

# REAL-TIME DYNAMIC HYBRID TESTING FOR SOIL-STRUCTURE INTERACTION ANALYSIS

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## ABSTRACT

The consideration of the infinite foundation radiation damping is a difficult problem in shaking-table tests. In this paper, a lumped parameter model of soil foundation is introduced into the numerical substructure of real time dynamic hybrid testing (RTDHT). By coupling foundation calculation and structure model testing, the whole system response that considers soil-structure interaction (SSI) can be obtained. A two storey-structure mounted on soil foundation is tested on the shaking table using RTDHT test method. The structure dynamic responses, including acceleration and shear force, are obtained under soft soil and hard soil conditions. The RTDHT results are compared with the results of the finite element analysis. It shows that RTDHT based on lumped parameter soil foundation model can achieve satisfactory precision for experimental tests considering the SSI effects.

## 1. INTRODUCTION

In the 1930s, researchers began to realize that soil and the structure on the soil should be considered as a whole system in dynamic analysis. In the past few decades, tremendous effort has been made by many researchers to obtain an accurate and effective numerical calculation method for soil-structure interaction analysis. Generally, the soil-structure interaction analysis method can be grouped into the direct method and substructure method (Wolf 1994). In the direct method, the structure and soil are treated as a whole system. The region of the soil adjacent to the structure-soil interface is also explicitly modeled. Artificial boundary must be introduced so as to cover the unbounded soil domain. In the substructure method, the structure and the soil are treated as two different substructures. Each substructure can be analyzed using a best-suited computational technique. Combining the force-displacement relationship of the soil with the discretized motion equation of the structure, results in the final system of equation of the total dynamic system.

Although many theoretical SSI analysis methods have been developed, considering SSI in shaking-table tests is still a problematic option. Conceptually, it can be done in a similar manner as the direct method that a finite region of soil modeled together with the structure (Chen, et al. 2005). The soil is filled in a model box fixed on the shaking-table. However, the box boundaries will reflect the wave back into the model and affect the dynamic response of the system (Shi, et al. 2007). On the other hand, limited by the bearing capacity of the shaking-table, the superstructures are commonly modeled by a small-scale model, which is hard to represent the dynamic properties of the prototype, especially for the dimension sensitive structures.

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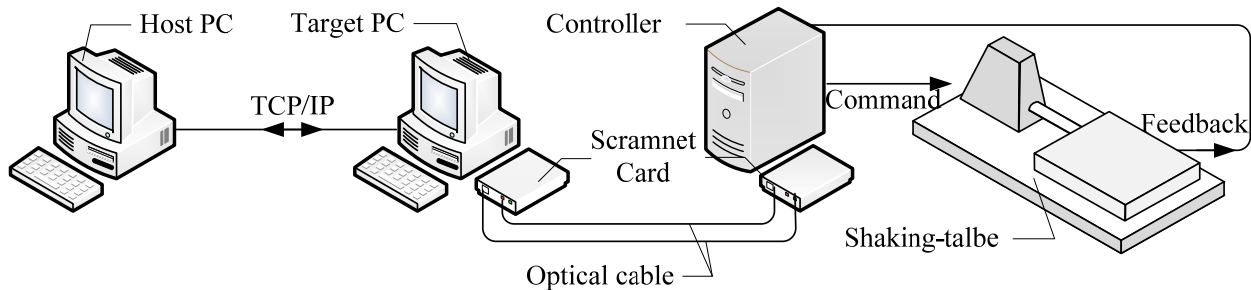
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Alternatively, the soil-structure interaction in shaking-table tests is considered by combining the numerical calculations of semi-infinite soil together with superstructure large scale model testing in this paper. This idea comes from the conception of real-time dynamic hybrid testing, a novel testing method developed in recent years (Nakashima and Kato 1992). The fundamental idea of real-time dynamic hybrid testing is to split the whole system under consideration into two parts. The one, so-called numerical substructure, will be simulated in the computer by some developed numerical models. The physical substructure, i.e. the remainder, which may be nonlinear, rate-dependent, or complex constitution, is built as a physical model and tested on dynamic testing equipments (commonly, shaking-tables or dynamic actuators).

Using the idea of real-time dynamic hybrid testing, the theoretical analysis model of semi-infinite soil can be introduced into the numerical substructure of RTDHT system, while the large scale model of the superstructure can be built and tested on the shaking-table. These two parts alternating in real-time and the dynamic response of the whole soil-structure interaction system can be obtained. In this paper, the dynamic soil-structure interaction response of a two storey structure founded on soil foundation is analyzed by the real-time dynamic hybrid testing system in Tsinghua University.

## 2. RTDHT SYSTEM IN TSINGHUA UNIVERSITY

The real-time dynamic hybrid testing system in Tsinghua University (TH-RTDHT) is illustrated in Figure 1. It is mainly composed of the distributed real-time calculation system, the shared common RAM network, and the shaking-table loading system.



**Fig. 1 Layout of Tsinghua University RTDHT system**

### 2.1 Distributed Real-Time Calculation System

Ordinary PC operating systems such as Windows and Linux are not real-time systems. They cannot guarantee to give out calculation results before the specified time point, especially when the problem being calculated is complicated and the calculation procedure has to contend for computer resources with other procedures. In the TH-RTDHT system, a real-time calculation system is constructed on standard PC with the help of the xPC TARGET software. Two computers are employed: the host PC and the target PC (Figure 2). The former is an ordinary computer with Windows operating system and the later only runs xPC TARGET kernel without any other operating system. They are linked with a net cable under the protocol of TCP/IP. Host PC is used to develop the diagram calculation procedure in the environment of SIMULINK. It also controls the beginning and ending of the test, while inspects the procedure during the test. When the diagram procedure is well-developed and debugged on the host PC, it is transformed

into the standard C code by REAL TIME WORKSHOP, which is also a toolbox of Matlab. The C code is downloaded onto the target PC and executed in real-time.



**Fig. 2 Distributed real-time calculation system**

## 2.2 Shared Common RAM Network

Shared common RAM network (SCRAMNet) is developed by Systran Corporation to satisfy the demanding real-time requirements of high-fidelity simulations. Its capabilities extend equally well to virtually all other distributed real-time applications. Based upon a replicated shared-memory concept, the SCRAMNet network is optimized for the high-speed, ultra-low-latency transfer of data among many computing platforms that are all solving portions of the same real-time problem.

In the TH-RTDHT system, target PC and shaking-table controller are equipped with two SCRAMNet cards SC150. They are linked with optical cables to form a ring topology. Each node in the net has an identical computer addressable memory up to 8 MB. The data transfer speed between the target PC and the controller can reach up to 16.7 MB/sec while the latency is not more than 250 nanoseconds. At any given time that an application process updates a data structure located in its local SCRAMNet memory, the address and data are immediately (and automatically) broadcasted to all other nodes on the SCRAMNet network. This automatic data transfer requires no software intervention or backplane loading, enabling the host computer to provide more processing resources to the application.

## 2.3 Shaking-Table Loading System

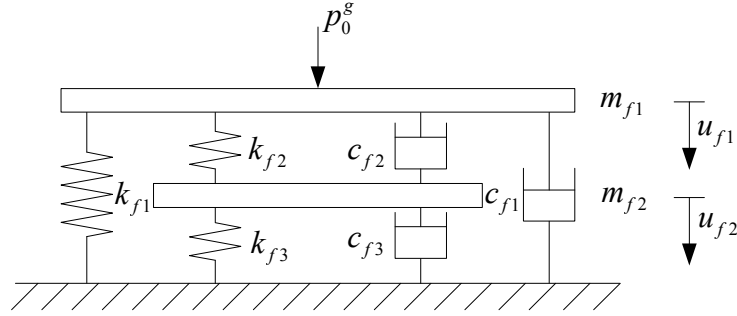
TH-RTDHT system has two uni-axial shaking-tables. Each table has a 1.5 m×1.5 m working area and a bearing capacity of 2 t. The frequency range is 0~50 Hz. The maximum acceleration can reach up to 3.6 g when the table is bare but 1.2 g when the table is loaded to full capacity.

An MTS 469DU controller is used in this system. The controller is configured to allow one uni-axial table to operate by itself or is configured to allow both tables to run two independent tests with separate data acquisition, function generation, and other functions at the same time. MTS integrates Mathworks' Simulink into its MTS Controller platform to achieve real time hybrid control capabilities. With this control solution, all controller feedback outputs are made available for use in Simulink, while commands or controller correction signals generated by Simulink can drive the controller command inputs. This makes it possible to construct an RTDHT system based on the shaking-table.

### 3. SOIL-STRUCTURE INTERACTION MODEL

#### 3.1 Lumped Parameter Model of Soil

Lumped parameter model was first developed by Lysmer in 1966. In the lumped parameter model, the dynamic stiffness of the soil is approximated by assembling several frequency independent springs, dashpots, and mass blocks. Later, many researchers made numerous efforts (Wolf 1986; Francisco 1990) in developing high accuracy lumped parameters in a wide frequency range. In this paper, a lumped parameter model with 2 degrees of freedom is used for its simplicity. The model comprises two mass blocks, three springs, and three dashpots and assembled together as shown in Figure 3 (Luan and Lin 1996).



**Fig. 3 Lumped parameter model of soil**

The motion equation of the lumped parameter soil model under external force excitation can be written as

$$M_f \ddot{U}_f + C_f \dot{U}_f + K_f U_f = P_f \quad (1)$$

where  $M_f$ ,  $C_f$ , and  $K_f$  are mass matrix, damping matrix, and stiffness matrix, respectively.  $U_f$  is the displacement vector, and  $P_f$  is the external force vector. They are expressed as follows:

$$M_f = M_{fe} \begin{bmatrix} m_{f1}^* \\ m_{f2}^* \end{bmatrix}, C_f = C_{fe} \begin{bmatrix} c_{f1}^* + c_{f2}^* & -c_{f2}^* \\ -c_{f2}^* & c_{f2}^* + c_{f3}^* \end{bmatrix}, K_f = K_{fe} \begin{bmatrix} k_{f1}^* + k_{f2}^* & -k_{f2}^* \\ -k_{f2}^* & k_{f2}^* + k_{f3}^* \end{bmatrix} \quad (2)$$

$$U_f = [U_{f1} \quad U_{f2}]^T, P_f = [P_{f1} \quad P_{f2}]^T \quad (3)$$

where  $K_{fe}$ ,  $C_{fe}$ , and  $M_{fe}$  are unified stiffness, damping, and mass, respectively. In case of strip footing on elastic foundation, the unified parameters can be calculated according to the following formulas (Wolf 1985):

$$K_{fe} = K_s = \pi \rho c_s^2 \quad (4)$$

$$C_{fe} = K_s \cdot \frac{r_0}{c_s} \quad (5)$$

$$M_{fe} = K_s \cdot \left( \frac{r_0}{c_s} \right)^2 \quad (6)$$

where  $K_s$  is the static stiffness;  $C_s$  is the shear wave velocity of soil; and  $d$  is characteristic length of the foundation (for strip foundation, it is the half width of the rigid footing). If the system is excited by harmonic force, the external force and displacement vector can be rewritten as follows:

$$P_f = [P_{f0} \quad 0]^T e^{i\omega t}, U_f = [U_{f1,0} \quad U_{f2,0}]^T e^{i\omega t} \quad (7)$$

The dynamic-stiffness coefficient of the lumped parameter model is defined as follows:

$$K(\omega) = P_{f0} / U_{f1,0} \quad (8)$$

Substituting Equation (7) into the dynamic-stiffness coefficient function leads to

$$\begin{aligned} K(a_0)/K_s &= k(a_0) + ia_0c(a_0) \\ &= \left[ k_{f1}^* + k_{f2}^* - a_0^2 m_{f1}^* - \frac{(k_{f2}^{*2} - a_0^2 c_{f2}^{*2})(k_{f2}^* + k_{f3}^* - a_0^2 m_{f2}^*) + 2a_0^2 k_{f2}^* c_{f2}^* (c_{f2}^* + c_{f3}^*)}{(k_{f2}^* + k_{f3}^* - a_0^2 m_{f2}^*)^2 + a_0^2 (c_{f2}^* + c_{f3}^*)^2} \right] \\ &\quad + ia_0 \left[ c_{f1}^* + c_{f2}^* - \frac{2k_{f2}^* c_{f2}^* (k_{f2}^* + k_{f3}^* - a_0^2 m_{f2}^*) - (c_{f2}^* + c_{f3}^*)(k_{f2}^{*2} - a_0^2 c_{f2}^{*2})}{(k_{f2}^* + k_{f3}^* - a_0^2 m_{f2}^*)^2 + a_0^2 (c_{f2}^* + c_{f3}^*)^2} \right] \end{aligned} \quad (9)$$

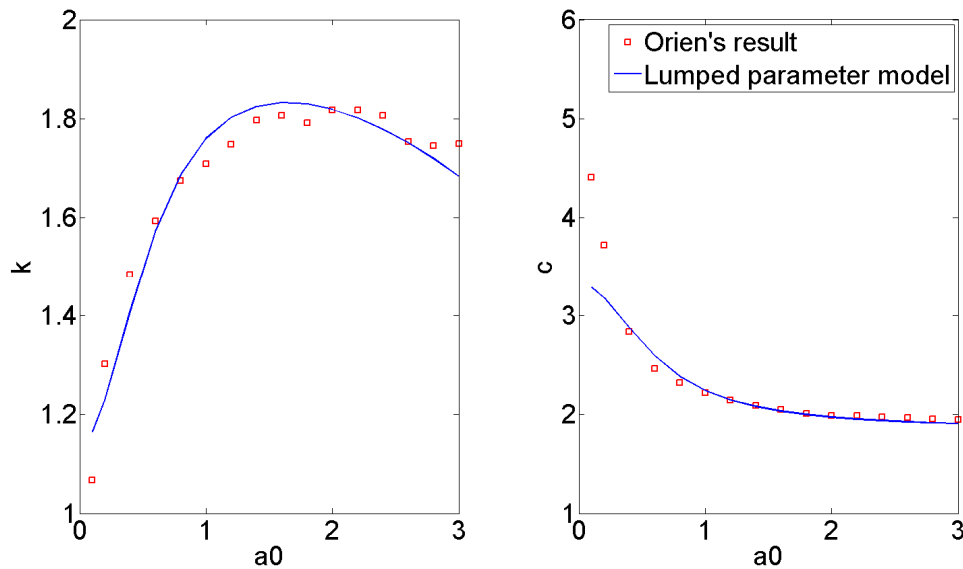
where  $a_0$  is a dimensionless frequency parameter defined as

$$a_0 = \frac{\omega r_0}{c_s} \quad (10)$$

The dynamic stiffness coefficient  $K(a_0)$  is determined by eight parameters:  $m_{f1}^* \dots c_{f3}^*$ . Base on the dynamic stiffness coefficient exactly developed by Orien (1971), the eight parameters in this lumped parameter model can be obtained through multiple regression analysis. Table 1 shows the eight coefficients obtained through multiple regression analysis. The dynamic stiffness of the lumped parameter model using these parameters is compared with Orien's results in Figure 4. In the region of dimensionless frequency from 0 to 3, the dynamic stiffness coefficients of the lumped parameter model coincide well with Orien's results.

**Table 1 Parameters of the lumped parameter model**

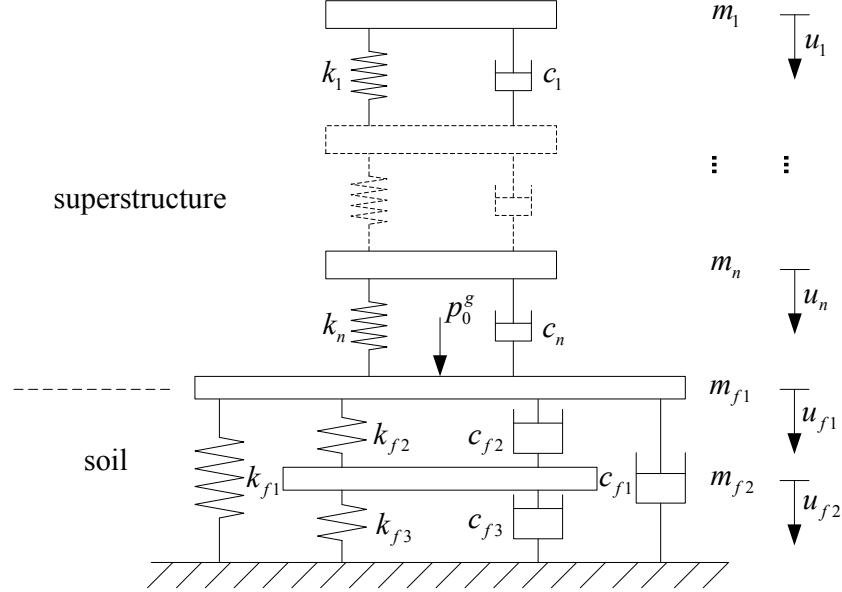
$m_{f1}^*$	$m_{f2}^*$	$k_{f1}^*$	$k_{f2}^*$	$k_{f3}^*$	$c_{f1}^*$	$c_{f2}^*$	$c_{f3}^*$
0.035	0.150	1.423	0.716	-0.202	1.724	0.126	0.815



**Fig. 4 Dynamic stiffness of elastic half plane foundation**

### 3.2 Motion Equation of SSI System

For a general condition, an  $n$ -storey building is founded on semi-infinite soil foundation. The soil is simulated with the lumped parameters developed in the last section while the structure also can be idealized into mass blocks, springs, and dashpots. The whole model is illustrated in Figure 5.



**Fig. 5 SSI model based on the lumped parameter soil model**

The motion equation of the SSI system can be written as follows:

$$\begin{bmatrix} m_1 & & & & & & \\ & \ddots & & & & & \\ & & m_n & & & & \\ & & & m_{f1} & & & \\ & & & & m_{f2} & & \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \vdots \\ \ddot{u}_n \\ \ddot{u}_{f1} \\ \ddot{u}_{f2} \end{Bmatrix} + \begin{bmatrix} c_1 & -c_1 & & & & & \\ -c_1 & \ddots & \ddots & & & & \\ & \ddots & \ddots & c_{n-1} + c_n & & -c_n & \\ & & \ddots & -c_n & c_n + c_{f1} + c_{f2} & -c_{f2} & \\ & & & -c_{f2} & -c_{f2} & c_{f2} + c_{f3} & \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \vdots \\ \dot{u}_n \\ \dot{u}_{f1} \\ \dot{u}_{f2} \end{Bmatrix} + \begin{bmatrix} k_1 & -k_1 & & & & & \\ -k_1 & \ddots & \ddots & & & & \\ & \ddots & \ddots & k_{n-1} + k_n & & -k_n & \\ & & & -k_n & k_n + k_{f1} + k_{f2} & -k_{f2} & \\ & & & -k_{f2} & -k_{f2} & k_{f2} + k_{f3} & \end{bmatrix} \begin{Bmatrix} u_1 \\ \vdots \\ u_n \\ u_{f1} \\ u_{f2} \end{Bmatrix} = \begin{Bmatrix} 0 \\ \vdots \\ 0 \\ p_0^g \\ 0 \end{Bmatrix} \quad (11)$$

where  $m_i$ ,  $c_i$ , and  $k_i$  indicate mass, damping and stiffness of the  $i^{\text{th}}$  storey, and  $P_0^g$  is the effective drive force at the interface of soil and structure. The index  $f$  indicates that the parameter is associated with the foundation. The foundation associated parameters in Equation (11) have the following relationship with unified mass, damping and stiffness:

$$m_{fi} = M_{fe} \cdot m_{fi}^* ; \quad c_{fi} = C_{fe} \cdot c_{fi}^* ; \quad k_{fi} = K_{fe} \cdot k_{fi}^* \quad (12)$$

To analyze soil-structure interaction, it is sufficient to think of the design motion as being derived from an observed record at this very site, or at least at a similar site. Starting from this point, the earthquake motion throughout the free field is calculated. For the surface foundation,

the effective foundation input motion  $u_0^g(t)$  equals the free field motion  $u_0^f(t)$ . The seismic excitation can be converted to an equivalent load acting on the interface of soil and structure.

$$P_0^g(t) = Ku_0^g(t) \quad (13)$$

where  $K$  indicates the dynamic stiffness of the soil foundation model. In this paper, it is equivalent to the dynamic stiffness of the lumped parameter model.

#### 4. SSI-RTDHT TEST SETUP

Without loss of generality, the problem in Section 3.2 is concretized as the problem of a two-storey structure founded on soft soil structure (Fig. 6). The structure has the following properties: the top storey mass  $m_1=5.28 \times 10^4$  kg, stiffness  $k_1=5.46 \times 10^7$  N/m, damping  $c_1=1.47 \times 10^5$  Ns/m; the bottom storey mass  $m_2=10.86 \times 10^4$  kg, stiffness  $k_2=4.40 \times 10^7$  N/m,  $c_2=1.66 \times 10^5$  Ns/m. It is founded on the surface of a semi-infinite soil foundation with a Poisson's ratio  $\nu=1/3$ , a mass density  $\rho=2000$  kg/m<sup>3</sup>. To consider the effect of soil foundation stiffness, two shear wave velocities are adopted:  $c_s=200$  m/s for soft soil and  $c_s=800$  m/s for hard soil. To substitute the parameters into Equations (4) ~ (6), the unified mass, stiffness, and damping of the lumped parameter model can be calculated as follows: (1) soft soil,  $K_{fe}=2.513 \times 10^8$  N/m,  $C_{fe}=3.016 \times 10^7$  Ns/m,  $M_{fe}=3.619 \times 10^6$  kg; (2) hard soil,  $K_{fe}=4.021 \times 10^9$  N/m,  $C_{fe}=1.2064 \times 10^8$  Ns/m,  $M_{fe}=3.619 \times 10^6$  kg.

##### 4.1 Substructuring

According to the idea of RTDHT, the whole system should be split into two parts. In soil-structure interaction problem, it is difficult to model the semi-infinite soil foundation as a physical specimen. As such, the soil foundation should be treated as a numerical substructure in the test to be calculated in real-time based on the theoretical model. Normally, the superstructure is a finite building that can be modeled as a physical model according to a proper similarity. On the other hand, as the development of earthquake-reduction theory, more and more earthquake-reduction components such as TMD are utilized on high-rise structures. These components are normally represented as nonlinear and rate-dependent, thus testing its big scale model or even its prototype is necessary in earthquake engineering research.

Therefore, in soil-structure interaction problem, it is appropriate to make the semi-infinite soil foundation as the numerical substructure, while the superstructure (together with some nonlinear earthquake-reduction components) serves as the physical substructure (Figure 6). As above-mentioned, the whole system motion equation (Equation 11) can also be divided into two parts:

##### 1) Physical substructure motion equation

$$\begin{bmatrix} m_1 & \\ & m_2 \end{bmatrix} \begin{Bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{Bmatrix} + \begin{bmatrix} c_1 & -c_1 \\ -c_1 & c_1 \end{bmatrix} \begin{Bmatrix} \dot{u}_1 \\ \dot{u}_2 \end{Bmatrix} + \begin{bmatrix} k_1 & -k_1 \\ -k_1 & k_1 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} = \begin{Bmatrix} 0 \\ -T \end{Bmatrix} \quad (14)$$

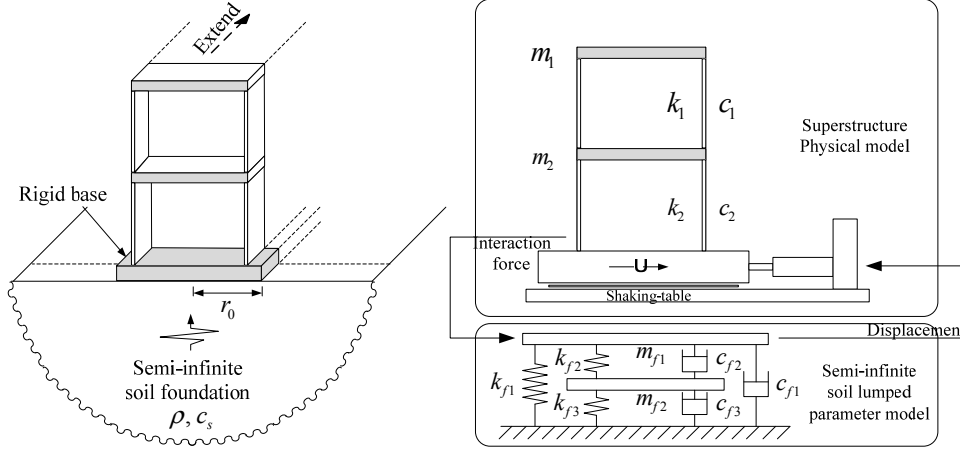
##### 2) Numerical substructure motion equation

$$\begin{bmatrix} m_{f1} & \\ & m_{f2} \end{bmatrix} \begin{Bmatrix} \ddot{u}_{f1} \\ \ddot{u}_{f2} \end{Bmatrix} + \begin{bmatrix} c_{f1} + c_{f2} & -c_{f2} \\ -c_{f2} & c_{f2} + c_{f3} \end{bmatrix} \begin{Bmatrix} \dot{u}_{f1} \\ \dot{u}_{f2} \end{Bmatrix} + \begin{bmatrix} k_{f1} + k_{f2} & -k_{f2} \\ -k_{f2} & k_{f2} + k_{f3} \end{bmatrix} \begin{Bmatrix} u_{f1} \\ u_{f2} \end{Bmatrix} = \begin{Bmatrix} p_0^g + T \\ 0 \end{Bmatrix} \quad (15)$$

where  $p_0^g$  is the effective drive force,  $T$  is the shear force between superstructure and soil foundation.

$$T = c_2 (\dot{u}_2 - \dot{u}_{f1}) + k_2 (u_2 - u_{f1}) \quad (16)$$

In the RTDHT tests, shear force  $T$  is measured directly from the sensors.



**Fig. 6 Rigid strip footing bonded to an elastic half space**

## 4.2 Similarity Model

Commonly, similarities of material properties, geometry, load, and dynamic properties should be considered in a structure dynamic test. However, when the model is idealized as spring oscillators mentioned above, only the similarities of load and dynamic properties have to be considered. These similarities include mass scale  $c_m$ , stiffness scale  $c_k$ , damping scale  $c_c$ , time scale  $c_t$ , displacement scale  $c_u$ , and force scale  $c_F$ . For security and convenience reasons, we constructed a small scale model in this study. The basic similarity ratios are chosen as  $c_m=10^4$ ,  $c_t=1$ , and  $c_u=1$ . Then, other similarity ratios can be derived from the basic similarity ratios:

$$c_k = c_c = c_m = c_F = 10^4 ; c_a = c_v = c_u = 1 \quad (17)$$

According to the above similarity ratios, the two-storey structure and soil foundation are scaled down as a test model (including the physical and numerical substructures). Parameters of the prototype and its scaled down model are listed in Table 2. The structure model and the shaking-table are shown in Figure 7. As the table is a uni-axial shaking-table, the test is only conducted in the horizontal direction.

**Table 2 Parameters of the prototype and model**

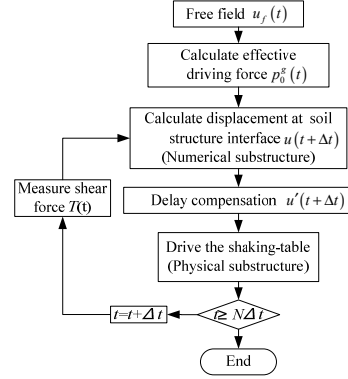
Parameters	Physical substructure		Numerical substructure	
	Top storey	Bottom storey	Soft soil	Hard Soil
Prototype				
Mass / Kg	$5.28 \times 10^4$	$10.86 \times 10^4$	$3.619 \times 10^6$	$3.619 \times 10^6$
Stiffness /N·m <sup>-1</sup>	$5.46 \times 10^7$	$4.40 \times 10^7$	$2.513 \times 10^8$	$4.021 \times 10^9$
Damping /Ns·m <sup>-1</sup>	$1.47 \times 10^5$	$1.66 \times 10^5$	$3.016 \times 10^7$	$1.206 \times 10^8$
Model				
Mass / Kg	5.28	10.86	361.9	361.9
Stiffness /N·m <sup>-1</sup>	5460	4400	$2.513 \times 10^4$	$4.021 \times 10^5$
Damping /Ns·m <sup>-1</sup>	14.7	16.6	3016	1206



Note : the parameters that correspond to soil foundation are unified mass  $M_{fe}$ , stiffness  $K_{fe}$ , and damping  $C_{fe}$ .



**Fig. 7 Specimen and loading system**



**Fig. 8 Flow chart of SSI-RTDHT**

### 4.3 Procedure of SSI-RTDHT Test

Figure 8 shows the flow chart of the SSI-RTDHT test. Before the test begins, the effective driving force is calculated from the free field record according to the dynamic stiffness of the lumped parameter model of soil. In each step, the shear force between two substructures is measured from the sensors on the physical model and fed back into the MTS controller. The controller is designed to be able to transfer the feedbacks into the SIMULINK procedure in the target PC. Using the shear force feedback  $T(t)$  and the effective driving force  $P_0^g(t)$ , the SIMULINK procedure calculates the displacement response at the interface based on the lumped parameter model. To address the problem of inevitable time delay in hydraulic servo system (Horiuchi et al. 1999), the calculated displacement  $u(t + \Delta t)$  is predicted forward as  $u'(t + \Delta t)$  using a delay compensation method. In this paper, the compensation method developed by Wallace et al. (2005) is used to compensate for the time delay, which is about 10 ms in this system. After that, the shaking-table is driven by the predicted command  $u'(t + \Delta t)$  and the superstructure model vibrates under the excitation. At this point, one loop of the test is finished. The  $t = t + \Delta t$  is then allowed to start another loop until the whole test is finished.

## 5. SSI-RTDHT TEST RESULTS

### 5.1 Finite Element Mesh and Artificial Seismic Wave

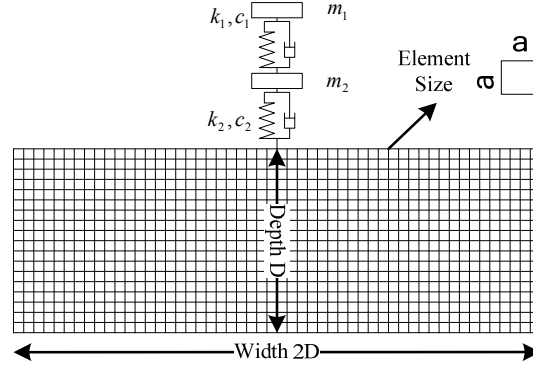
A finite element model with far fixed boundary is built to obtain the numerical exact results of the SSI problem. The mesh boundary is far enough so that the effect of the wave reflection and scattering from boundary on the structure-soil interface response can be avoided during the calculation duration. To do this, the mesh has to satisfy the following dimension requirements:

$$D \geq \frac{cT}{2}; \quad L \geq 2D; \quad a \leq \frac{c}{10f} \quad (18)$$

where  $D$  and  $L$  are the depth and width of discretized region, respectively;  $a$  the element size;  $c$  the wave velocity;  $T$  the calculation duration; and  $f$  the highest frequency component of the wave. The local part of the mesh is illustrated in Figure 9. The mesh parameters used in this model are listed in Table 3.

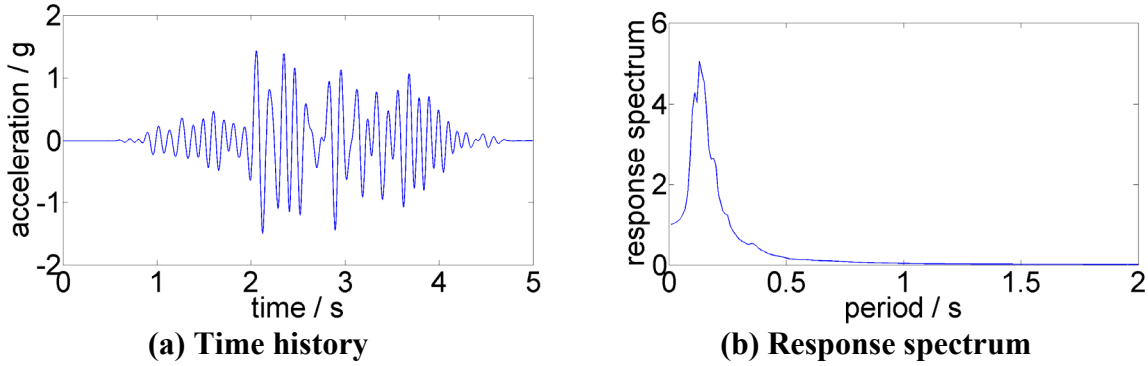
**Table 3 Finite element analysis parameters**

Wave velocity $c_s/m\cdot s^{-1}$	Calculation duration $T/s$	Highest frequency component / Hz	Depth $D/m$	Element size $a/m$	Node number
200	5	10	500	2	125751



**Fig. 9 Finite element mesh (Local)**

The artificial seismic wave used in this test is shown in Figure 10. Its main frequency components located in the range from 2 Hz to 10 Hz, and the structure vibration frequencies are  $f_1=2.5\text{Hz}$ , and  $f_2=6.6\text{Hz}$ .



**Fig. 10 Artificial seismic wave used in test**

## 5.2 Accuracy Verification of the Lumped Parameter Model

To verify the approximation accuracy of the lumped parameter model used in this paper, the same problem is respectively calculated by the finite element method (FEM) with the above-mentioned boundary and by the central differential method (CDM) with the lumped parameter model. The FEM results are considered to be the exact solution of the problem. The comparison between the CDM results and the exact solutions is shown in Figure 11, from which the accuracy of the lumped parameter model is illustrated.

Figure 11a shows the acceleration comparison between FEM and the lumped parameter model. They coincide well with each other in terms of amplitudes and frequencies. Peak of the top storey acceleration is 1.226 g by the lumped parameter model while the exact solution is 1.147 g. For the bottom storey, the peak is 0.793 g while the exact solution is 0.773 g. The errors are 6.9% and 2.6%, respectively.

Due to the limitation of the sensors utilized, only the displacement at the soil-structure interface can be measured in the test. Thus, in the following discussion, only the displacement at the interface is compared. Figure 11b shows the displacement comparison, where it can be seen that the two methods produce almost identical results. In the peak comparison, the exact solution is 9.594 mm while the lumped parameter model result is 9.751 mm, or an error of 1.6%.

Figure 11c shows the shear force comparison between FEM results and lumped parameter model results. Peak of the top storey exact solution is 597.3 kN, while the test result is 635.1 kN, or an error of 6.33%. For the bottom storey, the exact solution is 689.6 kN while the test result is 688.7 kN. Thus the error is 0.13%.

To conclude the above comparisons, the lumped parameter model can give a satisfactory approximation of the dynamic stiffness of the infinite soil foundation.

### **5.3 Accuracy Verification of the SSI-RTDHT Test**

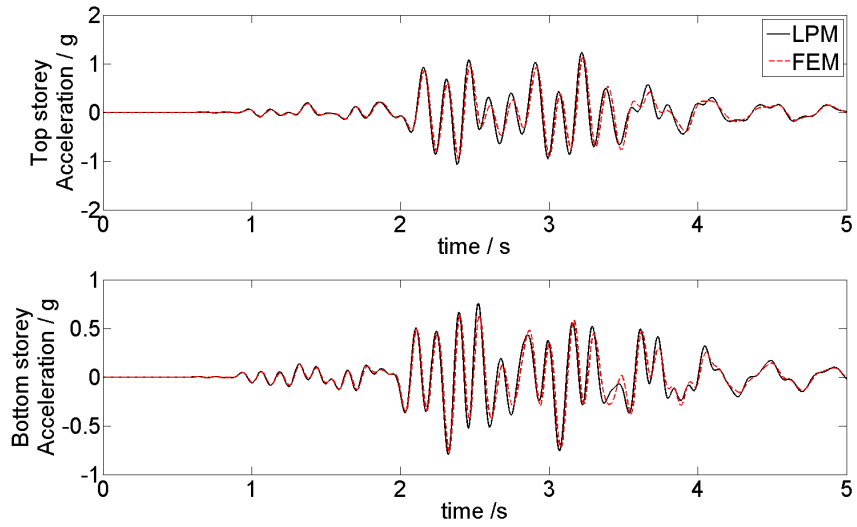
As the accuracy of the lumped parameter model itself is good enough, the focus shifts to the accuracy of SSI-RTDHT based on lumped parameter model. In this section, the SSI-RTDHT test is applied to test the structure response. The comparisons between SSI-RTDHT test results and FEM calculation results are shown in Figure 12. All the test results shown in Figure 12 have been transformed into the prototype response according to the similarity ratios in Equation (17).

The acceleration comparison in Figure 12a shows that RTDHT test results coincide well with FEM results in terms of the vibration frequency, but there are some amplitude differences between the two time histories. The test result of the top storey acceleration peak is 1.317 g with an error of 14.8% in relation to the exact solution. Test result of the bottom storey acceleration peak is 1.179g with an error of 52.5%.

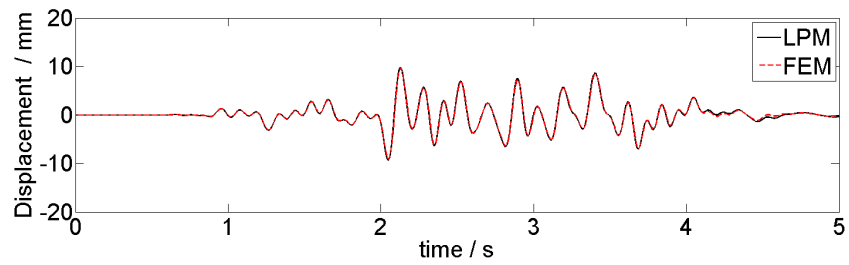
The soil structure interface displacement comparison is shown in Figure 12b. Obviously, the test displacement amplitudes are bigger than the exact solution. For the peak comparison, the test result is 10.39 mm while the exact result is 9.594 mm, or an error of 8.3%. Although not as good as the comparison in Figure 11b, the result is acceptable.

For the shear force comparison, peak of the top storey exact solution is 597.3 kN while the test result is 659.7 kN, or an error of 10.4%. Peak of the bottom storey exact solution is 689.6 kN while the test result is 728.2 kN, or an error of 5.6%.

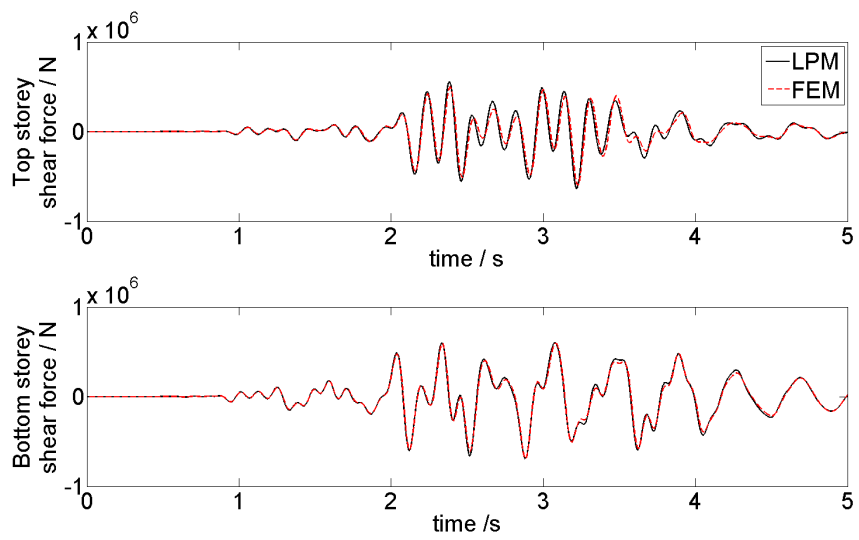
From the above comparison, it can be concluded that SSI-RTDHT tests give satisfying results though some test errors still occur. The test errors come from two places: one is the accuracy of lumped parameter model and the other one, which is the more important, is the accuracy of the RTDHT test system, such as the accuracy of the shear force feedback and accuracy of shaking-table controlling. By solving these two problems, a major accuracy improvement can be obtained.



**(a) Acceleration**

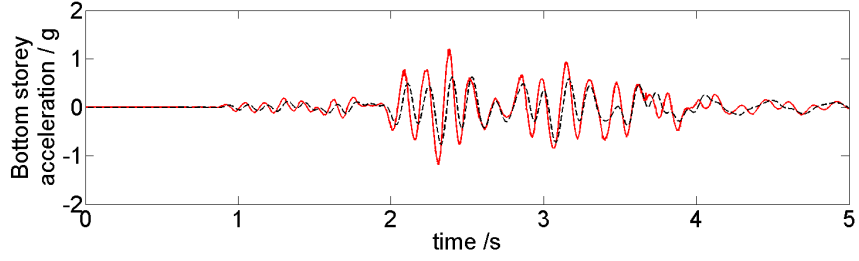
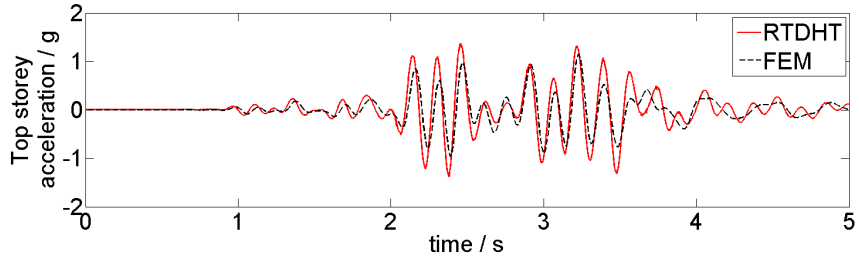


**(b) Displacement**

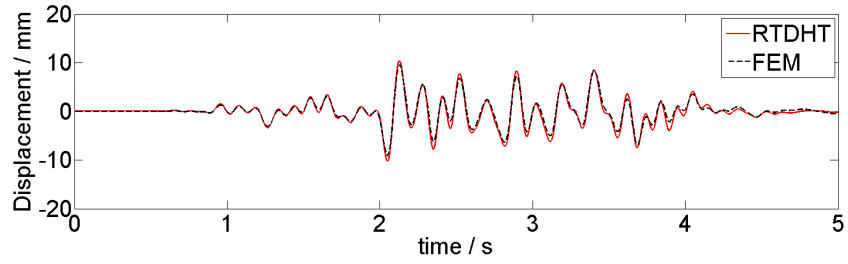


**(c) Shear force**

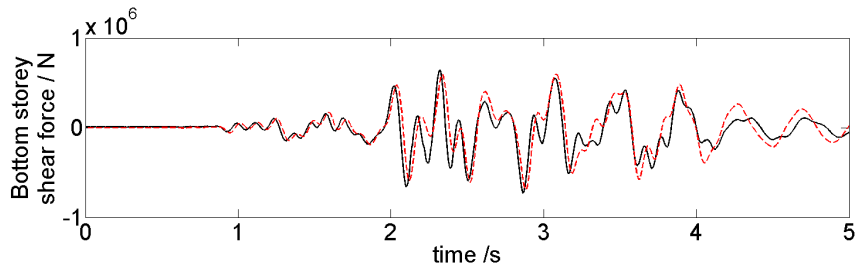
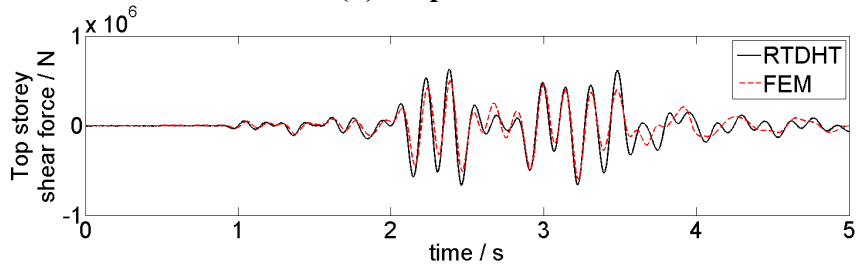
**Fig. 11 Results comparison between the lumped parameter model (LPM) and FEM**



**(a) Acceleration**



**(b) Displacement**

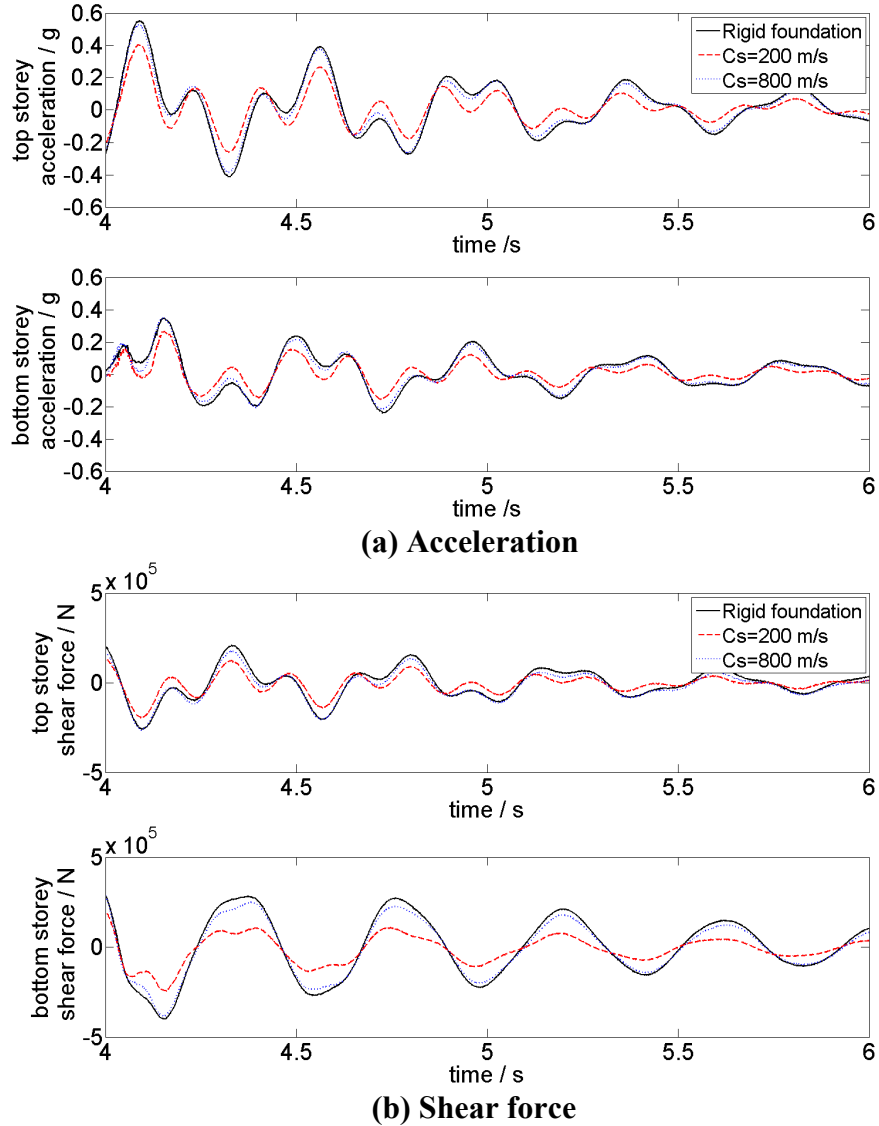


**(c) Shear force**

**Fig. 12 Results comparison between RTDHT and FEM**

## 5.4 SSI effect on Different Types of Soil

To illustrate the characteristic of soil-structure interaction, the dynamic response of the same two-storey structure founded on soft soil ( $C_s=200$  m/s) and hard soil ( $C_s=800$  m/s) foundations are compared. The comparison of the responses is shown in Figure 13.



**Fig. 13 Structure response comparison under different soil conditions**

The acceleration comparison in Figure 13a shows that the response under rigid foundation is bigger than the response under soft soil. The peak of the top storey acceleration under rigid foundation is 0.55 g but 0.40 g under soft soil. The reduction is thus, 27.3%. For the bottom storey, the peak is reduced from 0.35 g under rigid foundation to 0.27 g under soft soil for a reduction of 22.9%.

The shear forces comparison is shown in Figure 13b. The peak of the top storey shear force is reduced from 260 kN under rigid foundation to 196 kN under soft soil. The reduction is 24.6%.

The peak of the bottom storey shear force under rigid foundation is 399 kN, while under soft soil condition it is reduced to 245 kN; the reduction is 38.6%.

Comparison of rigid foundation results with hard soil ( $C_s=800$  m/s) results shows that the effect of SSI under hard soil is not very obvious. The structure responses are almost the same as the responses under rigid foundation.

It is concluded that the soil-structure interaction increases as more flexible the soil becomes. On the other hand, it will be negligible for a flexible structure founded on firm soil.

## 6. CONCLUSION

Lumped parameter model of soil is introduced into the numerical substructure of the RTDHT system. By doing this, the radiation damping of the infinite foundation can be simulated in the shaking-table tests. The test of a two-storey structure considering the soil-structure interaction is conducted. The following conclusions are obtained:

1). The lumped parameter model used in this paper can give a high accuracy approximation of the infinite half space elastic foundation.

2). Comparing the SSI-RTDHT test results with the FEM results shows that SSI-RTDHT can produce satisfying results in SSI analysis.

3). Comparing the tests results under rigid foundation, hard soil foundation, and soft soil foundation show that soil-structure interaction can affect the response of the structure: softer the soil is, more obvious the SSI effect becomes.

Using the idea of RTDHT, the infinite soil foundation model calculation and superstructure testing are combined together. Radiation damping of the infinite foundation can be included in shaking-table tests.

## ACKNOWLEDGEMENT

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