Substructure shake table test for equipment-adjacent structure–soil interaction based on the branch mode method

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Funding information
National Natural Science Foundation of China, Grant/Award Numbers: 51478312 and 51278335; Natural Science for Youth Foundation, Grant/Award Number: 51208356

Summary
A substructure shake table test (SSTT) based on the branch mode method was performed to reveal the mechanism and rules of equipment-adjacent structure–soil interaction (EASSI) under a seismic effect. EASSI system was divided into three substructures, namely, equipment-single structure, foundation soil, and adjacent structure. The coupling terms of interaction among the substructures were proposed. The branch mode method was effectively applied to the SSTT by decomposing and transforming the dynamic equation of the entire system and utilizing the coupling terms of interaction for data exchange among substructures. The degree of freedom was reduced for the linear substructures. Experiments indicated that in EASSI, the presence of soil magnified the flexibility and equivalent damping of the entire system. The overall effect was presented as a reduction in the dynamic response of the system. The dynamic feedback of the equipment inhibited the dynamic response of the main structure, which intensified the rate of vibration attenuation of the system. The seismic response analysis was also performed for the system when the mass ratio and frequency ratio between the equipment and the main structure and the position of the equipment in the main structure varied.

KEYWORDS
branch mode method, equipment-adjacent structure–soil interaction (EASSI), seismic response, shake table, soil–structure interaction, substructure test

1 | INTRODUCTION

Along with the rapid social and economic development, the costs of equipment add to total costs of construction. A seismic action may cause great damage to the equipment, leading to a growing concern on antiseismic properties of equipment and pipelines. These equipments and structures are considered as a system engaged in dynamic interaction. The methods for analyzing the dynamic interactions between the equipment and the structure include the theoretical and experimental approaches. The floor response spectrum is a typical theoretical approach. In the conventional floor response spectrum, forced decoupling is performed, with the response of the main structure as the seismic input of the equipment. However, the tuning effect and interactions between the equipment and the main structure, as well as nonclassical damping and space coupling, are neglected when computing the floor response spectrum. As a result, the computational result is relatively conservative and does not satisfy the economical requirements. This method was applied to determine the design peak acceleration of the equipment located in the nonelastic structure.[1] The modified floor response spectrum considers the dynamic features of equipment and main structure as well as the interactions between...
the two. Therefore, the dynamic response of the system calculated on the basis of floor response spectrum is more accurate. A modified approach was proposed on the basis of the nonclassical damping characteristics of the equipment–structure system. The combinatorial formula of response spectrum was proposed for estimating the dynamic response of structure with nonclassical damping based on the modal characteristics of each substructure. This effectively avoided the solving of complex eigenvalues. The stochastic computing of large nonproportionally damped structure was studied and a new method for calculating transfer function was proposed. Among various experimental studies, a shake table test for a prototype of five-layer frame structure was performed, which was equipped with a large amount of nonstructural components such as stairs, elevators, and pipelines. The seismic response of structural and nonstructural components was under fixed support and insulation support, respectively. Another shake table test for a two-layer transformer station with electrical installations was performed and the effect of equipment–structure interactions on the seismic response of the system was analyzed.

All the aforementioned studies were conducted under the assumption of a rigid foundation, but without considering the effect of foundation soil on the equipment–structure system. As a result, the computational results usually deviated from the real situation, or even to the extent of being unsafe. In engineering practice, equipment–structure systems over soft soil base and in dense high-rise buildings will not only be engaged in dynamic interactions with the foundation soil, but the adjacent buildings will affect the equipment–structure systems via the foundation soil. Therefore, equipment, main structure, foundation soil, and adjacent structures should be taken as a whole when assessing the seismic response of the equipment–structure system. This is known as the equipment-adjacent structure–soil interaction (EASSI) problem.

At present, researchers have made a series of valuable conclusions and achievements in theory and experiment considering the soil–structure interaction (SSI) or soil–adjacent structure interaction (SASI) problems under the action of earthquake. In experimental study, the shake table test is a preferred choice for the antiseismic analysis of the SSI (SSI) problem. The soil box method is the most commonly used for considering SSI in the shake table test. In this method, a soil box containing a soil mass is immobilized to the shake table, and the upper structure model is placed on the top of the soil mass. The seismic action is input via the shake table. However, the soil box method is subjected to a variety of restricting factors. The bearing capacity of the shake table is limited, and the reflection and scattering of seismic waves by the foundation soil in the soil box under the artificial boundary conditions may diverge from the real situations.

The substructure shake table test (SSTT) offers a new approach to the seismic response of complex structural systems. It represents a major development trend in antiseismic experiments for large complex structural systems. A construction is divided into several substructures, with the lower, middle, or upper substructures as physical substructures. Loading is performed using actuators, shake table, or both. The remaining substructures are taken as numerical substructures for seeking numerical solutions with the aid of a computer. Thus, the dynamic response of the entire structure is analyzed through real-time decoupling. A substructure shake table test for a double-layer steel frame that considered SSI was performed, and the foundation soil was taken as a numerical substructure. However, the foundation soil was oversimplified into two degrees of freedom, and the effect of foundation oscillation on the upper structure was not considered. In the aforementioned experiments, the interfacial force used for data exchange between the physical substructure and numerical substructure was generally horizontal shear. However, in fact, the experiment will more resemble the real engineering scenario by considering the effect of both translational motion and oscillation of the foundation on the upper structure to understand the SSI or EASSI problem. Two major challenges need to be tackled when applying SSTT to the SSI or EASSI problem. First, if the foundation soil is taken as the physical substructure, foundation soil over only a limited range can be simulated because of limited bearing capacity and specification of the shake table. This will amplify the influence of size effect on the experimental results. Second, if the foundation soil is taken as the numerical substructure, real-time computing or data exchange may be impossible because of large numbers of the degree of freedom. This is because the loading time step is only a few milliseconds under SSTT. Therefore, how to reduce the computational load of numerical substructures to achieve high-efficiency computation and how to make the reasonable simplification of the problem are the major concerns with SSTT.

The branch mode method is a dynamic substructure method that proves effective for reducing the degree of freedom and applies to the analysis and solving of dynamic response in linear structures. It can effectively reduce the degree of freedom of the structures as well as the computational load. The branch mode method in this study was applied to SSTT that considered EASSI and proposed the coupling term for the interaction. This coupling term was applied to data exchange and transmission between the physical substructure and numerical substructure, to pave the way for incorporating EASSI into SSTT for the antiseismic research. In the branch mode method, equipment, main structure, foundation soil, and adjacent structures were considered as a complex system, which was divided into substructures. The dynamic equation of the entire system was decomposed and transformed. The inertial coupling term between the substructures was used to incorporate the branch mode method into SSTT. The system comprising equipment, two identical four-story steel structures, and foundation soil was studied by SSTT considering EASSI. Experimental results under different working conditions were analyzed, and the mechanism and rules of EASSI were revealed. The findings of this study provided data support and experimental validation for SSTT that considers EASSI for a complex system.

2 | APPLICATION OF BRANCH MODE METHOD

Using the principle of branch mode method, the equipment-adjacent structure–soil system was divided into three substructures, as shown in Figure 1: foundation soil substructure D, upper substructure S1E (equipment E-structure S1), and upper substructure S2. Each substructure was analyzed separately.
2.1 Analysis of substructure D

When the foundation soil conditions are good, only a small area of foundation soil close to the upper structure will undergo plastic strain in seismic response analysis; however, areas of foundation soil distant away from the upper structure will remain in the linear working state throughout the entire course of seismic excitation. In other words, the entire structural system may present local nonlinear features. For the sake of convenience, the foundation soil was simplified as elastic. For linear foundation soil substructure, the eigenvalue of substructure D was calculated from stiffness matrix \( K_D \), mass matrix \( M_D \), and damping matrix \( C_D \) of elastic foundation:

\[
|K_D| = \lambda_D |M_D| |\phi_D|.
\]

Dominant modes of the first \( m \) orders were calculated to form the modal matrix:

\[
|\phi_D| = \left[ |\phi_D|_1 |\phi_D|_2 \cdots |\phi_D|_m \right].
\]

Each characteristic matrix of substructure D was subjected to modal transformation:

\[
|K_D| = |\phi_D|_i |K_D| |\phi_D|_i; \quad |M_D| = |\phi_D|_i |M_D| |\phi_D|_i; \quad |C_D| = |\phi_D|_i |C_D| |\phi_D|_i; \quad |F_D| = |\phi_D|_i |F_D|.
\]

where damping matrix \( |C_D| \) is calculated on the basis of the general damping theory; load matrix \( |F_D| \) is the seismic load input on the bearing platform.

2.2 Analysis of substructure S1E (S2)

Substructure S1E (S2) had fewer degrees of freedom, which were not reduced to perform the elastoplastic analysis. The bearing platform was supposed to be rigid, and the acceleration of each point of the bearing platform was supposed to be identical. The deformation pattern of the upper structure is shown in Figure 2. Horizontal and swing displacements of the upper surface of bearing platform on substructure S1E (S1) caused by foundation deformation were denoted as \( u_{c1} \) and \( \phi_{c1} \) (\( u_{c2}, \phi_{c2} \)), respectively (Figure 3). The upper structure would undergo rigid body displacement due to the deformation (or displacement) of foundation soil. Rigid body mode \( R_{S1} \) was calculated for upper substructure S1E using the base point method (with the center of the upper surface of the bearing platform as the base point). Thus, the horizontal displacement at any height \( h_i \) (\( i = 1, 2, \ldots, n - 1, n \)) of rigid upper structure S1 and at the top \( h^E_j \) (\( j = 1, 2, \ldots, m - 1, m \)) of equipment \( E_j \) was given by

\[
\begin{bmatrix}
    u_{c1} \\
    u_{c1} + h^E_j \phi_{c1} \\
    u_{c1} + h \phi_{c2}
\end{bmatrix} =
\begin{bmatrix}
    1 \\
    h_j \\
    1
\end{bmatrix}
\begin{bmatrix}
    u_{c1} \\
    \phi_{c1}
\end{bmatrix},
\]

where \( h_i \) and \( h^E_j \) are any height of rigid upper structure S1 and the distance from the top of equipment \( E_j \) to the top of the foundation, respectively (\( h^E_j = h_i + h_j \), \( h_j \) is the height of equipment \( E_j \), as shown in Figure 1). Thus, the rigid body mode of upper substructure S1E was given by

![FIGURE 1  Equipment-adjacent structure-soil interaction system](image-url)
where $[R]_{s1}$ is a $(n + m) \times k$-order matrix; $n$ is the number of stores; $m$ is the number of degrees of freedom of the equipment; and $k$ is the number of degrees of freedom of foundation soil. When only a certain story in structure S1 had equipment Ej, only the terms related to equipment Ej were preserved in $[R]_{s1}$, and terms related to the structure were constant. If there was no equipment, Equation (5) deteriorated into the form consisting of only structures (Equation 6). Similarly, the rigid body mode $[R]_{s2}$ of the upper structure S2 was calculated.
2.3 Establishment of equations of motion

Modal substitution and transformation were performed, and the rigid body modes \([R]_{S1}\) and \([R]_{S2}\) of upper substructures S1E and S2 were considered. Thus, the relationship between the physical coordinates and generalized coordinates of each substructure was obtained:

\[
\{u\}_D = \{\phi\}_D \{q\}_D, \quad \{\bar{u}\}_S = \{\phi\}_s \{\phi\}_D \{q\}_D, \quad \{u\}_S = \{q\}_s + \{R\}_D \{\phi\}_D \{q\}_D. \tag{7}
\]

In Equation (7), \([u]_D\), \([\bar{u}]_S\), and \([u]_S\) are displacements of foundation soil and upper substructures S1E and S2 under the physical coordinate system, respectively; \([q]_D\), \([\phi]_s\), and \([q]_s\) are the displacements under the generalized coordinate system. From Equation (7), the overall equations of motion were calculated for nonlinear substructures S1E and S2 under the physical coordinate system and linear foundation soil substructure under the generalized coordinate system, as shown in Equation (8).

Here, \([K]_D\), \([M]_D\), \([C]_D\), and \([F]_D\) are the generalized matrices of stiffness, mass, damping, and external load of the linear foundation soil substructure under dimensionality reduction, respectively; \([R]_{S1}, [M]_{S1}, [C]_{S1}\) and \([F]_{S1}\) are the matrices of stiffness, mass, damping, and external load of the upper substructure S1E, respectively; \([K]_{S2}, [M]_{S2}, [C]_{S2}\) and \([F]_{S2}\) are the corresponding matrices of substructure S2.

\[
\begin{bmatrix}
[M]_D \{\dot{q}\}_D + \{C\}_D \{\ddot{q}\}_D + \{K\}_D \{q\}_D = \{F\}_D - \{\phi\}_D^T [R]_{S1} \{\bar{u}\}_S - \{\phi\}_D^T [R]_{S2} \{u\}_S, \\
[M]_{S1} \{\dot{\bar{u}}\}_S + [C]_{S1} \{\ddot{\bar{u}}\}_S + [K]_{S1} \{\bar{u}\}_S = \{F\}_S - [R]_{S1} \{\phi\}_D \{\bar{u}\}_D, \\
[M]_{S2} \{\dot{u}\}_S + [C]_{S2} \{\ddot{u}\}_S + [K]_{S2} \{u\}_S = \{F\}_S - [R]_{S2} \{\phi\}_D \{\bar{u}\}_D,
\end{bmatrix}
\tag{8}
\]

2.4 Modal transformation equations for the substructures and their applications in SSTT

Equation (8) was the dynamic equation of the entire EASSI system. It can be seen that the terms \([\phi]_D^T [R]_{S1} \{\bar{u}\}_S\) and \([\phi]_D^T [R]_{S2} \{u\}_S\) along the off-diagonal of mass matrix coupled the three substructures together (upper structures S1E and S2 and foundation soil D). They were known as coupling terms of interaction. For SSTT, physical substructures and numerical substructures are realized through the transmission of interfacial force. Equation (8) was decomposed into dynamic equations of foundation soil D and upper substructures S1E and S2 to incorporate it into SSTT that considered EASSI; the unknown coupling terms were transposed to the right side of the equation, to produce Equations 9, 10, and 11:

\[
[M]_D \{\dot{\bar{u}}\}_D + [C]_D \{\ddot{\bar{u}}\}_D + [K]_D \{\bar{u}\}_D = \{F\}_D - \{\phi\}_D^T [R]_{S1} \{\bar{u}\}_S - \{\phi\}_D^T [R]_{S2} \{u\}_S, \tag{9}
\]

\[
[M]_{S1} \{\dot{\bar{u}}\}_S + [C]_{S1} \{\ddot{\bar{u}}\}_S + [K]_{S1} \{\bar{u}\}_S = \{F\}_S - [R]_{S1} \{\phi\}_D \{\bar{u}\}_D, \tag{10}
\]

\[
[M]_{S2} \{\dot{\bar{u}}\}_S + [C]_{S2} \{\ddot{\bar{u}}\}_S + [K]_{S2} \{\bar{u}\}_S = \{F\}_S - [R]_{S2} \{\phi\}_D \{\bar{u}\}_D, \tag{11}
\]

where

\[
\{F\}_S = -[R]_{S1} \{\bar{u}\}_S = -[R]_{S1} \{\bar{u}\}_S, \tag{12}
\]

\[
\{F\}_S = -[R]_{S2} \{\bar{u}\}_S = -[R]_{S2} \{\bar{u}\}_S. \tag{13}
\]

\[
\{F\}_D = \{\phi\}_D^T [K]_D \{\bar{u}\}_D = -\{\phi\}_D^T [R]_{S1} \{\bar{u}\}_S - \{\phi\}_D^T [R]_{S2} \{u\}_S, \tag{14}
\]

where \(\bar{u}_S\) and \(\bar{u}_D\) are the ground accelerations of bearing platform in structure S1 and structure S2, respectively; \(\bar{u}_S\) is the traveling wave of \(\bar{u}_S\) (Figure 1), with only time lag and amplitude attenuation. That is, the two satisfied the following equation:

\[
\bar{u}_S = \gamma \bar{u}_S \left( t - \frac{d}{v} \right). \tag{15}
\]
In Equation (15), $\gamma$ is the coefficient of amplitude attenuation ($\gamma < 1$), $d$ is the distance between the two bearing platforms, and $v$ is the propagation speed of seismic waves.

In Equations 9, 10, and 11, the coupling terms of interaction appear on the right side of equations in the form of load. They were multiplied by the relevant term to obtain the coupling force of interaction. The coupling force and the seismic forces $\{F\}_{D}$, $\{F\}_{S1}$, and $\{F\}_{S2}$ acting on foundation soil $D$ and upper substructures $S1E$ and $S2$ were the external forces of the three substructures. As long as the acceleration vectors $\{\ddot{u}\}_{S1}$ and $\{\ddot{u}\}_{S2}$ for upper substructures $S1E$ and $S2$ were known, the coupling forces of interaction could be calculated, and the dynamic equation (Equation 9) of the foundation soil could be solved. The acceleration vector $\{\ddot{q}\}_{D}$ thus calculated for the foundation soil was substituted into Equations (10) and (11) for the independent loading of upper substructures $S1E$ and $S2$, respectively. This would make the SSTT quite convenient.

### 3 | EXPERIMENTAL DESIGN

#### 3.1 | Division of physical and numerical substructures

The transformed branch mode method can utilize the coupling terms of interaction and facilitate the loading of upper substructures through shake table and numerical solution of foundation soil substructure. Thus, this method was applied to the SSTT for EASSI problem in this study. The EASSI system is composed of equipment, 2 four-story shear frames and foundation soil. Figure 4 shows the division of the substructures. In SSTT, the physical substructure should be set according to the equipment conditions and the components of interest. The foundation soil was related to the bearing capacity of shake table, table size, and setting of soil boundaries, and therefore, equipment E and structure S1 were considered as physical substructure $S1E$ for the loading experiment on the shake table. Structure $S2$ and foundation soil $D$ were two numerical substructures. The seismic response of the entire system was analyzed through real-time data exchange between the substructures. Simulink software was used for the modeling of signal transmission and solving of dynamic equation. The upper substructure $S2$ was considered a numerical substructure independently to get ready for the forthcoming shake tables test. The same method of data exchange between $S2$ and foundation soil was used as with the physical substructure.

#### 3.2 | Parameters of physical substructures and similarity relationship

The experimental model was designed on the basis of the theory of similarity to realistically reflect the dynamic features of the prototype structure. The 1:5 scale model was used for the physical substructure. The main model parameters and similarity relationship are shown in Table 1. The plain size of structure $S1$ (Figure 5) was $1.6 \times 1.6$ m$^2$, the height of the ground floor was 0.68 m, the height of other floors was 0.63 mm, and the total height was 2.57 m. Beams and columns were made of H-shaped steel with a cross-sectional size of $100 \times 45 \times 6 \times 8$ mm$^4$. The steel grade was Q345. The modulus of elasticity was measured as $E = 2.02 \times 10^5$ MPa by material testing. The floor slabs were steel plates with a thickness of 3 mm. One beam was arranged directly below the floor slab along the length and breadth. The mass of each floor was as follows: $m_1 = m_2 = m_3 = 1.7 \times 10^3$ kg, $m_4 = 1.54 \times 10^2$ kg. Acceleration sensor and strain sensor were used to determine the dynamic response of the superstructure. White noise scanning was first performed at the beginning of the experiment, and the obtained data were used for spectral analysis (the number of sampling point was 8,600). Thus, the dynamic characteristics of the model structure including frequency and damping ratio were calculated.$^{[26,27]}$ The fundamental natural period of vibration was 0.42 s for structure $S1$, and the damping ratio was 0.02.

![FIGURE 4 Division of physical and numerical substructures](image-url)
Some simplifications were done to the equipment model by replacing the equipment holder with seamless circular tubes of steel; the equipment itself was simplified into a lumped mass. The outer diameter of the circular steel tube was 28 mm; the tube wall thickness was 5 mm; the steel grade was Q245. Three equipment models were considered. Figures 6 and 7 show the calculation diagram and equipment installation, respectively. Equipment parameters are shown in Table 2.

### 3.3 Numerical substructure

#### 3.3.1 Adjacent structure (structure S2)

In the experiment, the geometric and material parameters of the structure S2 are the same as that of the S1. A tandem multi-mass-point shear model was used for numerical substructure S2 (Figure 8). The mass $m_i$ of each floor was concentrated on the floor slabe; stiffness $k_i$ of each floor was the sum of lateral resistance stiffness of all columns in this floor. The Ramberg–Osgood model shown in Figure 9 was used. Here, $F_y$ is yield load, $u_y$ is yield displacement, $K_1$ is preyield stiffness, and $K_2$ is postyield stiffness ($K_2 = \alpha K_1$, $\alpha$ is stiffness reduction factor). Zhang and Wang[28] studied the restoring force model for steel frame structure. Based on previous researches and by synthesizing a large amount of experimental data, the value range of stiffness reduction coefficient $\alpha$ in the double broken line constitutive model was determined 0.05–0.1. Hong et al.[29] performed a substructure test for the steel frame model, where $\alpha$ was taken as 0.1. Therefore, $\alpha$ was 0.1 in our study. In the antiseismic design of the structures, the interstory lateral stiffness is the sum of lateral stiffness of the columns when making the rigid floor assumption. The lateral stiffness of a single column is given by $12EI/h^3$. The mass of each floor in structure S2 was the same as in structure S1. The interfloor stiffness of the bottom floor was $3.87 \times 10^6$ N/m; the interfloor stiffness from the second to the fourth floor was $4.86 \times 10^6$ N/m.

#### 3.3.2 Foundation soil model

When the free lateral boundary is used for the foundation soil and the ratio of the plane specification of the foundation soil to that of the structure is more than 5, the boundary effect will produce a small impact on the dynamic response of the structure. The length (along the orientation of the
two adjacent structures), width, and depth of the foundation soil were taken as 30, 15, and 15 m, respectively; the specification of the embedded foundation was 2.2 × 2.2 × 0.4 m³. Material parameters of the foundation and soil of each floor are shown in Table 3. According to the consensus reached by the Geotechnical Simulation Technology Committee of International Society for Soil Mechanics and Geotechnical Engineering, when the ratio of model specification to soil particle size in the model is more than 175, the deformation of soil particles and particle agglomeration will differ little from the undisturbed soil. Hence, the soil is suitable as the experimental material of the model. Here, the similarity ratio of the foundation soil was taken as 1.0.

When the soil body was considered the numerical substructure, the 3D finite element analysis model was applied in this study, with the bottom of the soil fixed and the setting of viscoelastic boundaries. However, the finite element analysis model of the foundation had a high number of degrees of freedom, and the computation would take a large amount of time. Moreover, data exchange between the physical substructures and foundation soil as the numerical substructure was a matter of timeliness (usually from a few milliseconds to several dozens of milliseconds). Therefore, some simplifications were done to the foundation soil. The vibration did not primarily occur along the width of the foundation soil, and the relative displacements of different nodes along the width were not significant. Therefore, the displacement of the nodes along the width direction was considered identical. In this way, the vibration modes were reduced for the foundation soil. Table 4 shows the model parameters, and the model was implemented as follows:

### Table 2: Equipment Parameters

| No. | Type       | Height (m) | Mass Me (kg) | Mass ratio λ = mE / ms (%) | Theoretical value of fundamental frequency fE (Hz) | Measured value of fundamental frequency fE (Hz) | Frequency ratio η = fE / fS | Specification of equipment holder |
|-----|------------|------------|--------------|----------------------------|-----------------------------------------------|-----------------------------------------------|-------------------------------|----------------------------------|
| E1  | Single layer | 0.25       | 80           | 1.20                       | 16.0                                          | 11.0                                          | 4.62                          | 28                               | 5                               |
| E2  | Single layer | 0.5        | 80           | 1.20                       | 6.0                                           | 5.1                                           | 2.14                          |                                   |                                 |
| E3  | Double layer | 0.5        | 160          | 2.41                       | 5.7                                           | 4.6                                           | 1.93                          |                                   |                                 |

Note. ms, total mass of main structure S1; fS, fundamental frequency of main structure S1.
(1) Foundation soil substructure model was established using FEM software ANSYS. Mass, stiffness, and dominant modes of the foundation soil substructure were extracted.

(2) The extracted data were input into MATLAB. All modal numbers calculated of the soil were screened and ranked in a decreasing order of mass participation factor along the vibration direction. The final modal number was determined on the basis of the consideration of analytical precision and computational capacity (Table 4). The sum of modal mass participation factors was more than 95%.

(3) The selected modal number was substituted into Equation (3) to calculate the modal mass, stiffness matrix, and damping matrix of the soil. The results were the input of the Simulink program, by which the EASSI problem was solved. This method could reduce the dimensionalities of the linear characteristic matrix of foundation soil substructure. Thus, the number of degrees of freedom was reduced, and the computational efficiency was improved. This laid the basis for the real-time computation in SSTT.

### TABLE 3  Material parameters of foundation and foundation soil

| Materials type | Material no. | Thickness (m) | Modulus of elasticity (Pa) | Density (kg/m³) | Poisson’s ratio |
|----------------|--------------|---------------|-----------------------------|-----------------|----------------|
| Foundation     | 1            | 0.4           | $3.5 \times 10^{10}$       | 2650            | 0.2            |
| Soil           | 2            | 3.6           | $2.1 \times 10^8$          | 1730            | 0.3            |
|                | 3            | 8.4           | $3.6 \times 10^8$          | 1950            | 0.3            |
|                | 4            | 3.0           | $4.3 \times 10^8$          | 2030            | 0.3            |

Figure 8  Simplified model of numerical substructure S2

Figure 9  Romberg-Osgood model
3.4 Model implementation

As shown in Figure 4, the entire model was divided into physical substructure S1E and two numerical substructures (i.e., foundation soil and upper structure S2), using the branch mode method. Dynamic interactions among the substructures were realized through force balance at the connecting points and displacement coordination. The experiment was conducted on a $3 \times 3 \text{ m}^2$ shake table. The dead weight of the shake table was 60 kN, the maximum weight of the specimen was 100 kN, and the frequency range was 0.1–50 Hz. The horizontal unidirectional vibration was imposed; three-variable control technique and digital iterative control were used.

It can be seen from Equations (4) and (10) that the excitation of physical substructure S1E was composed of horizontal seismic acceleration excitation $\ddot{u}_{s1}$, foundation translational acceleration excitation $\ddot{u}_{c1}$ that caused the rigid body displacement of the upper structure, and foundation rotational acceleration excitation $\ddot{\varphi}_{c1}$. In SSTT based on the branch mode method, horizontal acceleration excitations $\ddot{u}_{s1}$ and $\ddot{u}_{c1}$ can be imposed onto the physical substructure S1E via the shake table, but for bidirectional shake table capable of imposing only horizontal vibration, oscillation acceleration $\ddot{\varphi}_{c1}$ cannot be imposed. The effect of oscillation acceleration $\ddot{\varphi}_{c1}$ on substructure S1E is a horizontal inertial load related to height. Therefore, according to the equivalence principle of base shear, the horizontal inertial force with an inverted-triangle distribution pattern at the mass concentration site of substructure S1E caused by oscillation acceleration $\ddot{\varphi}_{c1}$ was equivalent to the horizontal inertial force with uniform distribution (after equivalence, the horizontal inertial load was no longer related to height; equivalence coefficient was taken as 1.6).

For the foundation soil substructure, the acceleration vector $\{\ddot{r}\}_{s1}$ of upper substructure S1E detected by the sensor, calculated acceleration vector $\{\ddot{u}\}_{s2}$ of numerical substructure S2, and seismic force (see Equation 9) were input into the foundation soil numerical substructure. Thus, the numerical solution was obtained by running the Simulink program. The calculated acceleration vector $\{\ddot{q}\}_{d}$ of foundation soil was substituted into Equations (10) and (11) to calculate the coupling forces of interaction, which were superimposed with the seismic force for the loading of physical substructure S1E and numerical substructure S2.

The aforementioned process was divided into the following steps:

1. Original seismic acceleration $\ddot{u}_{s1}$ in the i-th step ($i = 1$) was input into the physical substructure S1E on the shake table. $\ddot{u}_{s2}$ in the i-th step ($i = 1$) was obtained considering the traveling wave effect and used for the loading of numerical substructure S2. The acceleration $\{\ddot{r}\}_{s1}$ of physical substructure S1E was measured in the i-th step ($i = 1$), and the coupling force (CF$_1$) of interaction was calculated (see Equation 9). The acceleration $\{\ddot{u}\}_{s2}$ of numerical substructure S2 in the i-th step ($i = 1$) was calculated to obtain the coupling force (CF$_2$) of interaction (also see Equation 9).

2. Coupling forces CF$_1$ and CF$_2$ and seismic force (in Equation 9) were input as external forces into the foundation soil numerical substructure to obtain the numerical solution. Thus, the acceleration vector $\{\ddot{q}\}_{d}$ of foundation soil numerical substructure was calculated in the i-th step ($i = 1, 2, 3, ...$).

3. $\{\ddot{q}\}_{d}$ obtained by loading in the i-th step ($i = 1, 2, 3, ...$) was superimposed with the time history $\ddot{u}_{s1}$ of original seismic acceleration in the $i + 1$ step. The result was input into the shake table for the loading of physical substructure S1E. $\ddot{u}_{s2}$, calculated from $\ddot{u}_{s1}$ in the $i + 1$-th step considering the traveling wave effect, and $\{\ddot{q}\}_{d}$ was input into numerical substructure S2 for the loading. The acceleration $\{\ddot{r}\}_{s1}$ of physical substructure at the end of the $i + 1$-th step was measured. Coupling force CF$_1$ was calculated. The acceleration $\{\ddot{u}\}_{s2}$ of numerical substructure at the end of the $i + 1$-th step was calculated to obtain coupling force CF$_2$.

4. Steps (2)–(3) were repeated until the end of the experiment.

Loading in each time step was finished in real time. Thus, the actual dynamic response under a given seismic excitation was obtained for the entire system. The workflow chart is shown in Figure 10.

Data of physical substructure S1E were input via the I/O device into the foundation soil numerical substructure in the control computer. The Newmark-$\beta$ method was combined with the Newton–Raphson method for seeking a numerical solution. The results of the seismic response of numerical substructure S2 were input into the foundation soil numerical substructure, which was solved using the space-state equation. The obtained data were the input of the shake table control system via the I/O device to drive the physical substructure S1E. Another portion of the data was directly transmitted to the numerical substructure S2. In this way, the three substructures were connected by a closed loop of real-time data exchange, using data acquisition system, I/O devices, and Simulink simulation platform in the control computer. This simulation...
platform consisted of state-space equation for foundation soil (main program), elastoplastic analysis of numerical substructure S2 (subprogram), Analog Input module, and Analog Output module. They together constituted the real-time SSTT system that considered EASSI on the basis of the branch mode method.

4 | RESULTS AND ANALYSIS

4.1 | Seismic input

Elcentro wave, artificial wave, and Tianjin wave were input in the experiment. As shown by numerical simulation before the experiment, substructures S1E and S2 and soil body were in the elastic stage under minor earthquakes. Under mild earthquakes, the soil body was still in the elastic state, whereas the substructures S1E and S2 were in the nonlinear state. The amplitudes of the three earthquake waves were modulated, and substructures were loaded by frequently occurring earthquakes (0.07 g) and moderate earthquakes (0.2 g). Only the seismic response was provided under the three earthquake waves (0.2 g) due to the limited space. The working conditions are shown in Table 5. Acceleration time history and the corresponding acceleration spectrum are shown in Figures 11 and 12, respectively.

4.2 | Analysis of experimental results

In the following section, investigations were performed with respect to these issues under different earthquake ground motions: influence of EASSI effect on the seismic response of the system, influence of the equipment on the dynamic response of the system, influence of different equipment types, that is, frequency ratio and mass ratio of equipment to main structure on the seismic response of the system, and the position of the equipment inside the main structure on the seismic response of the system. We attempted to identify the patterns in the seismic response of the equipment-adjacent structure-soil interaction system, so as to provide experimental basis and data for further research on EASSI.

(1) Effect of EASSI on the seismic response of the system

Figure 13 shows the comparison of seismic responses of rigid foundation and EASSI under the Tianjin wave (0.2 g). Equipment E3 was located at the center of the top floor of structure S1. Under rigid foundation (Working condition GK3), the peak acceleration at the equipment top, peak acceleration at the center of the top floor slab of main structure S1, maximum displacement of the top floor slab, and maximum shear force at the bottom of the structure were 0.238 g, 0.639 g, 30.04 mm, and 27.61 kN, respectively. The peak seismic response at the corresponding sites was 0.160 g, 0.508 g, 25.97 mm, and 25.67 kN, respectively, after considering EASSI (Working condition GK6); there was reduction by 32.7%, 20.5%, 13.5%, and 7.0%, respectively. The dynamic response regularities under El Centro wave (0.2 g) and artificial wave (0.2 g) were similar to this. Peak seismic responses can be found in Table 6.
| Working condition no. | Description of working condition | Working condition no. | Description of working condition | Working condition no. | Description of working condition |
|-----------------------|----------------------------------|-----------------------|----------------------------------|-----------------------|----------------------------------|
| GK1                   | E3(c)B(4)R-el                    | GK10                  | E1(c)B(4)R-el                    | GK19                  | E2(c)B(4)AS-el                   |
| GK2                   | E3(c)B(4)R-a                     | GK11                  | E1(c)B(4)R-a                     | GK20                  | E2(c)B(4)AS-a                   |
| GK3                   | E3(c)B(4)R-t                     | GK12                  | E1(c)B(4)R-t                     | GK21                  | E2(c)B(4)AS-t                   |
| GK4                   | E3(c)B(4)AS-el                   | GK13                  | E1(c)B(4)AS-el                   | GK22                  | E3(e)B(4)R-el                   |
| GK5                   | E3(c)B(4)AS-a                    | GK14                  | E1(c)B(4)AS-a                    | GK23                  | E3(e)B(4)R-a                   |
| GK6                   | E3(c)B(4)AS-t                    | GK15                  | E1(c)B(4)AS-t                    | GK24                  | E3(e)B(4)R-t                   |
| GK7                   | BAS-el                           | GK16                  | E2(c)B(4)R-el                    | GK25                  | E3(e)B(4)AS-el                   |
| GK8                   | BAS-a                            | GK17                  | E2(c)B(4)R-a                     | GK26                  | E3(e)B(4)AS-a                   |
| GK9                   | BAS-t                            | GK18                  | E2(c)B(4)R-t                     | GK27                  | E3(e)B(4)AS-t                   |

Note. E1, E2, and E3: equipment; c and e: equipment in the central and eccentric positions of the floor; B: building (main structure S1); 4: No. of floor where the equipment were located; R: rigid foundation; A: adjacent structure; S: soil; El Centro; a: Artificial wave; t: Tianjin.

FIGURE 11  Time history of acceleration under the three earthquake waves

FIGURE 12  Spectral analysis of acceleration under the three earthquake waves

FIGURE 13  Seismic responses of rigid foundation and EASSI under the Tianjin wave (0.2 g; working conditions GK3 and GK6; with equipment E3 at the center of the floor slab). (a) Acceleration at the top of equipment E3. (b) Acceleration at the center of the top floor slab of S1. (c) Displacement at the center of the top floor slab of S1. (d) Shear force at the bottom of structure S1.
A comparison was made with the peak seismic responses under rigid foundation condition. With the consideration of EASSI effect, the system showed a decrease in all peak seismic responses under the action of the three seismic waves. The EASSI effect seemed to reduce the dynamic responses of the equipment and main structure to a noninsignificant extent. As compared with the rigid foundation, the presence of soil body increased the flexibility and equivalent damping of the entire system. The frequency of the system decreased, and the dynamic response characteristics and amplitudes of equipment and main structure changed correspondingly.

(2) Effect of equipment on the dynamic response of the system

Figure 14 is the comparison of the effect of equipment E3 on the seismic response of the system under the Tianjin wave (0.2 g). When there was no equipment E3 (working condition GK9), the peak acceleration at the center of the top floor slab of main structure S1 was 0.528 g, the maximum displacement of the top floor slab was 26.14 mm, and the maximum shear force at the bottom of the structure was 27.75 kN. When equipment E3 was located at the center of the top floor of structure S1 (working condition GK6), the peak seismic response at the corresponding sites was 0.508 g, 25.97 mm, and 25.67 kN, respectively. Clearly, with a dynamic feedback from equipment E3, the peak seismic response at the corresponding sites of main structure S1 decreased by 3.4%, 0.7%, and 7.5%, respectively. The dynamic response regularities under El Centro wave (0.2 g) and artificial wave (0.2 g) were similar to this. Peak seismic responses can be found in Table 7.

According to code for seismic design of buildings (2010), the antiseismic design of the entire structural model is needed under the following conditions: The natural period of vibration of the system with appendages (including supports) is more than 0.1 s, and the gravity of the appendages exceeds 1% of the gravity of the floor where the appendages are located, or the gravity of the appendages exceeds 10% of the gravity of the floor where the appendages are located. In the test, the natural period of vibration of equipment E3 is 0.22 s, and the mass is 10.4% of the mass of the top floor where equipment E3 is located. The peak value of seismic response of the system is reduced to a varying extent after considering the influence of the equipment.

As indicated by the aforementioned analyses (Figures 13 and 14), EASSI had a great impact on the seismic response of the system when compared with the rigid foundation. The mass of equipment E3 (only 2.41% of that of the structure) and stiffness of equipment were much smaller than those of the main structure (Figure 14). As a result, the dynamic feedback of the equipment had only a mild influence on the seismic response of the main structure, but still it reduced the peak seismic response of the main structure and inhibited the dynamic response of the main structure. The analysis of the effect of EASSI on seismic response under different working conditions, namely, different equipment and the same equipment in different plane positions of main structure S1 is as follows.

(3) Effect of EASSI on seismic response for different equipment types

Figure 15, 16, and 17 are comparisons of seismic responses of the system with different equipment types under El Centro wave (0.2 g), artificial wave (0.2 g), and Tianjin wave. The peak seismic responses can be found in Tables 8, 9, and 10. The mass ratio \( \lambda \) and frequency ratio \( \eta \) between equipment and main structure varied for different equipment types (see Table 2). For equipment E1 and E2, the mass ratio \( \lambda \) was equal, but the latter had a smaller frequency ratio \( \eta \). Under the same seismic effect, the decrease in frequency ratio \( \eta \) would increase the peak displacements and peak accelerations of equipment and main structure while reducing the maximum shear force at the bottom of the structure (Tables 8, 9, and 10), whether under rigid foundation or soft clay foundation. This is because the frequency ratio \( \eta \) was far above 1 for both equipments. The resonance effect would increase as the frequency ratio \( \eta \) approached toward the resonance-sensitive region of the equipment–structure system (near \( \eta = 1 \)). As a result, the seismic responses of the equipment and main structure were amplified. For equipment E2 and E3, the frequency ratio \( \eta \) differed little (\( \eta = 5 \) approximately, and it was considered that \( \eta \) of the two equipment was basically identical). Equipment E3 had a larger mass ratio \( \lambda \). Under the same seismic action, an increase in mass ratio \( \lambda \) would reduce the peak displacements and peak accelerations of equipment and main structure while increasing the maximum shear force of the structure bottom (Table 6). This is because as the mass ratio increased, the damping of
the entire system would increase correspondingly. The equipment then had a stronger inhibitory effect on the dynamic response of the main structure. For this reason, the dynamic response of the entire system decreased. Apparently, seismic responses under two foundations were closely related to the mass ratio and frequency ratio between the equipment and main structure.

It can be seen from Tables 8, 9, and 10 that the seismic responses of the system had basically consistent variation trend under rigid foundation and soft clay foundation for three different types of equipment. However, the amplitudes differed significantly. On the whole, EASSI would reduce the peak seismic responses of the equipment and main structure. The soil body enhanced the equivalent damping and flexibility of the entire dynamic system while reducing the frequency of the system. As a result, the vibration of the main structure attenuated at a faster rate. Besides, the dynamic feedback from the equipment inhibited the seismic response of the main structure.

(4) Effect of EASSI on seismic response under the eccentric layout of equipment

| Earthquake wave | Working condition | Peak acceleration at the center of top floor (g) | Maximum displacement of top floor (mm) | Maximum shear force at the bottom of S1 (kN) |
|-----------------|------------------|-----------------------------------------------|--------------------------------------|------------------------------------------|
| El Centro       | GK7 (without equipment) | 0.309                                         | 9.56                                 | 11.24                                    |
|                 | GK4 (with equipment)   | 0.264                                         | 9.12                                 | 8.63                                     |
|                 | Relative difference (%) | −14.5                                        | −4.6                                 | −23.2                                    |
| Artificial wave | GK8 (without equipment) | 0.418                                         | 26.84                                | 14.21                                    |
|                 | GK5 (with equipment)   | 0.417                                         | 28.61                                | 12.62                                    |
|                 | Relative difference (%) | +0.2                                          | +6.6                                 | −11.2                                    |
| Tianjin         | GK9 (without equipment) | 0.528                                         | 26.14                                | 27.75                                    |
|                 | GK6 (with equipment)   | 0.508                                         | 25.97                                | 25.67                                    |
|                 | Relative difference (%) | −3.4                                          | −0.7                                 | −7.5                                     |

FIGURE 14 Seismic response of the system under the Tianjin wave (0.2 g; with equipment and without equipment; working conditions GK6 and GK9). (a) Acceleration at the center of the top floor slab of structure S1. (b) Displacement of the center of the top floor slab of structure S1. (c) Shear force at the bottom of structure S1.

FIGURE 15 Seismic responses of the equipment-adjacent structure–soil interaction system with different equipment types under El Centro wave (0.2 g; working conditions GK13, GK19, and GK4). (a) Acceleration at the top of equipment. (b) Acceleration at the center of the top floor slab of S1. (c) Displacement at the center of the top floor slab of S1. (d) Shear force at the bottom of structure S1.
TABLE 8  Comparison of peak seismic responses of the system with different equipment types under EI Centro wave

| Peak of seismic response | Rigid foundation (without considering EASSI) | Soft clay foundation (with considering EASSI) |
|--------------------------|-----------------------------------------------|-----------------------------------------------|
|                          | Equipment type | Relative difference | Equipment type | Relative difference |
|                          | E1  | E2   | E3   | d1 (%) | d2 (%) | E1  | E2   | E3   | d1 (%) | d2 (%) |
| Peak acceleration at the top of equipment (g) | 0.173 | 1.065 | 0.535 | +515.6 | −49.7 | 0.159 | 0.749 | 0.382 | +371.1 | −49.0 |
| Peak acceleration at the center of top floor (g) | 0.721 | 0.762 | 0.546 | +5.7 | −28.3 | 0.321 | 0.339 | 0.264 | +5.6 | −22.1 |
| Maximum displacement of top floor (mm) | 17.65 | 18.62 | 15.95 | +5.5 | −14.3 | 9.29 | 10.04 | 9.12 | +8.1 | −9.2 |
| Maximum shear force at the bottom of S1 (kN) | 21.73 | 21.97 | 18.13 | +1.1 | −17.5 | 10.58 | 11.32 | 8.63 | +7.0 | −23.8 |

Note. 1. Relative difference $d_1 = \frac{E_2 - E_1}{E_1} \times 100\%$; 2. Relative difference $d_2 = \frac{E_3 - E_2}{E_2} \times 100\%$. The following are the same as this. EASSI: equipment–adjacent structure–soil interaction.

FIGURE 16  Seismic responses of the equipment-adjacent structure–soil interaction system with different equipment types under artificial wave (0.2 g; working conditions GK14, GK20, and GK5). (a) Acceleration at the top of equipment, (b) acceleration at the center of top floor of structure S1, (c) displacement at the center of top floor of S1, and (d) shear force at the bottom of S1.

FIGURE 17  Seismic responses of the equipment-adjacent structure–soil interaction system with different equipment types under Tianjin wave (0.2 g; working conditions GK15, GK21, and GK6). (a) Acceleration at the top of equipment, (b) acceleration at the center of top floor of structure S1, (c) displacement at the center of top floor of S1, and (d) shear force at the bottom of S1.

TABLE 9  Comparison of peak seismic responses of the system with different equipment types under artificial wave

| Peak of seismic response | Rigid foundation (without considering EASSI) | Soft clay foundation (with considering EASSI) |
|--------------------------|-----------------------------------------------|-----------------------------------------------|
|                          | Equipment type | Relative difference | Equipment type | Relative difference |
|                          | E1  | E2   | E3   | d1 (%) | d2 (%) | E1  | E2   | E3   | d1 (%) | d2 (%) |
| Peak acceleration at the top of equipment (g) | 0.183 | 1.180 | 0.762 | +544.8 | −35.4 | 0.157 | 0.861 | 0.396 | +448.4 | −54.0 |
| Peak acceleration at the center of top floor (g) | 0.721 | 0.762 | 0.546 | +2.6 | −10.8 | 0.381 | 0.503 | 0.417 | +32.0 | −17.1 |
| Maximum displacement of top floor (mm) | 29.59 | 31.23 | 27.94 | +5.5 | −10.5 | 27.17 | 29.19 | 28.61 | +7.4 | −2.0 |
| Maximum shear force at the bottom of S1 (kN) | 19.31 | 19.77 | 18.61 | +2.3 | −5.9 | 12.85 | 13.91 | 12.62 | +8.2 | −9.3 |

Note. EASSI: equipment-adjacent structure–soil interaction.
Figure 18 is the comparison of seismic responses of the system on rigid foundation and soft soil foundation under Tianjin wave (0.2 g) when the equipment E3 was placed at an eccentric position on the top floor of main structure S1 (eccentric distance e = 470 mm, as shown in Figure 7). The peak acceleration at the top of equipment E3 decreased from 1.155 g on rigid foundation to 0.993 g on soft soil foundation, the reduction being 14.0%. Under rigid foundation condition, the peak acceleration at the center of the top floor in structure S1, maximum displacement of the top floor, and maximum shear at the bottom were 0.653 g, 30.45 mm, and 27.55 kN, respectively. After considering EASSI effect, these values were 0.519 g, 25.78 mm, and 25.55 kN, which was a reduction by 20.5%, 15.3%, and 7.3%, respectively. Similar seismic response regularities of the system were observed under El Centro wave (0.2 g) and artificial wave (0.2 g). The peak seismic responses can be found in Table 11.

As shown by the above analysis, EASSI effect caused a reduction in the seismic responses of the equipment and main structure to varying degrees as compared with the rigid foundation condition. Under different earthquake ground motions, the seismic responses decreased to varying extents. A comparison was made between the system with equipment E3 in an eccentric position (working conditions GK25, GK26, and GK27) and with equipment E3 at the center of the top floor of the main structure (working conditions GK4, GK5, and GK6). For each working condition,

### Table 10

| Peak of seismic response                  | Rigid foundation (without considering EASSI) | Soft clay foundation (with considering EASSI) |
|------------------------------------------|---------------------------------------------|---------------------------------------------|
|                                          | Equipment type                          | Relative difference | Equipment type            | Relative difference |
|                                          | E1  | E2  | E3  | d1 (%) | d2 (%) | E1  | E2  | E3  | d1 (%) | d2 (%) |
| Peak acceleration at the top of equipment (g) | 0.563 | 1.989 | 0.238 | +253.3 | -88.0 | 0.502 | 1.498 | 0.160 | +198.4 | -89.3 |
| Peak acceleration at the center of top floor (g) | 0.699 | 0.704 | 0.639 | +0.72 | -9.2 | 0.515 | 0.541 | 0.508 | +5.0 | -6.1 |
| Maximum displacement of top floor (mm) | 29.69 | 36.09 | 30.04 | +21.5 | -16.8 | 25.86 | 26.18 | 25.97 | +1.2 | -0.8 |
| Maximum shear force at the bottom of S1 (kN) | 28.51 | 26.07 | 27.61 | -8.6 | +5.9 | 26.09 | 23.5 | 25.67 | -9.9 | +9.2 |

Note. EASSI: equipment-adjacent structure-soil interaction.

### Table 11

| Earthquake wave | Working condition | Peak acceleration at the top of equipment (g) | Peak acceleration at the center of top floor (g) | Maximum displacement of top floor (mm) | Maximum shear force at the bottom of S1 (kN) |
|-----------------|------------------|-----------------------------------------------|-----------------------------------------------|----------------------------------------|-------------------------------------------|
| El Centro       | GK22 (without EASSI) | 1.152 | 0.641 | 17.04 | 18.77 |
|                 | GK25 (with EASSI) | 0.528 | 0.266 | 8.89 | 54.5 |
|                 | Relative difference (%) | -54.1 | -58.5 | -47.8 | -54.5 |
| Artificial wave | GK23 (without EASSI) | 1.382 | 0.569 | 27.8 | 19.16 |
|                 | GK26 (with EASSI) | 1.079 | 0.406 | 28.58 | 13.1 |
|                 | Relative difference (%) | -21.9 | -28.6 | -2.8 | -31.6 |
| Tianjin        | GK24 (without EASSI) | 1.155 | 0.653 | 30.45 | 27.55 |
|                 | GK27 (with EASSI) | 0.993 | 0.519 | 25.78 | 25.55 |
|                 | Relative difference (%) | -14.0 | -20.5 | -15.3 | -7.3 |

Note. EASSI: equipment-adjacent structure-soil interaction.
the peak seismic responses of the main structure were basically the same (see Figures 19, 20, and 21). However, the peak responses of the equipment fluctuated significantly. Under the action of El Centro wave, the peak acceleration at the top of the equipment placed in an eccentric position (0.528 g) was 1.4-fold that of the value (0.382 g) with the equipment placed at the center (working condition GK4; Figure 19). Under the artificial wave, the peak acceleration at the top of the equipment placed in an eccentric position (1.079 g) was 2.7-fold that of the value (0.396 g) with the equipment placed at the center (working condition GK5; Figure 20). Under the Tianjin wave, the peak acceleration at the top of the equipment placed in an eccentric position (0.993 g) was about 6.2-fold that of the value (0.160 g) with the equipment placed at the center (working condition GK6; Figure 21). The above results can be explained by the deviation of the center of mass and center of rigidity at the top floor of the main structured due to the eccentric position of the equipment. This further leads to additional torsion, which aggravates the seismic responses of the equipment.

FIGURE 19  Seismic responses of the EASSI system under the El Centro wave (0.2 g; working conditions GK4 and GK25). (a) Acceleration at the top of equipment E3, (b) acceleration at the center of top floor of structure S1, (c) displacement at the center of top floor of S1, and (d) shear force at the bottom of S1

FIGURE 20  Seismic responses of the EASSI system under the artificial wave (0.2 g; working conditions GK5 and GK26). (a) Acceleration at the top of equipment E3, (b) acceleration at the center of top floor of structure S1, (c) displacement at the center of top floor of S1, and (d) shear force at the bottom of S1

FIGURE 21  Seismic responses of the EASSI system under the Tianjin wave (0.2 g; working conditions GK6 and GK27). (a) Acceleration at the top of equipment E3, (b) acceleration at the center of top floor of structure S1, (c) displacement at the center of top floor of S1, and (d) shear force at the bottom of S1
5 | CONCLUSIONS

In this study, the branch mode method was used in SSTT that considered EASSI, consisting of equipment E, two adjacent four-story steel structures S1 and S2, and foundation soil. Loading was performed on the shake table with equipment E and main structure S1 as physical substructure S1E; foundation soil and structure S2 were taken as two numerical substructures for computer simulation. Seismic responses of the entire system were analyzed on the basis of real-time data exchange among the substructures. This represents a new approach to antiseismic experiment relating to EASSI. The effect of EASSI was analyzed on seismic responses of the system with different equipment types, or different plane positions of the same equipment in main structure S1. The following conclusions were reached:

(1) Utilizing the principle of branch mode method, equipment-adjacent structure-foundation soil system was divided into different substructures, with equipment and main structure taken as one substructure. The coupling terms of interaction were proposed and used for data exchange between the physical substructure and numerical substructure. Through these coupling terms, the branch mode method was effectively introduced into SSTT.

(2) With the consideration of EASSI, the presence of foundation soil increased the flexibility and equivalent damping of the entire system, causing the frequency of the system to decrease. The dynamic response characteristics and amplitudes of the equipment and main structure also varied significantly. The dynamic feedback of the equipment also inhibited the seismic responses of the main structure and accelerated the vibration attenuation of the structure. Therefore, the seismic responses of the equipment and main structure were much reduced by the overall effect of EASSI.

(3) Under the same seismic action, the seismic response of the system was closely related to the mass ratio and frequency ratio between the equipment and main structure, whether under rigid foundation or soft clay foundation. Under the same mass ratio, when the frequency ratio approached toward the resonance-sensitive region of the equipment-main structure system (near $\eta = 1$), the seismic response of the system would increase. Under the same frequency ratio between the equipment and main structure, equipment with a larger mass had a stronger inhibitory effect on the dynamic response of the main structure. The dynamic response of the system would decrease with the increase in mass ratio.

(4) Compared with the central layout of the equipment, an eccentric layout would cause separation between the center of mass and the center of stiffness. This further led to additional torsion and amplification of the seismic response of the equipment. Therefore, it is necessary to consider the adverse effect brought by the eccentric layout of equipment in the antiseismic design.

ACKNOWLEDGMENTS

This work was financially supported by the National Natural Science Foundation of China (51478312, 51278335), Natural Science for Youth Foundation (51208356).

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How to cite this article: He T, Jiang N. Substructure shake table test for equipment-adjacent structure–soil interaction based on the branch mode method. Struct Design Tall Spec Build. 2019;28:e1573. https://doi.org/10.1002/tal.1573