | 1/50   | 1/50   | 1/50   |                    |
|                          | Damping $c$       | $S_c = S_0c$ | 0.00447| 0.00548| 0.00632|                    |
|                          | Time $t$          | $S_t = (S_m/S_0)^{0.5}$ | 0.224  | 0.274  | 0.316  | CPDL               |
|                          | Frequency $f$     | $S_f = S_0f$ | 4.472  | 3.651  | 3.162  | CPDL               |
|                          | Velocity $v$      | $S_v = S_0v$ | 0.447  | 0.365  | 0.316  | CPDL               |
|                          | Acceleration $a$  | $S_a = S_0a$ | 2.0    | 4/3    | 1.0    | CPDL               |

CP-Control parameter of model design; CPDL-Control parameter of dynamic load

Different stiffness of the superstructure under the excitation of the Kobe wave is provided in Figure 9(b).

In Figure 9(a), for different buried depth parametric models under the same ground motion excitation, as
the burial depth of the model foundation decreases, the contact pressure of the endpoint along the X-axis on the foundation bottom continuously increases, and the zero-contact pressure case continuously decreases. The peak contact pressure and the zero-contact pressure case of SSND are 30.69% and −95.42% higher than those of SSFD, respectively. This is because the smaller the buried depth of the foundation, the smaller the self-weight stress of the soil at the bottom of the foundation, and the greater the contact pressure at the bottom of the foundation under the same conditions.

As can be seen from Figure 9(b), for different stiffness parametric models under the same ground motion excitation, as the stiffness of the superstructure increases, the contact pressure of the endpoint along the X-axis on the foundation bottom and the zero-contact pressure case continuously increase. The reason is that with the increase of structural stiffness, the deformation of the structure decreases and the displacement coordination of the contact surface becomes worse.

The contact pressure time-history of the endpoint along the X-axis on the foundation bottom of the parametric models with the different additional weight of the superstructure under the excitation of the El Centro wave is shown in Figure 10. It is shown that with the increase of the additional weight of the superstructure, the displacement coordination of the contact surface becomes better, the contact pressure increases, and the zero-contact pressure case decreases continuously.

![Figure 6. The acceleration time-history and Fourier spectrum of the four input ground motions.](image)

**Table 3.** Test loading schedules.

| Working conditions No. | Excitation code | Prototype Peak acceleration (g) |
|------------------------|-----------------|---------------------------------|
| 1,2,3,4                | EL1, KO1, TA1, TJ1 | 0.1 0.2 0.1333 0.1 |
| 5,6,7,8                | EL2, KO2, TA2, TJ2 | 0.2 0.4 0.2667 0.2 |
| 9,10,11,12             | EL3, KO3, TA3, TJ3 | 0.4 0.8 0.5333 0.4 |
| 13,14,15,16            | EL4, KO4, TA4, TJ4 | 0.62 1.24 0.8267 0.62 |

**Table 4.** The adjusted soil geological parameters.

| No. | Thickness (m) | Density (Kg/m³) | Elasticity modulus E (MPa) | Poisson’s ratio μ | Cohesion c (kPa) | Shear strength angle φ (°) | Shear wave velocity (m/s) |
|-----|---------------|-----------------|---------------------------|-----------------|-----------------|--------------------------|--------------------------|
| 1   | 0.2           | 1790            | 8.76                      | 0.33            | 4.8             | 13.2                     | 95.91                    |
| 2   | 1.1           | 1835            | 9.6                       | 0.4             | 4.85            | 14.5                     | 96.66                    |
| 3   | 0.3           | 1940            | 10.4                      | 0.35            | 3.8             | 17                       | 130                      |
Table 5. Letter representation of all working conditions.

| The buried depth | The stiffness | The additional weight | The type and amplitude | Combination example |
|------------------|--------------|-----------------------|------------------------|--------------------|
| SSFD             | C30          | IM                    | EL1, EL2, EL3, EL4     | SSFD-C30-IM-EL1-EL2-EL3 |
| SSHD             | C40          | UM                    | K01, K02, K03, K04    | SSHD-C40-UM-K01-K02-K03-K04 |
| SSD              | C50          | AM                    | TA1, TA2, TA3, TA4    | SSD-C50-AM-TA1-TA2-TA3-TA4 |

The distribution of the peak contact pressure and the peak separation displacement along the middle line on the bottom of the foundation under the excitation of the El Centro wave with different acceleration peaks are shown in Figure 11(a,b).

As can be seen from Figure 11, the approximately symmetrical distribution of the peak contact pressure along the middle line on the bottom of the foundation is large on two both sides, small in the middle, and almost constant on a large range of the middle portion. The distribution of the peak separation displacement along the middle line on the bottom of the foundation is "M" shape. As the acceleration peak of the input ground motion increases, the peak contact pressure and the peak separation displacement along the middle line on the bottom of the foundation increase continuously.

![Foundation bottom](image)

Figure 7. The location of the endpoint along the X-axis.

6.2. Peak acceleration Magnification factor

The nodes used to analyze the magnification factor of acceleration peak are taken from the central axis of the DSSI system except the soil within the height of the foundation. The distribution of the peak acceleration magnification factors of the different parametric analysis models along the central axis of the DSSI system is shown in Figure 12. In the figures, the peak acceleration of each story is shown compared to that of the soil box bottom. From these figures, some phenomena can be observed.

1. In general, for soil, the peak acceleration amplification factors, which is greater than 1, increase with increasing distance to the bottom for low-frequency ground motion, and the soft soil plays an important role in amplifying the seismic effect. When the input ground motion frequency component is high, the peak acceleration magnification factors, which are less than 1, decrease first and then increase with increasing distance to the bottom. The bottom soil plays an important role in reducing the seismic effect, and the middle and top soil play an important role in amplifying the seismic effect.

2. For the structure, due to the recombination of the foundation rotation, rocking, and multi-mode response, the magnification factor of each floor is significantly different. The peak acceleration magnification factor decreases first and then increases with increasing distance to the bottom, and the minimum is not at the bottom of the structure, but in the middle floor.

3. In Figure 12(a), for the same parametric model and the same waveform, as the acceleration peak of the input ground motion increases, the peak acceleration magnification factors decrease accordingly. The peak acceleration magnification factor at the top of the structure decreases 22.13% under TA2 excitation compared with these under TA1 excitation, and 29.78% under TA3 excitation compared with under TA2 excitation. This is because with increasing
excitation, the stiffness of soft soil and the ability of soil to transmit waves decrease, and soil nonlinearity increases.

4. In Figure 12(b), for different buried depth parametric models under the same ground motion excitation, as the buried depth of the foundation decreases, the peak acceleration magnification factors decrease accordingly. The peak acceleration magnification factor at the top of the SSND structure is 32.66% less than that of SSFD. This is because with the decrease of the buried depth of the foundation, the vibration energy received by the superstructure from the soil decreases, and the acceleration response also decreases.

5. In Figure 12(c,d), for different additional weight and stiffness parametric models under the same ground motion excitation, as the additional weight and the stiffness of the structure increase, the peak acceleration magnification factors increase accordingly.

Figure 9. The contact pressure time-history of the endpoint along the X-axis on the foundation bottom: (a) different buried depth of the foundation; (b) different stiffness of the superstructure.

Figure 10. The contact pressure time-history of the endpoint along the X-axis on the foundation bottom of the parametric models with the different additional weight of the superstructure.
6.3. Distribution of peak inter-story drift

The nodes used to analyze the distribution of peak inter-story drift in the superstructure are taken from the central axis of the superstructure. The distribution curves between the peak inter-story drift in the superstructure and the height of each floor of the parametric analysis models are shown in Figure 13. Some laws can be obtained from the following Figures 1. The distribution of peak inter-story drift in the superstructure is minimum in the top floor, and larger in the middle floors, especially the second floor, which reaches maximum.

2. In Figure 13(a), for the same parametric model and the same waveform, with the increase of the acceleration peak of the input ground motion, the peak inter-story drift in the superstructure increases continuously. The peak inter-story drift at the top of the structure increases 161.48% under TJ2 excitation compared with these under TJ1 excitation, and 116.01% under TJ3 excitation compared with under TJ2 excitation. Under the excitation of the ground motion with the same acceleration peak, the distribution of peak inter-story drift in the superstructure is mainly related to the spectral characteristics of the ground motion.

3. In Figure 13(b), for different buried depth parametric models under the same ground motion excitation, with the decrease of the buried depth of the foundation, the peak inter-story drift in the superstructure increases continuously. The peak inter-story drift at the top of the SSND structure is 45.24% higher than that of SSFD.

4. In Figure 13(c), for different additional weight parametric models under the same ground motion excitation, with the increase of the additional weight of the superstructure, the peak inter-story drift in the superstructure decrease continuously.

5. In Figure 13(d), for different stiffness parametric models under the same ground motion excitation, with the increase of the stiffness of the superstructure, the peak inter-story drift in superstructure increase continuously. Since the three curves almost coincide, it can be shown that the stiffness has little effect on the distribution of peak inter-story drift in the superstructure.

6.4. Components of the total acceleration at the top of the superstructure

The components of the total acceleration at the top of the superstructure are shown in Figure 14 and Equation (5).

\[ \ddot{u}_e = \ddot{u} - \ddot{u}_g - H\dot{\theta} \]  
(5)

where is the total acceleration at the top of the superstructure, \( \ddot{u}_g \) and are the translational component, the rocking component, and the elastic deformation component of the total roof acceleration, respectively, \( uu\theta \left( R + \frac{R}{L} \right) \).

The time-history and corresponding Fourier spectrum of each component of the total acceleration at the top of the superstructure of the parametric analysis models are shown in Figures 15-18, from which some laws can be obtained.

1. The total acceleration at the top of the superstructure is mainly composed of the rocking component, followed by the elastic deformation component, and the translational component is the smallest. The reason is that soft soil increases the motion of the foundation.

2. The three components are out of sync with each other; they may be in phase or out of phase, overlapping or offset from each other.

3. In Figure 15 With the increase of the acceleration peak of the input ground motion, the time-history and corresponding Fourier spectrum of each component of the total roof acceleration increases, and the spectrum composition of each component shift to low frequency, especially the rotation component and
Figure 12. Distribution of peak acceleration magnification factors: (a) different acceleration peaks; (b) different buried depth of the foundation; (c) different additional weight of the superstructure; (d) different stiffness of the superstructure.

translational component shift more significantly. The peak total acceleration, the rocking component, the translational component, and the elastic deformation component increase 55.75%, 174.37%, 98.36% and 232.08% respectively under EL2 excitation compared with these under EL1 excitation, and 40.44%, 54.74%, 64.65% and 55.03% respectively under EL3 excitation compared with under EL2 excitation.

4. In Figure 16, as the buried depth of the foundation decreases, the time-history of each component of the total roof acceleration decreases continuously. The peak total acceleration, the rocking component, the translational component, and the elastic deformation component of SSND are 30.86%, 56.14%, 11.40% and 86.13% respectively less than those of SSFD. This is mainly because with the decrease of the buried depth of the foundation, the energy of the ground motion transmitted by the soil decreases. The Fourier spectrum of the total roof acceleration, translational component, rotational component, and elastic deformation component decrease in the frequency range of 4 Hz to 14 Hz, 4 Hz to 10.4 Hz, 4 Hz to 12.2 Hz and greater than 4 Hz, respectively, and increase in other frequency range. This is related to the selective magnification of the ground motion frequency components by the soil.

5. In Figure 17, as the additional weight of the superstructure increases, the time-history and corresponding Fourier spectrum of each component of the total roof acceleration decreases, the Fourier spectrum shift to the low frequency, which corresponds to the decrease of the resonance frequency of the DSSI system with the increase of the additional weight.

6. In Figure 18, With the increase of the stiffness of the superstructure, the time-history and corresponding Fourier spectrum of the total roof acceleration and the translational component increase continuously, whereas that of the rocking component and the elastic deformation component decrease continuously.

7. Conclusions

The following are some conclusions drawn from this study, which can provide a reference for future engineering designs:

1. The factors affecting the contact pressure caused by different parameters are different. In general, the contact pressure and the zero-contact pressure case of the contact surfaces between the foundation and the surrounding soil are closely related to the strong non-linearity of the soil, the self-weight stress of the soil at the bottom of the foundation, the deformation of the structure and displacement coordination of the contact surfaces.
2. The approximately symmetrical distribution of the peak contact pressure along the middle line on the bottom of the foundation is large on two both sides, small in the middle, and almost constant on a large range of the middle portion. The distribution of the peak separation displacement along the middle line on the bottom of the foundation is “M” shape. The position of the minimum peak contact pressure corresponds to the maximum peak separation displacement.

3. Under the excitation of four input ground motions with the same acceleration peak, the contact pressure on the foundation bottom is sequentially from large to small: Tianjin wave, Kobe wave, El Centro wave, and Taft wave. The result shows that the contact pressure of the endpoint along the X-axis on the foundation bottom and the distribution of the peak contact pressure along the middle line on the bottom of the foundation are closely related to the intensity and spectral characteristics of the input ground motion.

4. The peak acceleration magnification factors and the role of the soil are in connection with the spectral components of the input ground motion. For low-frequency ground motion, the peak acceleration magnification factors are greater than 1 and the soil plays a role in amplifying the seismic effect; for high-frequency ground motion, the peak acceleration magnification factors are less than 1 and the foundation plays a role in dampening the seismic effect.

Figure 13. Distribution of peak inter-story drift: (a) different acceleration peaks; (b) different buried depth of the foundation; (c) different additional weight of the superstructure; (d) different stiffness of the superstructure.

Figure 14. The components of the total acceleration.
Figure 15. The time-history and corresponding Fourier spectrum of each component of the total roof acceleration (SSFD-C40-IM parameter model, Excitation EL).

Figure 16. The time-history and corresponding Fourier spectrum of each component of the total roof acceleration (SSFD/SSHHD/SSND-C40-IM parameter models; Excitation TA1). (a) SSFD-C40-IM parameter model, Excitation EL1. (b) SSFD-C40-UM parameter model, Excitation EL1. (c) SSFD-C40-AM parameter model, Excitation EL1.
Figure 17. The time-history and corresponding Fourier spectrum of each component of the total roof acceleration.
Figure 17. The time-history and corresponding Fourier spectrum of each component of the total roof acceleration (SSFD-C30/C40/C50-IM parameter models, Excitation K01).

Figure 18. The time-history and corresponding Fourier spectrum of each component of the total roof acceleration (SSFD-C30/C40/C50-IM parameter models, Excitation K01).
magnification factors are less than 1 and the bottom soil plays a role in reducing the seismic effect, and the middle and top soil play a role in amplifying the seismic effect.

5. As the acceleration peak of the input ground motion increases, the buried depth of the foundation decreases, the additional weight of the superstructure decreases, and the stiffness of the superstructure increases, the peak inter-story drift in superstructure increase continuously. With the decrease of the buried depth of the foundation, the peak inter-story drift at the top of the no-buried depth structure is 45.24% higher than that of the full-buried depth structure.

6. The total acceleration at the top of the superstructure is mainly composed of the rocking component, followed by the elastic deformation component, and the translational component is the smallest.

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No potential conflict of interest was reported by the author(s).

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