

SHAKING TABLE MODEL TESTING ON DYNAMIC SOIL-STRUCTURE INTERACTION SYSTEM

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SUMMARY

Shaking table model tests on dynamic Soil-Structure Interaction (SSI) system are described in this paper. In the design and fabrication of the test models, the similitude formulas and similitude factors of all physical quantities are studied. Through controlling the ratio between diameter of the container and plan size of the structure, and using a flexible container, the box effect resulted from the boundary conditions is minimized. The simulation design of soil boundary is proved considerably effective in present SSI tests. Nine specimens are designed and fabricated, including 5 pile foundations, 3 box foundations and 1 fixed base. Single column with mass block on top and 12-story cast-in-place R.C. frame model are used as superstructure, and Shanghai soft soil is employed as model soil. The test results are analyzed and simulated in detail with ANSYS program. The simulation results show that the three-dimensional finite element model is proper for the analysis of SSI problem. In the simulation analysis, commonly used equivalent linearity model is chosen to consider the non-linearity of soil, and the changing-status non-linearity on soil-structure interface is considered by surface-to-surface contact element. The flexible soil container and influence of gravity are also simulated properly in the analysis.

Key Words: Soil-structure interaction, Shaking table, Model testing, Nonlinear behavior.

INTRODUCTION

In the last 30 years, the effect of SSI on earthquake response of structures has attracted an intensive interest among researchers and engineers. Most of these researches focus on theoretical study and analysis, while less has been done on the experimental study. More importantly, many theoretical outcomes have not been verified to achieve a general accuracy for practical use. In the recent decade some countries such as Japan and America have set a start to carry out site tests and shaking table model tests on dynamic SSI system (References [1]~[8]). With the development of model similitude theory and structural seismic testing technology, shaking table model test has played more and more important role

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on the research of SSI. However, this kind of test is rather difficult due to its complexity. This paper presents the shaking table model tests on dynamic SSI system in detail, including similitude design of test models, simulation of soil boundary condition, design and fabrication of test models, arrangement of measuring points and test loading schedules. The summary of test results is presented briefly as well. Based on shaking table model tests and combining general-purpose finite element program ANSYS, three-dimensional finite element analysis on soil-structure test is also described in this paper.

SHAKING TABLE TESTS AND RESULTS

Similitude Design of Models

The reliability of model tests depends on whether the model can represent the real behavior of the prototype system. In the shaking table model test on SSI, similitude law should also be taken into account. However, only similitude design of superstructure and foundation is generally taken into account in former tests, while the similitude simulation of soil is often ignored due to the complexity and particularity of soil (see Aso [1]). A certain kind of material, such as sand, is often selected as soil material. So there is rather large difference between model and real SSI system. In order to reproduce the real behavior of prototype furthest, similitude design of the test model is performed herein. The test objectives and arrangement are shown in Table 1.

Phase	Test Code	Model Scale	Soil	Foundation	Superstructure	Test Content
	FF20S	1/20	Uniform Soil	-	-	FF
	FF20	1/20	Uniform Soil	-	-	FF
	FF10	1/10	Uniform Soil	-	-	FF
	PC20S	1/20	Uniform Soil	Pile	Column with Mass	SPSI
I	PC20	1/20	Uniform Soil	Pile	Column with Mass	SPSI
	PC10	1/10	Uniform Soil	Pile	Column with Mass	SPSI
	BC20	1/20	Uniform Soil	Box	Column with Mass	SBSI
	BC10	1/10	Uniform Soil	Box	Column with Mass	SBSI
_	PS20	1/20	Layered Soil	Pile	12-Story Frame	SPSI
	PS10	1/10	Layered Soil	Pile	12-Story Frame	SPSI
	BS10	1/10	Layered Soil	Box	12-Story Frame	SBSI
	S10	1/10	-	-	12-Story Frame	FS

Table 1 Test Objectives and Arrangement

Note: (1) Test Content: FF--Test of free field; SPSI--Test of soil-pile foundation-superstructure interaction; FS--Test of a frame structure on the fixed base; SBSI--Test of soil-box foundation-superstructure interaction; (2) When a single column with mass block on its top is used as superstructure, the mass block is adjusted in the test.

To study the seismic characteristics and response of the dynamic SSI system, the similitude design of test models is based on the following principles.

(1) The same similitude relation is applied to soil, foundation and superstructure. (2) Distortion of gravity is permitted. The method of adding additional weight is not adopted in present study in that it is almost impossible to be realized in soil and pile foundations. (3) Parameters of dynamic loads are controlled to meet the performance requirements of shaking table. (4) Requirements of construction and capacity of equipment must be accessible in laboratory.

Consequently, non-gravity model with similitude rules controlled materials is adopted in present test. Similitude formulas and similitude factors of all physical quantities are induced from Bockingham π theorem (see Table 2). A 12-story cast-in-place frame is used as prototype superstructure, and Shanghai soft soil is selected as prototype soil. Thus, the prototype system can be regarded as a typical small high-

rise building system of Shanghai. The scales of models are 1/10 and 1/20, respectively. The similitude factor of mass density is 1, and the similitude factors of elasticity modulus for both soil and structure are 1/4.

Table 2 Dynamic Similitude Relation of Models						
	Physical Quantity	Similitude	Similitude Factor			Domork
	Physical Quantity	Formula	Soil	Foundation	Superstructure	nemark
	Strain <i>e</i>	S _• =1.0	1	1	1	CP
	Stress σ	<i>S</i> ₀ = <i>S</i> _E	1/4	1/4	1/4	
Material	Young's modulus <i>E</i>	SE	1/4	1/4	1/4	CP
Parameter	Poisson ratio μ	S _µ =1.0	1	1	1	
	Density <i>P</i>	S,	1	1	1	CP
	Length I	S ₁	1/10	1/10	1/10	CP
Geometry	Area S	$S_S = S_I^2$	1/100	1/100	1/100	
Parameter	Linear displacement X	$S_X = S_I$	1/10	1/10	1/10	
	Angular displacement β	S _ℓ =1.0	1	1	1	
Load	Concentrated force P	$S_P = S_E S_I^2$	1/400	1/400	1/400	
Parameter	Area load q	$S_q = S_E$	1/4	1/4	1/4	
	Mass <i>m</i>	$S_m = S_\rho S_l^3$	1/1000	1/1000	1/1000	
	Rigidity k	$S_k = S_E S_I$	1/40	1/40	1/40	
Dynamic	Time t	$S_t = (S_m / S_k)^{1/2}$	0.2	0.2	0.2	CPDL
Parameter	Frequency f	$S_{f=1/S_t}$	5	5	5	CPDL
	Damping <i>c</i>	$S_c = S_m / S_t$	0.005	0.005	0.005	
	Velocity v	$S_v = S_t / S_t$	0.5	0.5	0.5	
	Acceleration a	$S_a = S_t / S_t^2$	2.5	2.5	2.5	CPDL

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

Simulation of Soil Boundary Condition

The simulation of soil boundary condition plays a key role in the validity of the test design. In shaking table tests, model soil cannot be held in an infinite dimension box. Due to wave reflection on the boundary and variation of system vibration mode, certain error so called 'box effect' will occur in test results. In order to reduce the box effect, a flexible container and the proper constructional details are designed in model test, and the ratio between ground plane diameter D and structure plane size d is taken as 5 by controlling the size of the structure plane (Lou [9]). The cylindrical container is 3000mm in diameter and its lateral rubber membrane is 5mm in thickness, and the reinforcement of $\Phi 4@60$ is used to strengthen the outside of the container. The lateral side of the cylinder is fixed with the upper ring plate and the base plate by bolt. The upper ring plate is supported by four columns fixed on the base plate. Height adjustable screw rod is installed on the column to adjust the upper plate to horizontality and adjust the cylinder to a proper state. A universal joint is installed on the column top to enable the ring plate to displace laterally. The base plate is made of steel plate. In order to minimize relative slip between the soil and the container on the base surface, a kind of crushed rock is bonded to the base steel plate by epoxy resin to make the surface rough.

Fig.1 shows the flexible container used in present test. In order to verify the rationality of the flexible container simulation method, three free field-shaking table tests are performed. Shanghai soft soil is used as the model soil. The soil in test FF20s, FF20 and FF10 is 0.8m, 0.8m and 1.6m in depth respectively. The container with soil is excited with two levels of intensity of El Centro earthquake record and Shanghai artificial wave.

Fig.2 shows the curves of relation between the amplification coefficient and the boundary distance (the amplification coefficient is the ratio between the acceleration peak value of each measuring point on the soil surface and the acceleration peak value excited at the container base). In the figure, it is obvious that the boundary influence exists in the region near the boundary, whereas, the acceleration amplification coefficient of each measuring point tends to uniformity in the region certain distance off the boundary, which indicates that the boundary influence becomes insignificant. Fig.3 shows the acceleration time-history curves of soil surface measuring point S19 and S6 under EL1 excitation. Measuring point S19 is in the center, while measuring point S6 is 0.6m off the boundary. The consistency of two acceleration time-history curves demonstrates that the boundary influence on measuring point S6 is very small. The same phenomenon is found when comparing the response of corresponding measuring points in soil at a certain depth. This indicates that the design of the container is rational in minimizing the box effect, although the boundary condition still brings some influence on the edge region.



Design and Fabrication of the Models

Having taken the test purpose, test condition, model material, and construction technique into account, 3x3-group piles foundation and box foundation are adopted in present tests. Foundation and soil are designed according to the similitude relation. The scaling factors of two models are 1/10 and 1/20, respectively.

The tests are implemented in two phases (see Table 1). In the first phase, a single column with mass block on top is used as superstructure. In order to simulate different superstructures, the mass block on the column top is adjusted to change the dynamic characteristics. Four kinds of mass blocks, namely, A (80kg), B (160kg), C (320kg) and D (480kg), are used in 1/10 model tests; while A (10kg), B (20kg), C (40kg) and D (60kg) are used in 1/20 model tests. In the second phase, a 12-story reinforced concrete



frame structure with single span is employed to simulate the superstructure. The layout and reinforcement details of the models are shown in Fig.4.

Fig.4 Model Layout and Reinforcement Detail

The superstructure and foundation is made of micro-concrete and fine steel bar. Shanghai soft soil is used as model soil. Properties of all materials are measured by material tests before the shaking table test. Fig.5 is typical $G_d/G_0 \sim \gamma_d$ and $D \sim \gamma_d$ curves of Shanghai soft soil in this test, where G_d, G_0, D, γ_d is dynamic shear modulus, initial dynamic shear modulus, damping ratio and shear strain. Nonlinear properties of soil can be seen clearly in the figure.

Arrangement of Measuring Points

Accelerometers and strain gauges are used to measure the dynamic response of the superstructure, foundation and soil. Pore pressure gauges are embedded in soil to measure the change of pore pressure. Soil pressure gauges are used to measure the contact pressure between piles and soil. The measuring point arrangement of test PS10 and test BS10 in the second phase is shown in Fig.6. Those of other tests are similar to these and omitted here.



Fig.6 Sketch of Measuring Point Arrangement

Test Loading Schedules

Unidirectional (horizontal X direction) excitations are applied in the first test phase, including El Centro earthquake record (1940, N-S component), Shanghai artificial wave (SHW2), and sinusoidal wave. Both unidirectional (horizontal X direction) excitations and bi-directional excitations (horizontal X direction) and vertical Z direction) are employed in the second test phase, including El Centro earthquake record, Shanghai artificial wave, and Kobe earthquake record. Acceleration peak value, which is determined according to the corresponding epicentral intensity in the seismic code of China, and time interval, are adjusted to accord with the similitude relation.

Table 3 is the loading schedules in the first test phase. Under the condition of a certain acceleration peak value, various waves are inputted when mass block is A. Repeat the process when mass block is increased to B, C, and D in turn. Then, the acceleration peak value is increased to next grade. Before and after the altering of mass block, White Noise with small amplitude is inputted to observe the change of dynamic

characteristics of the soil-structure system. Due to the limitation of paper length, the loading schedules of the second test phase are omitted here.

Table 3 Test Schedules							
Excitation Waya	Excitation Code			Peak Valu	Peak Value of Acceleration (g)		
	Mass A	Mass B	Mass C	Mass D	Prototype	1:20	1:10
						Model	Model
White Noise	WN1	WN3	WN5	WN7		0.07	0.07
El Centro Wave	EL1a	EL1b	EL1c	EL1d	0.1	0.488	0.244
Shanghai Artificial	SH1a	SH1b	SH1c	SH1d	0.1	0.488	0.244
Wave							
Sinusoidal Wave	F1a	F1b	F1c	F1d	0.1	0.488	0.244
White Noise	WN2	WN4	WN6	WN8		0.07	0.07
White Noise	WN9	WN11	WN13	WN15		0.07	0.07
El Centro Wave	EL2a	EL2b	EL2c	EL2d	0.2	0.976	0.488
Shanghai Artificial	SH2a	SH2b	SH2c	SH2d	0.2	0.976	0.488
Wave							
White Noise	WN10	WN12	WN14	WN16		0.07	0.07
White Noise	WN17	WN19	WN21	WN23		0.07	0.07
El Centro Wave	EL3a	EL3b	EL3c	EL3d	0.3	1.464	0.732
Shanghai Artificial	SH3a	SH3b	SH3c	SH3d	0.3	1.464	0.732
Wave							
White Noise	WN18	WN20	WN22	WN24		0.07	0.07

Summary of Test Results

The container and model fixed on the shaking table is shown in Fig.7. The summary of some important findings from SSI system tests is presented as follows. It should be noted that only parts of test evidences are described here due to the limitation of paper length.

(1) The phenomena of shaking table tests show that the tests reproduce the earthquake damage on the whole. The settlement of the structure occurs during excitation. The incline of structure is little for SSI system with pile foundation. However the settlement and incline are much larger for SSI system with box foundation, and sometimes the incline is so large as to turnover. The degree of settlement and incline has close relation with characteristics of soil. No crack appears on the superstructure when excitation is slight, and the crack on the superstructure is slight even when the excitation is moderate. There is not any crack on box foundation, while many horizontal bending cracks are distributed along the pile on pile foundation, system.

(2) Comparison of dynamic characteristics shows that the natural frequency of the SSI system under the soft soil condition is lower than that of the structure on the fixed base by 20~60 percent, and the damping ratio of the system is 1.5~6 times of that of the structure material. Table 4 shows the frequency and damping ratio of some test results measured in test PS10.

(3) Due to the effect of SSI, the mode shape of the SSI system under the soft soil condition is greatly different from that of the structure on the fixed base. Fig.8 shows the mode shape curves of each test phase measured in test PS20. It is obvious from this figure that there are rocking and swing at the foundation.

(4) It is commonly considered that field soil will magnify vibration transformed from bedrock. But it is observed in tests that soft soil can filter and isolate vibration. Taking test PC10 as example, S7, S6, S5 are measuring points in soil, with the same plane location while depth increase gradually. S8 is a measuring point on container base. Fig.9 shows acceleration time-history and corresponding Fourier spectra of each

measuring point under EL1a excitation. When the vibration wave travels upwards, soft soil filters most of the high-frequency components, leaves behind the low-frequency components and minimizes the peak value of the acceleration. Similar results can be observed in each test under each excitation.

	•	SSI	System	Freq. of
No.	Excitation Code	Freq. (Hz)	Damping Ratio	Structure on Fixed Base(Hz)
			(%)	
1	1WN	2.643	9.816	3.273
10	10WN	2.139	12.586	3.147
16	16WN	1.636	16.616	2.895
22	22WN	1.384	15.964	2.644
28	28WN	1.258	17.198	2.267
34	34WN	1.133	21.770	2.014
40	40WN	1.133	18.636	1.762

Table 4.	Frequency	and Damning	Ratio	(Test PS10)
\mathbf{I}	ricquency	and Damping	nauv	

(5) Due to the vibration feedback from the superstructure, the spectral components of the foundation motion are changed and the motion of foundation is different from that of free field. The motion of foundation in SSI system is smaller. The components around the natural frequency of the system are enhanced and some frequency components are weakened.

(6) The whole system response of the acceleration amplitude is in 'K' shape in height (see Fig.10). The response of the foundation top is smallest, while the response of the superstructure is larger than that of the



Fig.7 Photograph of Container and Model (The Second Test Phase)

foundation top. With increasing of the input acceleration, the amplification factors of the acceleration peak value are reduced due to the non-linearity of soil.

(7) The acceleration response at the top of the superstructure consists of rocking and swing of the foundation, and the deformation of the structure. Fig.11 shows the acceleration time-history and corresponding Fourier spectra of each component when EL1a excites in test PC20S. The total



(a) The First Mode Shape (b) The Second Mode Shape Fig.8 Mode Shape Curves in Test PS20



Fig.10 Distribution of the Amplification Factors of the Acceleration Amplitude (Test PC20S, Mass A, Excited by Different El Centro Wave)



Fig.11 Acceleration Time-history and Corresponding Fourier Spectra of Each Component (Test PC20S, Excitation EL1a)

acceleration \ddot{u} , rocking component $H\ddot{\theta}$, swing component \ddot{u}_g and deformation component of the superstructure \ddot{u}_e are shown from above to below in the figure. It can clearly be seen that the rocking and the swing components are the main parts of the acceleration response. Therefore, it is necessary to take the rocking and the swing into account in analysis when the foundation is in soft soil.

(8) The distribution of the strain amplitude along the pile is the shape where the large strain is at the top of the pile and the small strain is at the tip of the pile (see Fig.12). And the distribution of the contact pressure on the pile-soil interface is the shape where the small pressure is at the top and the large pressure is at the bottom (see Fig.13).



Fig.12 Distribution of the Strain Amplitude on Pile (Test PS20, Excited by Different El Centro Wave)

Fig.13 Distribution of the Contact Pressure Amplitude on the Pile-soil Interface (Test PS20, Excited by Different El Centro Wave)

(9) The response of the system under the excitation of Shanghai artificial wave is obviously larger than that under the excitation of El Centro wave and Kobe wave.

(10) Vertical excitations have little effect on the responses of the dynamic soil-structure interaction.

Through these tests, abundant experimental data are obtained, which can be used to verify the results of theoretical and analytical research, and improve or put forward better computational models and analytical methods. The present work provides the basis for further research.

COMPUTER SIMULATION ON THE TESTS

Modeling Method

Simulation of flexible container

The lateral rubber membrane of the container is meshed by shell element in modeling. The base plate of the container is fixed with shaking table by bolt, and proper measures are taken to make the surface of the base plate rough in test. So the relative slip between the soil and the bottom of the container can be ignored and the bottom of soil can be considered as fixed in modeling. The reinforcement loops outside the container are used to provide radial rigidity and permit soil to deform as horizontal shear layer in test. In modeling, the reinforcement loops can be considered that the nodes along the container perimeter with same height have the same displacement in excitation direction (X-axis direction of shaking table), which can be realized by coupling of degrees of freedom in ANSYS program.

Dynamic constitutive model of soil and simulation of material nonlinearity

In this paper, equivalent linearization model of soil is adopted. In calculation, assume a pair of shear

modulus G_{d1} and damping ratio D_1 for each layer of soil, calculate corresponding effective shear strain Y_{d1} , then find out corresponding shear modulus G_{d2} and damping ratio D_2 in the relationship curves of $G_d/G_0 - Y_d$ and $D - Y_d$, respectively (see Fig.5). The curve of $G_d/G_0 - Y_d$ denotes the relationship between the effective shear strain Y_d and the ratio of shear modulus G_d and the initial shear modulus G_0 , while the curve of $D - Y_d$ denotes the relationship between the damping ratio D and the effective shear strain Y_d . Repeat the above steps until the differences between twice results of the shear modulus and the damping ratio are in allowable range. In calculation, 0.65 times of the maximal shear strain is taken as Y_d (Wang [10]).

In ANSYS program, there is a kind of parametric design language named APDL, which is a scripting language. Users can use it to automate common tasks or even build models in terms of parameters. The equivalent linearity model is realized in ANSYS program by using the APDL, and the calculation of material nonlinearity is realized automatically.

Simulation of the change-status nonlinearity on soil-structure interfaces

Due to the material characteristics difference between the soil and concrete, there are sliding and splitting phenomena on the soil-structure interface when interface stress increases to a certain level, and the gappy interface could reclose under a certain load condition. Earlier researchers adopted interface elements such as Goodman element, lamina element and lamina soil element to simulate this changing-status nonlinearity on the interfaces of soil and structure (foundation). In ANSYS program, contact analysis is realized by overlaying a thin layer of elements upon the contact interface of analysis model. The soil surface is taken as contact surface, while the structure (foundation) surface as target surface due to its larger rigidity. Contact elements and target elements are formed on the contact surface and the target surface, respectively. Then, the corresponding contact elements and target elements are taken as one contact pair by defining the same real constant number. Rational parameters are chosen to simulate the status of sticking, splitting or reclosing on the soil-structure interface.

Damping model

In SSI system, the damping ratio of soil is usually larger than that of superstructure. So the damping ratio of soil and superstructure should adopt different value. A material-damping ratio inputting method is given in ANSYS program. Using this method, different damping ratio can be inputted according to different material. Using the material-damping input method can take different damping ratio of soil and superstructure into account.

Consideration of gravity

The initial stress produced by gravity has large influence on the status of contact. Great error will occur if the gravity is not taken into account in the dynamic calculation. The contrast of contact pressure time-history on the center of the foundation bottom between taking gravity into account and not taking is shown in Fig.14. It is



Fig.14 Contact Pressure Time-history on Center of Foundation Bottom (BC20 Test Model)

obvious that the contact pressure of taking gravity into account is larger than that of not taking gravity into account. And separation between the soil and the foundation bottom occurs when not taking gravity into account, while no separation occurs when taking gravity into account that agrees with the test results.

In this paper, gravity is applied as a dynamic load in calculation. Before the seismic wave is applied, gravity is applied on the system as a vertical acceleration field and transient analysis is carried out. After the response drove to a constant value, the seismic wave and gravity are applied to the system together and the transient analysis continued without pause. The dynamic response can be obtained by subtracting the constant value from the total response.

Fig.15 shows meshing of PS10 test model satisfying the above modeling principles.

Verification of the Model

Some acceleration time-history curves of computational results and test results of the PS10 test are given in Fig.16. The code EL1 in the Fig.16 denotes the excitation of El Centro wave, whose peak value of acceleration is 0.093g. It shows that the acceleration time-history curves of corresponding points are approximately coincident. It's verified that the computational model is rational and appropriate for research on SSI.



Fig.15 Meshing of test model (PS10)



Fig.16 Comparison between calculation and test result (PS10 test model, under excitation of EL1)

Calculation Results of PS10 Test Model

The effect of the material nonlinearity of soil and the changing-status nonlinearity of soil-structure interface on computational results

In order to analyze the effect of the material nonlinearity of soil and the changing-status nonlinearity, the computational analysis is carried out under following three conditions in this paper. (1) Without any nonlinearity; (2) Just taking the soil nonlinearity into account; (3) Taking both the soil nonlinearity and the changing-status nonlinearity into account.

The comparison of acceleration response between the above-mentioned condition (2) and condition (1) is shown in Fig.17. The code EL2 in the Fig.17 denotes the excitation of El Centro wave, whose peak value of acceleration is 0.266g. Point A7 in Fig.17 is the central point on the top of the frame (as shown in Fig.6), and point S8 is on the surface of soil, which is 0.6m away from the container boundary. It is obvious that the acceleration response remarkably decreases when the soil nonlinearity is considered.

The comparison of acceleration response between the above-mentioned condition (2) and condition (3) is shown in Fig.18. It shows that the acceleration response of the point A7 changes obviously, because the status between the group piles and the soil, and the rotation of cap, which has large effect on the response of superstructure, can be simulated factually by the contact analysis. The acceleration response of point S8 hardly changes, because the changing-status nonlinearity on soil-structure interface has little influence on the point S8, which is far away from the foundation.

The other findings of the calculations demonstrate that large error will occur when the material nonlinearity of soil and the changing-status nonlinearity of the soil-structure interface are not considered in calculation of soft soil.



Fig.17 Comparison of computational analysis between linearity and soil nonlinearity (PS10 test model, under excitation of EL2)



Fig.18 Comparison of computational analysis between soil nonlinearity and contact analysis (PS10 test model, under excitation of EL2)

Contact analysis on soil-pile interface

In order to learn the response characteristics of each pile of the group piles foundation, the normal strain along the pile, contact pressure and sliding between the soil and the piles are outputted in calculation process. The plan of group piles is shown in Fig.19, in which No.2 pile is a middle pile at side row, No.3 pile is at the corner, and No.5 pile is the center pile.

Distribution of the strain amplitude on pile

Fig.20 is the distribution of the strain amplitude along the No.2, No.3 and No.5 piles under excitation of EL2. The distribution of the strain amplitude along the pile is the shape where the large strain is at the top of the pile and the small strain is at the tip of the pile, which consist with the crack distribution observed in the test. The strain amplitudes of different



Fig.19 Plan of group piles

piles have large difference at the top of piles, while little difference at the tip of piles. The strain amplitude of the pile at the corners is larger than the middle pile of side row and the center pile.

Distribution of the contact pressure on the pile-soil interface

Fig.21 is the contact pressure time-history at the position 80mm below top of No.3 pile. It can clearly be seen that the condition of zero contact pressure exists, which demonstrates that the phenomena of separation and reclosing occur between soil and pile foundation. Fig.22 is the distribution of the contact pressure amplitude between soil and the No.2, No.3 and No.5 piles under excitation of SH2. The code SH2 denotes Shanghai artificial wave, whose peak value of acceleration is 0.266g. The distribution of the contact pressure on the soil-pile interface is the shape where the smaller pressure is at the middle of pile and the larger pressure is at both the top and the bottom.



Sliding on the soil-structure interface

Fig.23 is the distribution of the sliding amplitude between soil and the No.2, No.3 and No.5 piles under excitation of EL2. The sliding amplitude on the soil-pile interface of the No.5 pile is smaller than that of the No.2 and No.3 pile. The pile's sliding of the side row is larger than that of the middle row along the vibration direction.

CONCLUSIONS

In this paper, shaking table model tests on dynamic SSI system are described, and some important findings from the present tests are concluded. Combining general-purpose finite element program ANSYS and based on shaking table model tests, three-dimensional finite element analysis on SSI system has been

carried out. By comparison analysis between the calculation and the tests, it is verified that the modeling methods are rational and suitable for the numerical analysis of dynamic SSI system.



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