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Division V

INVESTIGATION ON THE SEISMIC RESPONSE OF FULLY EMBEDDED NUCLEAR POWER PLANT ON LIQUEFIABLE GROUND USING CENTRIFUGE TESTS

Xin Wang[1], Mi Zhao[2], Jiacheng Guan[3], Xu, Zhao[4], Zilan Zhong[5], Jie Yang[6], Xiuli Du[7]

[1] Graduate Student, Beijing University of Technology, Beijing, P.R. CHINA
[2] Professor, Beijing University of Technology, Beijing, P.R. CHINA (zhaomi@mails.bjut.edu.cn)
[3] Graduate Student, Beijing University of Technology, Beijing, P.R. CHINA
[4] Associate Professor, Beijing University of Technology, Beijing, P.R. CHINA
[5] Associate Professor, Beijing University of Technology, Beijing, P.R. CHINA
[6] Senior Engineer, Shanghai Nuclear Engineering Research & Design Institute Co., LTD., Shanghai, P.R. CHINA
[7] Professor, Beijing University of Technology, Beijing, P.R. CHINA

ABSTRACT

In order to obtain the earthquake response of a new type of buried nuclear power plant structure in a liquefiable site, a series of dynamic centrifuge tests are planned to be performed at early 2024. The test scheme is introduced in this paper. Limited by the capacity of the centrifuge, a geometric scaling ratio of 1:200 was set. As the original structure was rather complex with many inner walls, a simplified structure was designed to substitute the actual one. Then the generalized scaling law was discussed and the similarity theory was verified with the simplified physical model. Finally, numerical simulations for the physical model were carried out in a liquefiable interlayer site. These works provides guidelines for the tests to be started.

INTRODUCTION

New generation of small modular reactors (SMR) has been the order of the day. A new type of buried SMR structure has been designed. As structure safety is very important for such structures, it is necessary to study their seismic response on non-bedrock sites which may be liquefiable. seismic response of this buried nuclear power plant structure is significantly different from that of the conventional surface-sited reactor and is more similar to that of an underground structure as shown in Figure 1.

Centrifuge shaking table tests could provide equal stress with that of the prototype and has been widely used in the seismic research of underground structures (Zhang et al.,2017; Xu et al.,2021;). This research aims to investigate the seismic response of the buried nuclear power plant structure in liquefiable site through five centrifuge shaking table tests focusing on dynamic soil-structure interaction.

THE PROTOTYPE

As the model will be scaled from the complex prototype structure with a large size, a simplified prototype as a new prototype will be discussed in this section. This simplified prototype serves as a bridge between the original structure in ground and the model in centrifuge.

Geometrical scaling factor

A container is need in these tests as soil-structure interaction tests. A rectangular soil box in designed to meet the need that this box should be put into a centrifuge with a shaking table. Thus， the internal dimensions of the soil box used in this test are 0.8 m × 0.6 m × 0.6 m according to the limitation of the test space in the centrifuge. Shaking is input along the long side of the box, which is 0.8 m. The plan dimensions of the original structure are 78 m × 58.8 m, of the 2 different lengths are considered in determining the direction of shaking input on the model of the structure in the box in the test. As a result, it is necessary to perform numerical simulation to identify the shaking direction resulting stronger dynamic response among the 2 directions of the structure in the box.

The nuclear power plant structure-site model was established in ABAQUS (Dassault Systemes, 2009), which consists of a buried nuclear power plant structure and its geotechnical site, as shown in Figure 1. The modelled nuclear power plant structure consists of five parts: auxiliary building (AB), spatial steel truss (SST), containment internal structures (CIS), steel vessel (SV) and nuclear island basemat (NIB). The buried depth of the nuclear power plant structure is 37.2 m, the above-ground part is 25 m, and the plan dimension of the structure is 78 m × 58.85 m. The concrete grade is C45 (Poisson's ratio 0.17), and the steel is Q235 steel (Poisson's ratio 0.3). The structure of the nuclear power plant has 77015 elements and 70639 nodes, of which the plant is composed by shell element, the containment by shell element, the space steel truss by truss element, and the internal structure by shell element and solid element.

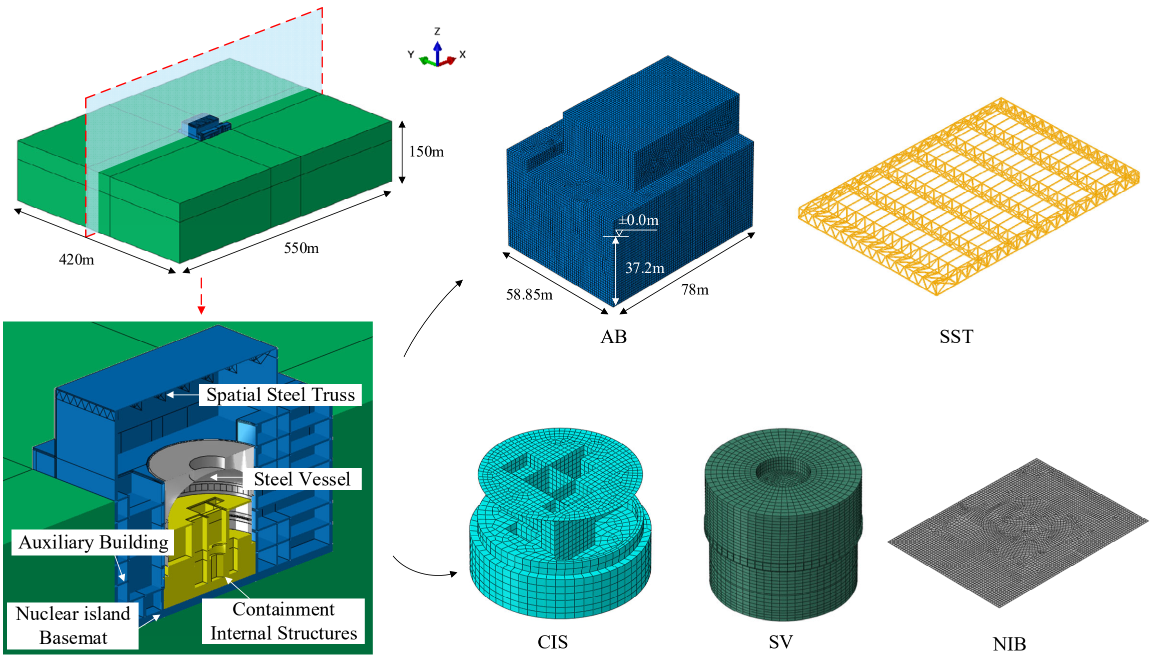


Figure 1. The profile and the components of the finite element model.

The original site model is 550m long, 420m wide and 120m high. The X direction is along the long side direction, and the Y direction is along the short side direction; the site model is meshed by solid elements whose size were set according to the fluctuation theory. The total number of elements is 946,205. In order to take into account the nonlinearity of the soil, a frequency-domain equivalent linearized model (Bardet et al., 2000) was used. The soil parameters of the equivalent linearized model and the shear wave velocity of the site are displayed in Figure 2.

The near-fault impulse-containing seismic record "Northridge-01", the far-field seismic record "Tottori_Japan" and the artificial seismic wave fitted based on the RG 1.60 (U.S. NRC, 2014) design spectra are selected as the input ground motion. These seismic waves were downloaded from PEER (Pacific Earthquake Engineering Research Center, 2023). The acceleration time histories and the corresponding acceleration response spectra are shown in Figure 3.

In the numerical model, seismic wave of 0.15g is forced input in in the X and Y directions at the bottom of the soil, respectively. The response of this soil-structure system was calculated, table 1 shows the peak relative displacement of the top of the containment internal structures for different seismic. Where, for convenience in presentation, the artificial seismic wave fitted based on the RG 1.60 design spectra is abbreviated as RG1.60. Of the results, the response in the Y direction is larger.

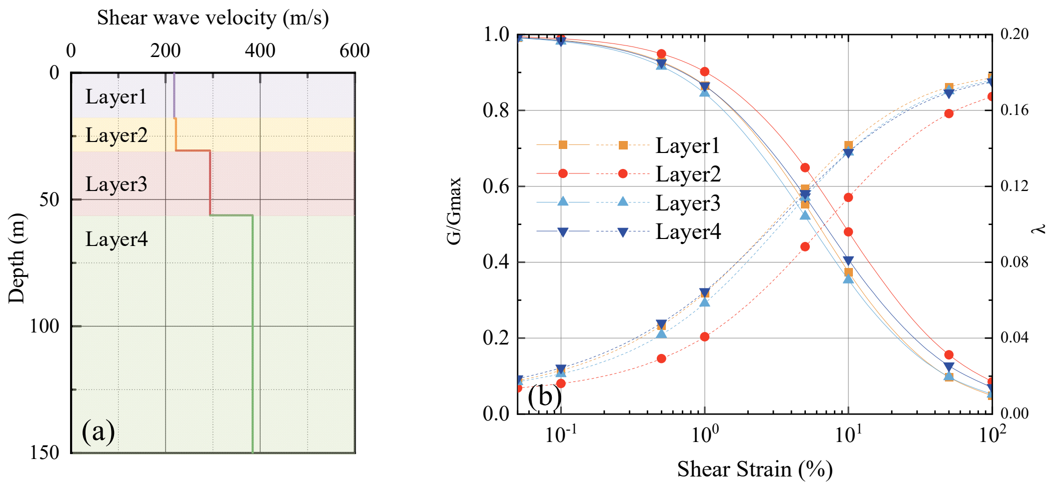
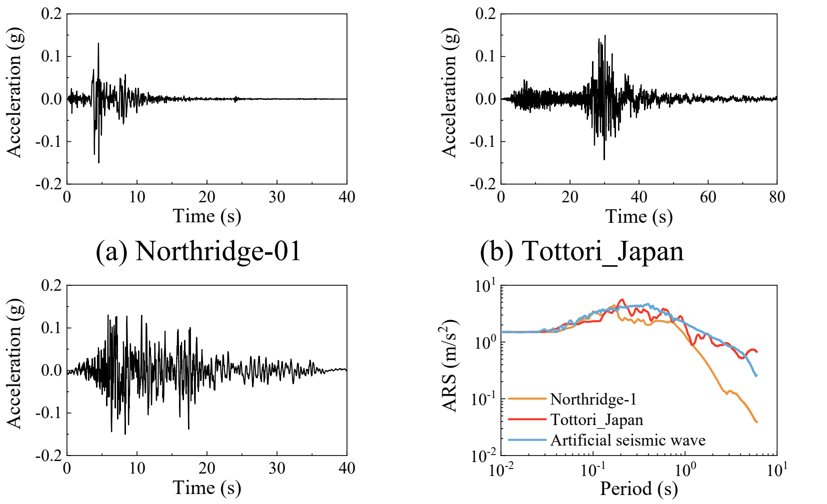


Figure 2. The properties of the site. (a) shear wave velocity of layered sites. (b) the shear modulus reduction - shear strain and damping ratio - shear strain fitting curves of each layer of soil.



(a) Northridge-01 (b) Tottori_Japan

(c) Artificial seismic wave (d) Acceleration response spectra

Figure 3. The acceleration time history of the input wave.

Table 1: Comparison of Peak relative displacement at the top of the containment internal structures (Units: mm).

	Northridge-01	Tottori_Japan	RG 1.60
X- direction	3.69	3.76	3.94
Y- direction	5.08	4.97	5.84

The peak relative displacement of the observation points of the auxiliary building are presented in Figure 4. The relative displacement here refers to the displacement relative to the bottom of the structure. Based on the displacement results, it can be found that the response in the Y direction is larger.

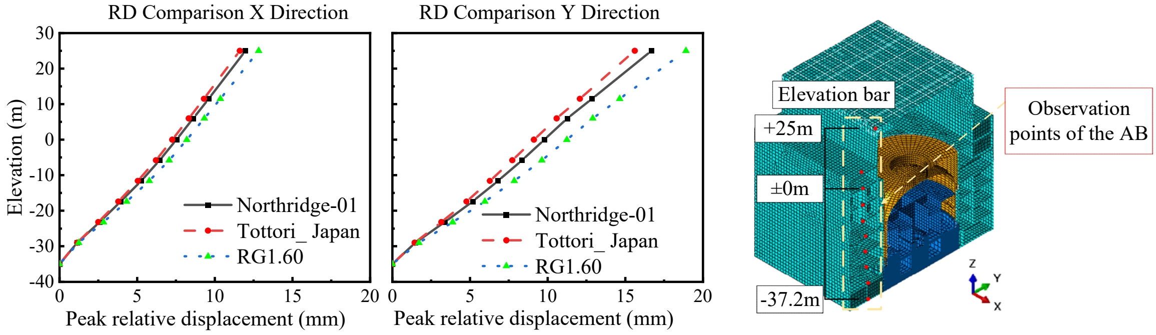


Figure 4. Comparison of Relative displacement at the observation points of the auxiliary building.

From the above results, it can be determined that the direction with lager dynamic response of the structure is the Y direction, which is a weaker direction of the structure. So, it is necessary to set the short side of the structure to correspond to the shaking direction of the shaking table. So, the planes of the structure in the box are represented in the plan view with different considerations of scaling ratios, as shown in Figure 5.

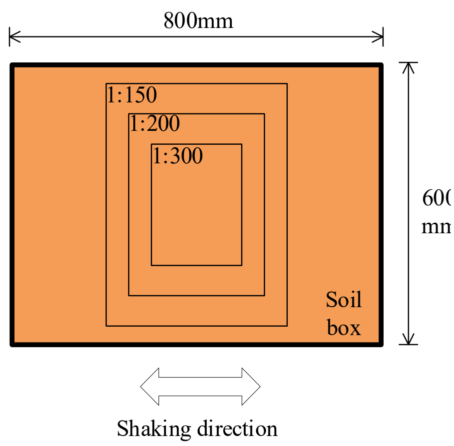


Figure 5. Different structural scaled dimensions in soil box.

As can be seen in Figure 4, 1:200 is a more reasonable geometric scaling ratio, when the total length of the soil in the shaking direction is 2.7 times the length of the structure.

Simplified structure

As the geometry scaling factor is set as 200, the detailed configuration of the original structure with very complex inner substructures could not be made into a model with small thickness, so it was necessary to simplify the original structure. Since the main part of the structure is underground, it is the inertial and kinematic interactions, i.e. mass and structural stiffness, that play a controlling role in the dynamic calculations, so the simplified design is according to the principle of mass and stiffness approximation.

The simplified structure is divided into 3 parts: main structure, containment, and roof. Concrete is used for the main structure, and Plexiglas is used for the containment and roof, the simplified structure is shown in Figure 6. Plexiglass is easy to assemble and can be thinner, so it is used in places like containment and roof where it does not affect the whole response. The main parameters of the simplified structure compared with the original structure are shown in Table 2.

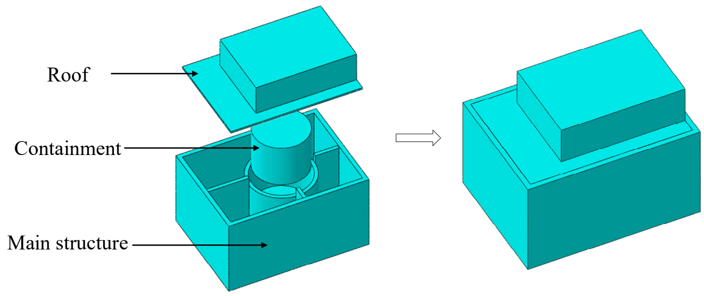


Figure 6. The assembly of the simplified structure.

Table 2: Comparison of the main parameters of the simplified structure and the original structure

	Moment of inertia			Center of gravity (m)	Total mass (kg)
	I_{xx}	I_{yy}	I_{zz}		
Original structure	1.04×10^{11}	1.45×10^{11}	1.61×10^{11}	(37.46,28.35,18.26)	1.81×10^{8}
Simplified structure	1.08×10^{10}	1.49×10^{11}	1.65×10^{11}	(37.48,29.22,18.52)	1.84×10^{8}
Relative deviation	3.9%	2.8%	2.5%	1.5%	1.6%

From Table 2, the main parameters of the simplified structure and the original structure are close to each other, which basically proves that the simplification of the structure is reasonable.

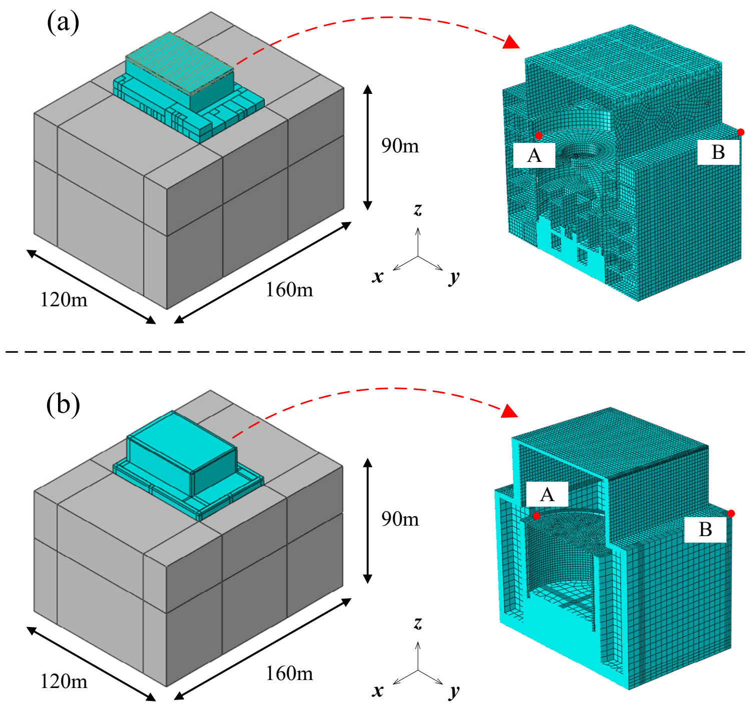
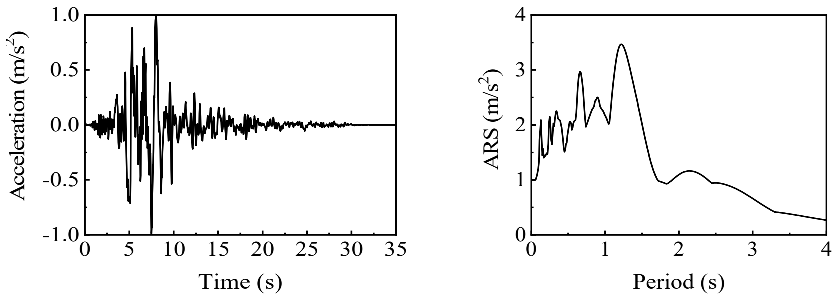


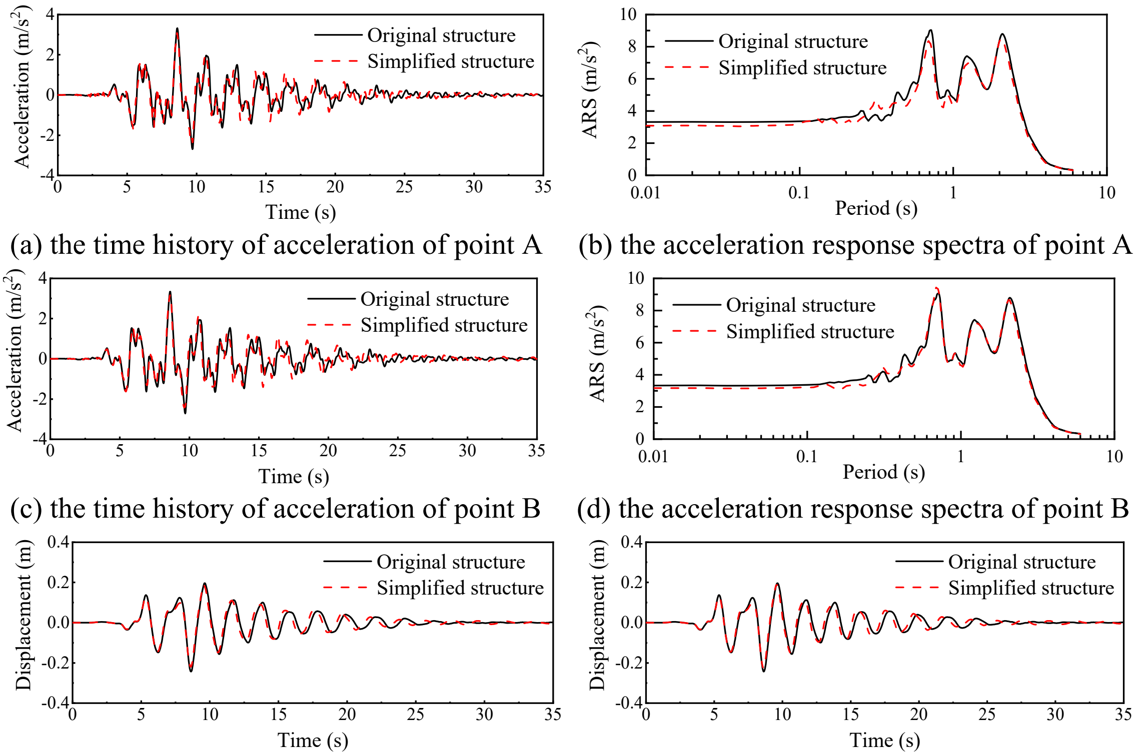
Figure 7. Location of observation points on the original structure-soil system and the simplified structure-soil system. (a) Original structure-soil system. (b) Simplified structure-soil system.

In order to further verify that the simplified structure is close to the original structure in stiffness, numerical simulations considering soil-structure interactions were carried out for both the original and simplified structures by selecting the seismic record " Kobe, Japan" from Kobe University station as input and building a site model scaled up by 200 times the size of the soil box, which is the same with that of the structure scale; the site material was set as sandy soil with a shear wave velocity of 178.9 m/s and roller boundaries were used. The location of observation points is shown in Figure 7. Kobe

Wave will be used for seismic record "Kobe, Japan" at Kobe University station in the following. The acceleration time history and response spectra of Kobe wave are shown in Figure 8. The amplitude of seismic wave was adjusted to 0.15g and input at the bottom of the site. The acceleration response of observation points A and B on the structure is shown in Figure 9.



(a) the acceleration time history (b) the acceleration response spectra
Figure 8. The acceleration time history and response spectra of Kobe wave.



(a) the time history of acceleration of point A (b) the acceleration response spectra of point A
(c) the time history of acceleration of point B (d) the acceleration response spectra of point B
(e) the time history of displacement of point A (f) the time history of displacement of point B
Figure 9. Results of the observation points on the original structure and simplified structure.

From Figure 9, the acceleration and displacement responses of the simplified structure and the original structure at each observation point are basically the same. Combining the acceleration response and displacement response of the structure, it can be concluded that the simplification of the original structure is reasonable, and the simplified structure can effectively represent the response of the original structure.

THE SCALING LAW

In conventional centrifuge scaling law (Garnier et al., 2007), the geometry scaling factor (N) and centrifugal acceleration (Ng) of the centrifuge model are equal. According to the above scaling law, this test requires a centrifugal acceleration of 200g, which exceeds the maximum limit of the equipment to be used. This should be solved by a proper method.

The generalized scaling law

Iai et al. (2005) extends the centrifuge scaling law by using the 1g scaling law of separate scaling for stress and strain, and developed the generalized scaling law (GSL) for centrifuge shaking table test, which supports the use of existing centrifuges to achieve larger geometrical scaling factor. In the first stage of the GSL, the prototype is scaled down to an intermediate virtual model using the 1g scaling law with a geometrical scaling factor of length (μ); in the second stage, the intermediate virtual model is further scaled down to a physical model using the conventional centrifuge scaling law with a geometrical scaling factor of length (η), as shown in Figure 10. As a result, the total geometrical scaling factor of GSL is $\lambda = \mu\eta$. In this way, it was possible to carry out experiments by simply varying μ and η to suit different centrifuges.

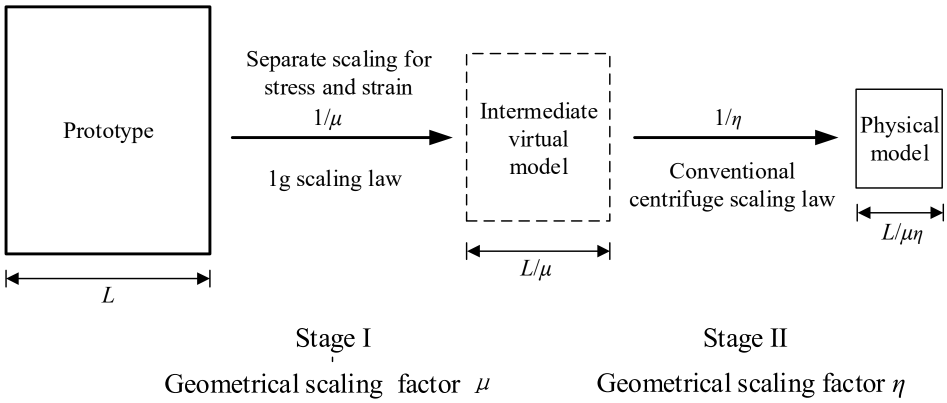


Figure 10. Concept of two-stage GSL.

The GSL is classified into type I, type II and type III. Type I assumes that the scaling factor of density is 1, i.e., the density of the prototype and the physical model are the same, by which the scaling factor of strain can be obtained by analysing the wave velocity of the prototype and the physical model. Type II is applicable to the case of small strains, by which it is assumed that the shear modulus is proportional to the square root of the confining pressure. Type III is used in the case of large strains in which it is assumed that the scaling factor of density and strain are both 1. Of these, the Type 2 GSL has been widely used in centrifuge modelling of soil liquefaction and soil-structure interaction problems (Tobita et al., 2011; Ueda et al., 2019; Zhou et al., 2021; Korre et al., 2021; Borghei and Ghayoomi,2021). The details of scaling law are given in Table 3.

Verification of generalized scaling law

Although quite a number of tests have been conducted to prove the reasonability of GSL, the present experiments has a much larger structure, which is obviously different from the above studies. To further verify the reasonability of the GSL, soil-structure system models before and after scaling down were also established for seismic response comparison. Since the structure was expected to enter the plastic state in the experiments, reinforcement was added to the model, and the scaling down finite element model was modelled as shown in Figure 11.

The 0.3g Kobe Wave was used as input. The site material was sandy soil with a shear wave velocity of 178.9 m/s, and Mohr-Coulomb model was used with a friction angle of 30° and a cohesion of 1 kPa. The ideal elastoplastic model was used for the steel reinforcement with a yield stress of 240 MPa. The concrete was with the Concrete Damage Plasticity model (CDP), and the parameters were selected with reference to Xu et al. (2021). In calculating the soil-structure system, ground stress balance is carried out first, and then ground motion is applied for calculation. The maximum strain envelope cloud of the structure is shown in Figure 12 and the response of the observed point is shown in Figure 13.

Table 3: The scaling factors used in the experiments

Quantity	Scaling factors of GSL	Virtual 1g test (prototype / intermediate virtual model)	Virtual centrifuge test (intermediate virtual model/ physical model)	Scaling factors used in the experiments
Length	$\mu\eta$	4	50	200
Density	μ_ρ	1	1	1
Time	$(\mu\mu_\varepsilon)^{0.5}\eta$	2.8284	50	141.42
Frequency	$(\mu\mu_\varepsilon)^{-0.5}\eta$	1/2.8284	1/50	1/141.42
Acceleration	$1/\eta$	1	1/50	1/50
Velocity	$(\mu\mu_\varepsilon)^{0.5}$	2.8284	1	2.8284
Displacement	$\mu\mu_\varepsilon\eta$	8	50	400
Stress	$\mu\mu_\rho$	4	1	4
Strain	μ_ε	2	1	2
Permeability	$[(\mu\mu_\varepsilon)^{0.5}/\mu_\rho]\eta$	2.8284	50	141.42
Pore pressure	$\mu\mu_\rho$	4	1	4

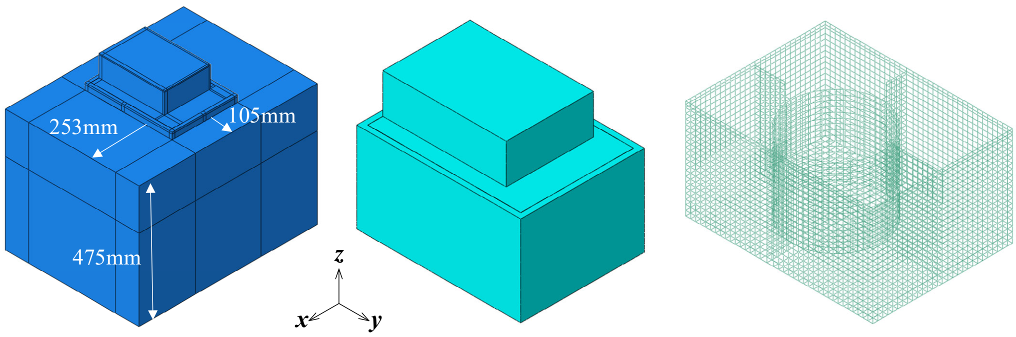
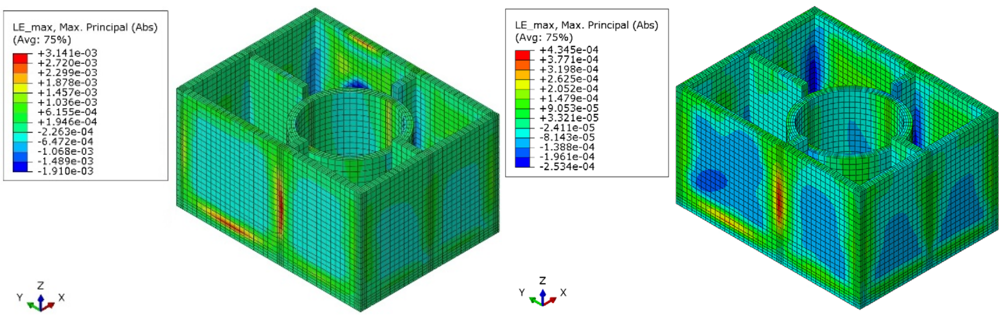


Figure 11. Finite element model of the physical model used in the experiments.



(a) prototype model (b) physical model

Figure 12. Maximum strain envelope nephogram of the structure.

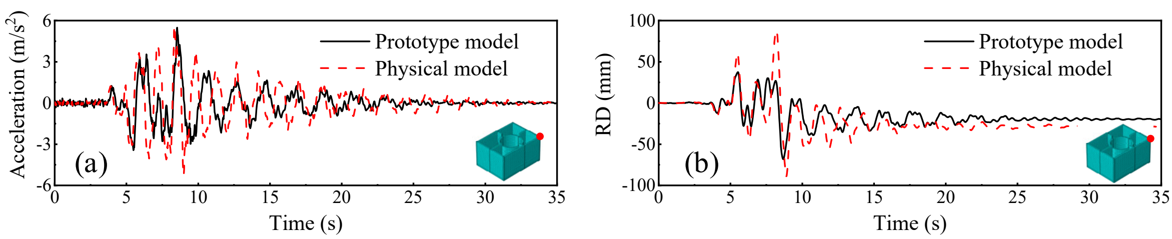
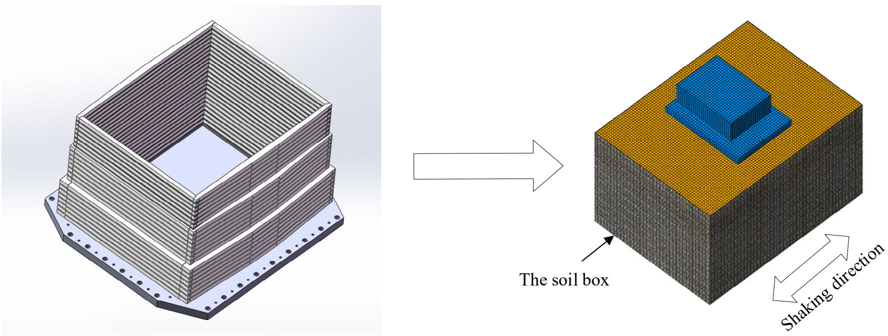


Figure 13. Results comparison of the observation point on the prototype model and physical model. (a) the time history of acceleration. (b) the time history of relative displacement.

According to Figure 12 and Figure 13, the acceleration amplitude of the prototype model and the physical model are close and the waveforms are similar; the displacement amplitude and waveforms are similar. Basically, it can be considered that the GSL is reasonable and meets the test requirements.

NUMERICAL SIMULATION OF PHYSICAL MODEL

Numerical simulation of the physical model was performed using the finite element software GFE-SSA (Cao et al., 2019). The site was a liquefied interlayer site with an upper clay layer of 50 mm thickness, a middle sandy layer of 100 mm thickness, and a bottom clay layer of 325 mm thickness. The elasticity parameters of soil are referenced to Yan et al. (2023)., and the loosely coupled effective stress method developed by Chen et al. (2021). is used for the site. This method coupling the expanded DCZ model (Chen et al., 2020) with a newly proposed excess pore water pressure generation model of Chen et al. (2019)., can effectively simulate the seismic response of liquefiable sites. As shown in Figure 14, a finite element model of the soil box-soil-structure was modelled. The concrete of structure is modelled in elastoplastic materials with concrete damage plasticity model. The amplitude of Kobe wave was adjusted to 25g (corresponding 0.5g in prototype) and input at the bottom of the soil box in the X-direction. Some typical results are shown in Figures 15.



The soil box The soil box-soil-structure

Figure 14. Finite element model of the soil box-soil-structure.

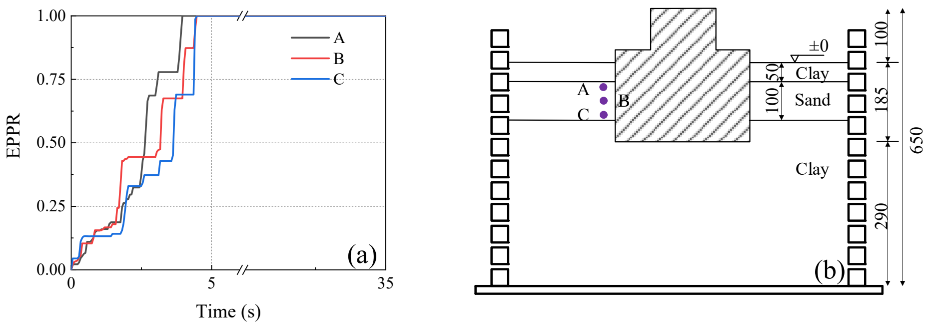


Figure 15. Results for typical locations. (a) time history of excess pore water pressure ratio (EPPR) at typical locations (in prototype). (b) the positions of observation points A, B, and C (in physical model).

Based on Figure 15, it can be seen that the liquefied interlayer in the liquefied interlayer site is sufficient to produce liquefaction, and the higher the height is the later the point liquefies.

CONCLUSION

The test scheme of centrifuge shake table for seismic response of a new type of buried structure at liquefiable site is introduced. The key issues such as the simplification of the structure, the generalized scaling law used are described in detail in the paper, and finally numerical simulation is carried out for the response of liquefied interlayer site, which has good effect. The test will be carried out soon.

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