Effect of soil-pile-structure interaction on seismic behaviour of nuclear power station via shaking table tests

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Abstract: To better understand the characteristics of the seismic response of nuclear power stations with fixed-base and pile-raft foundations, a series of shaking table tests were performed. The shaking table tests included three cases: free-field soil, fixed-base structure without soil, and superstructure with a pile-raft foundation embedded in soil. One white noise excitation and three earthquake motions with different earthquake intensity were selected as the ground motion to identify the seismic response of the structure, raft, piles, and soils. The effects of earthquake intensity, earthquake frequency, soil-pile-structure interaction, and soil nonlinearity on the system's dynamic responses were analysed. The test results show that the natural frequency and the damping ratio of the superstructure for the pile-raft foundation and the fixed-base structure are different, owing to the soil-pile-structure interaction effect. The acceleration amplification ratio of the fixed-base superstructure shows a significant higher value than that of the superstructure with a pile-raft foundation. The average peak acceleration ratio of the raft for the pile-raft foundation (raft/soil surface) is 1.2. Under the long period wave excitations, the bending moment of the pile is greater than that of short-period wave excitations, and the peak bending moment occurs at the pile head. The vertical and horizontal displacement and residual displacement increases with the earthquake intensity. These observations suggested that the design of nuclear power station with a pile-raft foundation embedded in clay cannot be simplified as a fixed-base structure. Moreover, adopting pile-raft foundation for nuclear power station will extend the choice of finding suitable construction sites for the nuclear power stations, and the test results could provide references for engineers.

Keyword: Shaking table test; fixed-base structure; pile-raft foundation; nuclear power station; soil-pile-structure interaction

1 Introduction

The damage of buildings under earthquake imposes great threats to lives and properties, especially for nuclear power stations, which could induce catastrophic secondary disasters. Many nuclear power stations with fixed-base raft foundation are built in rock sites to limit both settlement and differential settlement requirements. With limited number of rock sites and other considerations in the process of nuclear power station construction, alternative methods to construct nuclear power stations in non-rock areas are worth studying. Pile-raft foundation has been widely adopted in supporting various structure systems in soft soil areas. The pile-raft foundation under earthquake load endures two different forces: firstly, the inertial forces from the superstructure, and secondly, the ground deformations induced by the seismic load. In the process of pile-raft foundation design, the soil, the foundation, and the superstructure are supposed to be calculated as an integrated structural system [1]; however, the calculation method is not provided by related codes. Studies on the different characteristics of the seismic response of nuclear power station with a fixed-base or with a pile-raft foundation are critical in identifying the impacts of foundation type on the seismic response of structures, understanding the soil-pile-structure interaction effect and satisfying the high safety requirement of in the design of power stations.

Many studies on the seismic response of structures with fixed-base raft foundations were carried out by 1 g shaking table tests [2-5], in which, raft or load-bearing elements (column and shear wall) were attached to a shaking table to test the seismic response of the structures. Researches have been done to analyse the dynamic characteristics of structures considering the soil structure interaction (SSI), the topic has gained significant attention [6-11]. In some cases, the forces were applied to the pile head or on the raft directly, without superstructure [12-13] . For example, Mostafa and EI Naggar [14] proposed a method to analyse the dynamic lateral response of pile group under harmonic excitation. Basack and Nimbalkar [15] developed a numerical model based on a two-dimensional (2D) dynamic finite-element (FE) approach.. For cases when superstructures were considered, the seismic response of the pile-soil-structure interaction were studied experimental [16-19] and numerically [20-25]. As n-g centrifuge experiment, liquefication study and numerical simulation beyond the scope of this paper, so they are not discussed in this paper. Durante et al. [26], Mohammad Hassan Baziar et al. [27] and Roy et al. [28] simplified the superstructure as an oscillator using shaking table test. They found that SSI produced a small amount of in period elongation, superstructure frequency strongly affects the pile-raft foundation frequency, and shorter period superstructure system exhibits significant lengthening of period. There is a general agreement [29-32] that limited effort was attempted for systems modelled with detailed superstructure. Omar et al. [33] and Aslan et al. [34] found that the type of foundation is a major contributor to the seismic response of building with SSI, and the foundation experiented a considerable amont of rocking dissipated much more earthquake than other types of foundation. Rajib et al. [35] studied the effect of SSI under different intensity sine wave excitations and found that the base shear of the structure may significantly increase considering SSI. Wu et al. [36] also used sine waves to study the SSI effect in coral sand and found that the horizontal displacement of the superstructure, bending moment of the columns and piles in coral sand site are smaller than that in the quartz sand site. Liang et al. [37] carried out multiple shaking table tests to study the transverse response of pile group foundation, and also proposed that it is not appropriate to simulate the superstructure with a single degree of freedom system. Nevertheless, the studies analysing the effect of SSI on seismic behaviour of a nuclear power station in clay soil under earthquake excitations are very few and await further investigation.

In this study, a series of shaking table tests were designed to understanding the seismic response of nuclear power stations with a pile-raft foundation on clay, and comparing the different characteristics between a fixed-base and a pile-raft foundation structure. The effects of earthquake intensity, earthquake frequency, soil-pile-structure interaction, and soil nonlinearity on the dynamic response of the system are considered in the tests.

2 Test set-up

2.1 Test facility

The shaking table tests were carried out in the Key Laboratory of Concrete and Prestressed Concrete Structures of Ministry of Education, Southeast University. The dimension of the shaking table is 4 m ×6 m (width × length), and the bearing capacity is 25 ton under the maximum acceleration of 1.5 g. The maximum displacement of the shaking table is ±250 mm, and the shaking frequency arranges from 0.1 Hz to 50 Hz. The laminar shear model box is 2 m long, 2.0 m wide, and 1.3 m high.

2.2 Model configuration

The design of the shaking table test model was based on the scaled (one-tenth) third generation nuclear power station in China, the same as a series of dynamic centrifuge test model carried out in the Geotechnical Centrifuge Modelling Laboratory, Tongji University, and the relevant papers are currently under preparation. Although studying the behavior of a scaled 1-g model do not provide exact data for practicing engineering design, the behavior of a scale model allows better understanding of the fundamental mechanics hence it gives instights to nuclear power station design. Tab.1 summarised the scaling factors applied to the shaking table tests. Based on the size and the maximum acceleration capacity of the shaking table, the scaling factors for geometry and acceleration are 1/25 and 3, respectively. The aluminium pipe pile was selected, which is the same material as the mentioned dynamic centrifuge test, so the scaling factor of modulus of elasticity is 1. The aluminium pile cannot satisfy the scaling factor for the elastic modulus and the density simultaneously. As calculated in Tab. 1, the density and mass scaling factors are 12.5 and 5.33×10^{-4}. It is challenging to satisfy the density and mass scaling factors for the aluminium pipe pile [38], and no extra mass was added in the test.

2.3 Test soil

The soil used in the test is natural clay soil in Nanjing. Measures were taken to ensure the uniformity of the soil in the laminar shear model box, and the soil was placed in it layer by layer. The total height of the soil is 1.2 m. The density of the soil is 1780 kg/m^3, and the water content is 23.5 %. The liquid limit and plastic limit are 46.6% and 28.8%, respectively. The Poisson's ratio is about 0.35 based on the empirical value recommended by Geological Engineering Handbook in China [39]. The fundamental period of the soil is about 0.1 s, and the height of the soil in the laminar shear model box is 1.2m. So the shear wave velocity can be calculated as 48 m/s. Based on the consolidated drained triaxial tests of the clay soil the cohesion and internal friction angle are 7.2 kPa and 21°, respectively. To obtain the nonlinear soil properties, resonant column tests were carried out using the Stokoe resonant column apparatus. The dynamic shear modulus and damping ratios results of the soil under confining pressure of 100 kPa, 200 kPa and 300 kPa are shown in Fig. 1 and Tab. 2.

2.4 Test system

The laminar shear model box was firmly mounted on the shaking table using a bolt connection. The laminar shear model box filled with a height of 1.2 m soil without structure and piles were utilised to identify the effect of the laminar shear model box boundary on the dynamic characteristic of soil. The shaking table's dimension is 4 m × 6 m, which is capable of placing the laminar shear model box and the fixed-base nuclear power station model on the table. The fixed-base nuclear power station was fixed on the shaking table by a steel plate. Thus, the excitation of the fixed-base nuclear power station model and the free-field soil in the laminar shear model box were carried out together. This excitation was named case A, which included the free-field soil case (case A-1) and fixed-base model (case A-2), as shown in Fig. 2. The accelerometers installed at the top, middle, and raft of the structure are A-1a, A-2a, and A-3a. Fig. 3 plots the schematic figure of case A, including case A-1 and case A-2. There are five accelerometers (A-4a~8a) installed in the soil. The distance from the accelerometers A-4a and A-5a to the laminar shear model box boundary are 600 mm and 300 mm, respectively. The comparison of those two accelerometers recorded data can identify the boundary effect. Furthermore, the characteristic of acceleration along the depth of soil can be generated from the recorded data of A4a, A6a, A7a and A8a.

After case A, the nuclear power station model with pile-raft was constructed in the soil in the laminar shear model box, as shown in Fig. 4. The excitation of the model with pile-raft foundation was called case B. The location of the accelerometers, meters and strain gauges for structure with pile-raft foundation are demonstrated in Fig. 5. The accelerometers located at the top, middle, and raft of the structure are A-1b, A-2b, and A-3b. The locations of accelerometers A-4b, A-6b and A-7b were near the piles, and the depths of these accelerometers were the same as those of A-4a, A-6a, and A-7a as shown in Fig. 3(a). Accelerometers, A-5b and A-8b, were equipped at the surface and bottom of the soil. A steel beam is equipped at the laminar shear model box to hold the laser displacement meters. The laser displacement meter LS1 and LS2 are used to record the vertical (settlement) and the horizontal displacement of the structure, respectively. The locations of four instrumented piles (P1~P4) were shown in Fig. 5(a). P1 and P2 are located at the first (outside) row, and P3 and P4 are located at the second (inner) row. P1 is the corner pile. Each instrumented pile was equipped with twelve strain gauges at six heights along the pile shaft to monitor the strain of the pile and to generate the bending moment of the pile.

2.5 Applied ground motions and test program

One artificial seismic wave and two natural ground motions were adopted in the tests. The artificial seismic wave was designed based on the EUR soft design response spectrum, named YG. The other two natural ground motions are the Mexico City wave (MEX) and the El Centro wave (EL). Fig. 6 plots the time-history acceleration and response spectrum (RS) of the applied ground motions. Note that the applied ground motions shown in Fig. 6 are not scaled. The MEX wave has a long dominant period (2.03 s), and the EL wave has a short dominant period (0.52 s). For the artificial earthquake YG wave, the normalised response spectrum has a platform value of 0.31g and the corresponding characteristic period is from 0.14s to 0.39s. Those three motions were selected as the input ground motions because of their various dominant frequencies and response spectra characteristic periods. For better understanding the effect of earthquake intensities on the dynamic characteristics of the model, each motion was designed with three model excitation intensities of 0.3 g, 0.6 g, and 0.9 g, corresponding to the three prototype intensities of 0.1 g, 0.2 g, and 0.3 g respectively, as shown in Tab. 3.

The detailed test program is summarised in Tab. 3. Both case A and case B contained one 0.05 g White noise excitation to identify the dynamic characteristics of the soil, structure or the whole system, and three earthquake excitations with three different earthquake intensities. The prototype acceleration in Tab. 3 refers to the insitu acceleration. The model acceleration and the recorded acceleration are the designed input model ground acceleration based on the scaling law and the actual input acceleration of the shaking table, the difference is due to the limitation in calibrating the excitation control system of the shaking table.

3 Test results and discussion

3.1 Frequency and damping of the cases

The white noise excitation was used to identify the fundamental frequencies and damping ratios of the cases. Rational fraction polynomial (RFP) method was employed to extract the modal parameters (natural frequency and damping ratio) of the experimental model. The RFP method utilises the frequency response function (FRF) which is generated by an input signal and an output signal. Similar to the Laplace domain model [40], the dynamic frequency domain model, which is built using FRF, is employed. The form of the frequency domain model is exactly the same as the Laplace domain model, but with frequency response functions replacing transfer functions and Fourier transforms replacing Laplace transforms of the structural excitations and responses. Herein, the measurement of the FRF is the heart of modal analysis, which is defined as the Fourier transforms of the system response r(t) to the excitation u(t). Herein, the FRF is calculated by

$$H(f) = \frac{G_{ur}(f)}{G_{uu}(f)} \tag{1}$$

where $H(f)$ represent the FRF, $G_{ur}(f)$ is the cross-spectrum between the response $r(t)$ to the excitation $u(t)$, and $G_{uu}(f)$ is the power spectrum of the excitation $u(t)$. The measured FRF can be fitted using two polynomials, which is expressed as

$$H(\omega) = \frac{\sum_{k=0}^{2n} b_k (i\omega)^k}{\sum_{k=0}^{2n} a_k (i\omega)^k} \tag{2}$$

where a_k and b_k represent the coefficients of the polynomials for the numerator and denominator, and $\omega=2\pi f$. Accordingly, a linearised error function is defined as the difference between the measured FRF and this fitted model. The coefficients a_k and b_k can be evaluated by minimising the error function. Subsequently, the natural frequency and damping ratio can be derived readily from the fitted FRF. More detailed information regarding the RFP method can be found in Richardson and Formenti [41]. Note that the robustness of the RFP method has been verified by previous researches [42-43].

The fundamental frequencies and damping ratios of the fixed-base structure and the structure with pile-raft foundation are shown in Tab. 4. The fundamental frequencies in the middle of the superstructure for the fixed-base structure (case A-2) and for the structure with pile-raft foundation (case B) are 16.49 Hz and 10.05 Hz, respectively. Meanwhile, the damping ratios in the middle of the superstructure for case A-2 and for case B are 0.47% and 1.11%, respectively. The reason why the natural frequency of the superstructure of the pile-raft foundation is lower (damping is higher) than that of the fixed-base structure is because of the SSI between the soil-pile underneath the raft and the superstructure. However, the superstructure remains almost in elastic state during shaking table tests. For example, for case B with pile-raft foundation, both frequency and damping characteristics of superstructure at the top (A-1b), in the middle level (A-2b) and at the raft level (A-3b) are almost the same at around 10 Hz and 1 %.

3.2 Dynamic response of the soil

3.2.1 Dynamic response of the free-field soil

The peak soil surface acceleration, and the acceleration response spectrum ratios of the soil surface to the soil base were analysed to study the dynamic response of the free-field soil. Besides, the boundary effect of the laminar shear model box is evaluated during the tests.

The laminar shear model box filled with soil without the model was equipped at the shaking table. Two accelerometers (A-4a, A-5a), near and far from the laminar shear model box boundary, were located at the surface of the free-field soil to evaluate the boundary effect of the laminar shear model box and to calculate the frequencies and damping ratios of the free-field soil. Tab. 5 lists the fundamental frequencies and damping ratios calculated based on the recorded acceleration data using the RFP method under white noise excitation. The fundamental frequencies and damping ratios for those two accelerometers have a neglectable difference, which means the boundary effect on the soil is acceptable. The recorded peak acceleration value under test AW0 to AM3 for A-4a and A-5a is compared in Tab. 6. The difference between the acceleration value of A-4a and A-5a is about 5%, which also provides evidence that the boundary effect is neglectable.

Fig. 7 plots the relationship between PGA and soil surface acceleration for free-field soil. The slope of the dashed line in Fig. 7 is 1:1. The soil surface acceleration values all above the dashed line, which means the soil surface acceleration is higher than the PGA. The soil exhibit an amplification effect on seismic waves. The soil surface accelerations under the MEX excitation are higher than that under the EL excitation and the YG excitation for the MEX owning long period and may cause resonant during excitations.

The soil response spectrum ratios of the soil surface (A-4a) to the soil base (A-8a) were analysed, as shown in Fig. 8. Under the excitation of white noise, AW0, the peak acceleration spectrum ratio occurred at 0.1 s, which is the site predominant period. Under the YG, EL, and MEX excitation, the acceleration spectrum ratios show different characteristics. For the YG excitation case, the peak spectrum ratio is 2.82 at 0.23s. With the increase of earthquake intensity, the peak spectrum ratio decrease, because of the different level of soil nonlinearity. With the increase of earthquake intensity, the level of soil nonlinearity increased. The soil may suffer from plastic deformation, in which process the earthquake energy is dissipated. Under the EL excitation, a similar characteristic is shown with that under the YG excitation. However, the peak spectrum ratio under the EL excitation is smaller than that under the YG excitation for the YG wave owning more energy than the EL wave. For the MEX excitation, the period corresponding to the acceleration spectrum ratio is more extended than that of the YG and the EL excitation. The peak location of the spectrum ratio curves is primarily influenced by the site predominant period.

3.2.2 Influence of piles on seismic response of soil

Fig. 9 summarised the acceleration characteristics of soil with piles and the free-field soil. The red line represents the recorded soil acceleration data from the shaking table test of case A-1 of the free-field soil. The soil acceleration data for the black line is generated from case B with a pile-raft foundation. Comparing soil surface acceleration between the soil with piles and the free-field soil shows that the former one owns more excellent acceleration. The accelerometers (A-4b, A-6b, A-7b) in the soil of case B located at the soil depth of 0.0 m, -0.3 m, and -0.6 m are near piles. The accelerometer (A-8b) at the soil depth of -1.2 m located at the bottom of the laminar shear model box, and it has an over five times pile diameter distance from the boundary of the laminar shear model box. Comparing to the acceleration (A-8b) at the depth of -1.2 m, the soil surface acceleration (A-4b) has an amplification. Because of the nonlinearity of the soil, the soil acceleration shows a nonlinear characteristic under earthquake loadings. Fig. 10 demonstrates the soil surface acceleration (A-5b and A-4b) near and far from the piles. The solid lines and dashed lines represent the experimental results of accelerations far from and near piles, respectively. Soil surface acceleration under MEX excitation is higher than that of the YG and EL exicitation results, because of the long-period characteristic of MEX wave. With the increase of PGA, the peak soil surface acceleration also shows a nonlinear increase.

3.3 Influence of peak ground acceleration on raft and superstructure response

3.3.1 Influence of peak ground acceleration on raft acceleration

The designed model acceleration for each case may not be the same as the actual acceleration excited by the shaking table. The actual input acceleration of the shaking table is also summarised in Tab. 3, named recorded acceleration. In the following discussions, the actual input acceleration of the shaking table is utilised as the peak ground acceleration (PGA). The influence of PGA on the fixed-base structure of case A-2 is illustrated in Fig. 11. The acceleration of raft increased with the PGA with an excellent linear relationship, which means the fixed-base structure mainly works in the elastic state.

Fig. 12 plots the relationship between PGA and the peak raft acceleration of case B. The peak accelerations of the raft increase with the PGA increase. The peak acceleration value of the raft located among two dashed lines, one line with a slope of 3:2 and the other line with a slope of 1:1. The relationship between those two parameters is not linear, and the increase ratio decrease with the increase of PGA.

Case A-2 and case B are the fixed-base structure and structure with pile-raft foundation shaking table tests. Fig. 13 shows the comparison of raft acceleration between case A-2 and case B under three earthquake excitations with different earthquake intensity. The characteristics of peak raft acceleration of fixed-base structure show significant differences from that of structure with a pile-raft foundation. The former acceleration is lower than the latter one. For the soil condition of this model, the analysis is more reasonable than simplifying the system into a fixed-base structure, considering the soil-pile-structure interaction.

3.3.2 Influence of peak soil surface acceleration on raft acceleration

The relationship between peak soil surface acceleration and peak raft acceleration of the pile-raft foundation is plotted in Fig. 14. All the peak accelerations are above the line with 1:1 slope, which means that the peak raft accelerations are higher than the peak soil surface accelerations. Besides, the average peak acceleration ratio (raft/soil surface) is 1.2, as shown in Fig. 14. In practice, using the soil surface acceleration as the input acceleration of superstructure without considering the SSI effect is normal. The test results provide evidence that utilising the soil surface acceleration as the input acceleration of the superstructure is not reasonable.

3.3.3 Acceleration amplification ratios ~~characteristics~~

The accelerometer equipped in the middle of the superstructure could monitor the time-history acceleration of the superstructure. The peak acceleration generated from the recorded data is utilised to get the acceleration amplification ratio by dividing the corresponding PGA. Fig. 15 plots the acceleration amplification ratios for the superstructure of the pile-raft foundation under three earthquake excitations with different earthquake intensity. The acceleration amplification ratios all over 1.5, and it decreases with the increase of PGA. For instance, under the excitation of 0.3 g, 0.6 g, and 0.9 g EL, the acceleration amplification ratios are 2.16, 1.83, and 1.71. The acceleration amplification ratio under MEX excitation is the greatest among those three different excitations.

The superstructure acceleration of the fixed-base case (case A-2) could be used to identify the difference between the fixed-base case and pile-raft foundation case. Because of the problem of the acquisition system under the shaking table test, only the time-history acceleration data of the superstructure under the YG 0.3 g, EL 0.3 g, EL 0.6 g cases were recorded. The amplification ratios of the fixed-base superstructure under the excitation of the YG 0.3 g, EL 0.3 g, EL 0.6 g are 2.77, 2.28, and 2.16, respectively. As shown in Fig. 15, the amplification ratios of superstructure with pile-raft foundation of case B under the excitation of the YG 0.3 g, EL 0.3 g, EL 0.6 g are 2.37, 2.16, and 1.83. The former ratios significantly higher than that of the latter, and increased by 17%, 6%, and 18%. That is because, in the propagation of earthquake waves, the soft soil and pile-raft foundation consumed part of the earthquake energy, and the energy transported to the superstructure is decreased.

3.4 Bending moment results and discussion

Four instrumented piles (P1, P2, P3 and P4) were equipped with spaced strain gauges to detect the bending moments of the piles. The bending moments of the piles were measured at -0.55, -0.45, -0.35, -0.25, -0.15 and -0.05 m under the pile head (±0 m). Each side of the piles at the measuring point was attached with one strain gauge, symmetrically. Moreover, the bending moments of the piles at the measuring point was determined by the average maximum and minimum strain based on the classical beam theory [44]. Utilising the average strain can eliminate the effects of time lag and reduce test error [17]. Fig. 16 plots the recorded strain (ε) data at each side of the corner pile (P1) at the depth of -0.05 m. The peak bending moment of measuring point is:

$$M = \frac{EI(\varepsilon_t - \varepsilon_c)}{2r} \tag{3}$$

where E is the elasticity modulus of the pile; I is the inertia moment of the pipe pile; ε_t is the average strain of Max left and Max right; ε_c is the average strain of Min left and Min right; r is the radius of the pile. In order to avoid the error caused in the process of gauges sticking and waterproof protection, all the strain gauges were calibrated, as shown in Fig. 17. The pile head is fixed, and the pile acted as a cantilever beam. Four calibration loads (2.84, 4.68, 5.61, and 6.54 kg) were equipped at the pile tip, and a linear relationship between the bending moment and strain can be generated. For instance, the corner pile calibration results of calibration moment and calculated bending moment using equation (3) were compared in Fig. 17, and the values are close to each other with a neglectable difference.

The peak bending moments of each instrumented piles under three earthquake excitations with different earthquake intensity are presented in Fig.18. The bending moment increased along with the pile from pile tip to pile head. The maximum bending moment occurred at the top of the piles, which is agree with previous studies [16, 45-46]. The results of the pile bending moment demonstrate that the pile at the middle of first row P2 had a significantly higher bending moment than the other instrumented piles, because the pile P2 shared more load and it was influenced by the shadowing effect. The shadow effect was also found in other researches [47-48]. Comparing the bending moment of piles under 0.3g, 0.6g, and 0.9g YG excitations, the bending moment of piles under greater earthquake excitations has a greater bending moment. For instance, the bending moment at the head measuring point of pile 2 under 0.3g, 0.6g, and 0.9g YG excitations were 1.30 N·m, 4.23 N·m, and 6.28 N·m. The same bending moment characteristics could be found under the earthquake excitation of EL and MEX, as shown in Figs. 18(b) and 18(c). The bending moment of piles under the excitation of MEX excitation is the greatest in three different earthquake excitations. Under the 0.9 g earthquake excitation, the bending moment of P2 head for the case under the YG, EL, and MEX excitation are 6.28 N·m, 5.33 N·m, and 22.08 N·m. This is because the MEX wave is a long-period wave, and during the excitation, the resonant effect generating more energy.

3.5 Vertical and horizontal displacements results and discussion

One laser meter (LS1) for vertical displacement measurement is equipped at the top of the superstructure. Fig. 19 shows the vertical displacement of superstructure under the EL excitations with three earthquake intensity, 0.3 g, 0.6 g, and 0.9 g. It demonstrates that the vertical displacement is proportional to the earthquake intensity. The vertical displacements under the EL excitation of 0.3 g, 0.6 g, and 0.9 g are -0.05 mm, -0.14 mm, and -0.32 mm, respectively. The same characteristics could be found with the earthquake excitations of the YG and MEX. Fig. 20 shows the vertical displacement of the superstructure under the YG, EL, and MEX excitations with earthquake acceleration of 0.3g. The time-history vertical displacement and the residual displacement after the shaking could be generated from Fig. 20. with the same earthquake intensity, the vertical displacement under the MEX excitation is the greatest for the resonance effect between the earthquake and the soil.

Fig. 21 plots the horizontal displacement of the superstructure under the EL excitations. The laser meter (LS2) monitoring the horizontal displacement is fixed at the laminar shear model box by a steel beam, which means the recorded horizontal displacement is the absolute displacement of the superstructure. The maximum horizontal displacement of the superstructure under the 0.3 g, 0.6 g, and 0.9 g EL excitations are -1.99 mm, -6.59 mm, and -13.35 mm, respectively. The maximum horizontal displacement increases with the earthquake intensity. Fig. 22 shows the response of horizontal displacement under three earthquake excitations. The maximum horizontal displacements under the 0.3 g earthquake excitation of YG, EL, and MEX are -2.28 mm, -1.99 mm, and -12.12 mm, respectively. The residential horizontal displacement is almost zero even under the MEX excitation, which means the superstructure is under the elastic state.

4 Conclusion

In this study, a series of shaking table tests on a nuclear power station model with a fixed-base and with a pile-raft foundation embedded in soft clay were carried out to investigate the effect of earthquake intensity, earthquake frequency, soil-pile-structure interaction, soil nonlinearity on the dynamic response of the system. The major conclusions are as follows:

(1) The soil acceleration shows a nonlinear characteristic due to an increased influence of the nonlinearity of soil under increasing earthquake excitation. The acceleration at the soil surface is amplified from the base of soil, and the soil surface acceleration under the long-period MEX excitation is higher than that under short-period excitations. The average peak acceleration ratio of the raft for pile-raft foundation (raft/soil surface) is 1.2, which provides evidence that utilising the soil surface acceleration as the input acceleration of superstructure is not reasonable.

(2) The acceleration amplification ratios of the superstructure with pile-raft foundation are all over 1.5, decreasing with the increase of earthquake intensity. Comparing the two foundation types, the natural frequency of the fixed-base superstructure is higher than that of the pile-raft foundation because of soil-pile-structure interaction. The damping ratio at the top of the fixed-base superstructure is lower than that of the pile-raft foundation. Since the peak raft acceleration of the fixed-base structure is nearly the same as the peak ground acceleration, it is lower than that of the pile-raft foundation. However, the amplification ratios of the fixed-base superstructure are higher than that of the pile-raft foundation.

(3) The pile at the middle of the first row (P2) had a greater bending moment than the other instrumented piles, because it shared more load and was influenced by the shadowing effect. The bending moment of piles under greater earthquake excitations owns a larger bending moment. Under the long period MEX excitations, the bending moment of the piles is larger than that under the YG and EL excitations.

(4) The vertical and horizontal displacement and residual displacement increase with the earthquake intensity. With the same earthquake intensity, the displacement under the MEX excitation is the greatest due to the resonance effect between the earthquake and the soil.

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