3D seismic response characteristics of a pile-mat-founded AP1000 nuclear-island building considering nonlinear hysteretic behavior of soil

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Abstract

Given the increase of nuclear power plants, it has become unavoidable for the pile-supported nuclear-island buildings to be constructed on the coastal deposits potentially influenced by strong earthquakes. The coupling influences of the hysteresis nonlinearity of soil and the soil-pile-structure interaction (SPSI) have not yet been considered comprehensively in the seismic response analysis of the nuclear-island building, although it is a vital issue. On the basis of a newly-developed generalized non-Masing hysteretic constitutive model, a 3D integrated simulation method is proposed to evaluate the seismic responses of the pile-mat-founded nuclear-island building system subjected to multidirectional earthquake motions. This integrated method involves an explicit parallel algorithm framework, comprising the nuclear-island building modeling, the pile-mat foundation modeling, the inhomogeneous soil domain modeling, and the artificial boundary condition. The bedrock records of near-field, moderate-far field and far-field earthquake scenarios are assumed for determining the bedrock motions of the ultimate and operational safety earthquakes. For an actual pile-mat-founded AP1000 nuclear-island building, the simulation results show the complexity and significance of the coupling effect of the site, the tridirectional earthquake shaking, and the secondary nonlinearity of soil. Such a complex coupling effect significantly increases the seismic responses of the pile-mat-founded nuclear-island building. A notable finding is that the scenario earthquakes with abundant long period components may have more destructive potential to the pile-mat-founded nuclear-island buildings than the scenario earthquakes with characteristics of abundant short period components and shorter durations. The results provide insights into the seismic design of the pile-mat-founded nuclear-island buildings, which could guide the design and construction of such similar facilities in the high seismic intensity regions.

Keywords Nuclear-island building · Seismic response · Nonlinear hysteretic behavior of soil · Soil-pile-structure interaction · Tridirectional earthquake shaking
1 Introduction

With the increasing demands for nuclear power plants, sites for constructing nuclear-island buildings with high-quality hard rocks gradually diminish; consequently, some future plants may have to be built on soft soil sites. Currently, more than half of the nuclear power plants in the United States and France are located on soft rock sites (CISS, 2011). Therefore, the prospect of nuclear-island buildings to be built on non-rock sites has become an unavoidable issue. However, the vulnerability of nuclear power plants in coastal soil sites during strong earthquakes can be devastating, as evidenced in the 2007 severe damage of the secondary facilities in the Kashiwazaki-Kariwa nuclear power plant and the disastrous event of the 2011 Fukushima Nuclear Reactor meltdown. Since 2011, more attention has been paid to the study of seismic hazards and the vulnerability of nuclear power plant structural components (Kumar and Whittaker 2017; Cai et al. 2018; Chen et al. 2019; Wang et al. 2019). Meanwhile, the dynamic effect of the complex soil site is highlighted in the soil-structure interaction (SSI) modeling and analysis part of the latest Seismic Analysis of Safety-Related Nuclear Structures (ASCE, 2017). Consequently, the effect of strong earthquake-induced nonlinear responses of the coastal deposits on the nuclear power plants is a critical issue for civil engineers.

The early studies of dynamic SSI analyses were mainly motivated by the need for seismic design and safety assessment of nuclear power plants (Kausel 2010). Further, observations from strong earthquake events (e.g., the 1985 Michoacán earthquake, the 1995 Kobe earthquake, the 1999 Chi-Chi earthquake, the 2008 Wenchuan earthquake, the 2011 Tohoku earthquake) also show evidence of the destructive nature of SSI in various circumstances (Chen et al. 2020). In particular, nuclear-island buildings are characterized by the large plan size, stiffness, and mass, often leading to a significant SSI effect. Thus, the dynamic SSI effect is one of the critical issues to consider in the seismic design of nuclear-island buildings on non-rock sites (ASCE 2021).

Kausel (2010) outlined the milestones for the state-of-the-art of SSI. In addition, many studies on the effect of dynamic SSI have also been made to gain insights into the seismic response characteristics of nuclear facilities founded on non-rock sites (e.g., Tunon-Sanjur et al. 2007; Bolisetti et al. 2015; Coleman et al. 2016; Abell et al. 2018; Li and Chen, 2020; Van Nguyen et al. 2020; Huang et al. 2021). For example, Tunon-Sanjur et al. (2007) showed that the three-dimensional (3D) lumped-mass stick (LMS) models could be used to determine the critical parameters of the AP1000 nuclear-island building needed in the SSI analyses. Huang et al. (2021) proposed an integrated simulation method for the nonlinear SSI analysis of a realistic APR1400 nuclear containment building. They highlighted the importance of the nonlinear SSI effect for seismic analysis of nuclear structures.

In addition, pile foundations have frequently been adopted for nuclear-island buildings on non-rock sites, such as the nuclear power plants in Point Beach, United States, Gösgen-Däniken, Switzerland, Angra, Brazil (Zou et al. 2020), and Fuqing, China. The existence of dynamic soil-pile-superstructure interaction (SPSI) directly affects the response characteristics of the nuclear-island building and its surrounding site. From the seismic design viewpoint, nuclear-island buildings must survive all possible effects of large earthquakes. However, fewer studies have focused on the dynamic SPSI impact on the seismic response of nuclear-island buildings, which should be worth pursuing (Luo et al. 2016; Zou et al. 2020). For example, Zou et al. (2020) recently used a refined 3D finite element model to
conduct the seismic damage analyses for a pile-mat-founded AP1000 nuclear-island building under tridirectional shaking. The results showed that the most severe damage to the pile group occurred at the pile-mat interface under conditions beyond design earthquakes.

Nonlinear behaviors of soil in the SSI (or SPSI) may be identified into one of the two types: primary nonlinearity and secondary nonlinearity (Králík and Šimonović 1999; Pitiilakis and Clouteau 2010). The primary nonlinearity represents the nonlinear behavior of soil induced by the propagation of seismic waves in the free field. The secondary nonlinearity is caused by the SSI (or SPSI) effect. Previous studies on the dynamic SSI (or SPSI) analysis of nuclear power plant structures primarily focused on linear (or equivalent-linear), homogeneous (or horizontally layered) soil domains. They generally ignored the influence of the secondary nonlinearity of soil on the seismic response of the SSI (or SPSI) system and oversimplified the nuclear structure modeling. Such analyses often introduced the variability and biases in the seismic responses.

In short, the implications of an idealized simulation on the seismic response characteristics of an actual nuclear-island building founded on soil sites are poorly understood. Thus, little is known about the effect of the soil nonlinearity and dynamic SPSI on the seismic responses of pile-mat-founded nuclear-island buildings subjected to strong earthquakes. In addition, fewer studies have considered the effect of actual tridirectional shaking and nonlinear SPSI on the earthquake responses of a soil-pile-mat-nuclear-island building system. In reality, the pile-mat-founded nuclear-island buildings are sensitive to the 3D features of earthquake wave fields. Firstly, they are both stiff and heavy with high fundamental frequencies, implying that they are susceptible to high-frequency components of strong earthquake motions. Secondly, some components of nuclear-island buildings are sensitive to acceleration responses (e.g., heavy equipment) and displacement responses (e.g., fluid sloshing of the cooling water tank).

Furthermore, it is difficult to perform large-scale physical model (field or laboratory) tests to simulate the dynamic SPSI effect for the pile-mat-founded nuclear-island buildings due to the enormously complicated configurations. Thus, numerical simulation is a practical alternative approach. Unfortunately, the current state-of-the-art dynamic nonlinear SPSI analysis for nuclear-island buildings is far from adequate.

This paper focuses on the 3D seismic response characteristics of an actual pile-mat-founded AP1000 nuclear-island building in a coastal region of China. The present study to simulate the 3D seismic response of the fully coupled SPSI system involves two components, and both are implemented in ABAQUS/Explicit platform (DSSC 2014). One is the generalized non-Masing constitutive model proposed by Chen et al. (2020, 2021), which models the sophisticated nonlinear hysteretic behavior of soil subjected to the cyclic loadings associated with earthquake shaking. The other is the 3D LMS model for AP1000 nuclear-island building modeling (Tunon-Sanjur et al. 2007). The SPSI system is assumed to be subjected to multiple scenario earthquake shakings representative of broadband frequency contents. This paper aims to clarify the seismic response characteristics of a pile-mat-founded AP1000 nuclear-island building considering the influence of the dynamic interaction with the surrounding soil. Moreover, the influences of the foundation flexibility, soil secondary nonlinearity, and the dimension differentiation of seismic input on the seismic responses are analyzed, respectively. To sum up, this paper systematically investigated the influences of various factors on the seismic response of nuclear island building through...
a lot of calculations and analyses, and some valuable results and conclusions are discussed and obtained.

## 2 Regional historical earthquakes and bedrock input motions

The zone that surrounds the AP1000 nuclear power plant site (116°00’ ~ 119°40’E, 36°80’ ~ 39°50’N) is located in a strong seismicity region, mainly affected by the North China seismic belt. Since 408 AD, there have been 108 earthquakes with Ms≥4½ recorded, including two earthquakes of Ms 5½ in 1624 (with the epicenters about 47 km away from the plant site), thirteen earthquakes with Ms between 6.0 and 6.9, and six earthquakes with Ms>7.0 (including the Ms 7½ event in 1888 about 120 km and the Ms 8.0 event in 1679 about 200 km away from the plant site). Therefore, the earthquake risk is relatively high in the plant site.

Due to the absence of historically strong seismic records near the plant site, it is challenging to identify bedrock motions. The seismic input levels in the code for seismic design of nuclear power plants in China include: ultimate safety earthquake (SL-2 level) and operational safety earthquake (SL-1 level), and the PGA of the SL-1 level is one half of the SL-2 level. At the plant site, the peak ground acceleration (PGA) at bedrock based on the SL-2 level was obtained by combining the results from the probabilistic seismic hazard analysis (at the level of 10⁻⁴ annual exceedance probability per CNS, 2005) and the deterministic seismic hazard analysis (using the tectonic earthquake method and the historical earthquake estimate). Thus, the horizontal and vertical PGAs for the SL-2 level are 200.0 cm/s². Given the magnitude and epicenter distance similarity between the actual earthquakes and the historical earthquakes in the region surrounding the plant site, the array records of three
scenario earthquakes are adjusted by amplitude scaling to determine the bedrock input motions with the PGAs of the SL-1 and SL-2 levels. Figure 1 shows the adjusted seismograms, the Fourier amplitude spectra, and the 5% damping spectral accelerations for the SL-1 level, respectively. For comparison, the 5% damping AP1000 standard design spectra of 100 cm/s$^2$ (Westinghouse 2012) is also shown in Fig. 1(c). As shown in Fig. 1, the near-field moderate-strong earthquake seismograms (FKSH21 record) are rich in high-frequency components within narrow spectral bandwidth, the moderate-far field strong earthquake (OITH10 record) are rich in low-frequency components within large spectral bandwidth, and the far-field large earthquake seismograms (SUCHIL record) are rich in low-moderate frequency components. The significant duration $D_{5-95\%}$ of between 5% and 95% of the Arias intensity is defined as the ground motion duration in this article. The details of the selected earthquake records used for bedrock input motions are given in Table 1. As shown in Table 1, the $D_{5-95\%}$ values of far-field and moderate-far strong-motion scenarios are much longer than those of the near-field strong-motion scenario. Thus, the selected three earthquake records cover a broad range of earthquake frequency contents with different $D_{5-95\%}$. Note that the peak accelerations of a vectorial combination of two orthogonal horizontal components for FKSH21, OITH10, and SUCHIL records with the SL-1 level are 133.13 cm/s$^2$, 125.63 cm/s$^2$, and 120.81 cm/s$^2$, respectively.

3 Geology and soil nonlinear cyclic behaviors of the site

3.1 Engineering geologic characteristics

The physiognomy of the nuclear power plant site belongs mainly to the coastal plain landform and locally to the fluvial landform. The superficial layers are covered by the Holocene sediments ($Q_{mc}$) and the upper Pleistocene sediments ($Q_{mc}'$), mainly consisting of silty clay and locally sandy soil from the coastal marine transitional sedimentary facies. The lower sediments are the upper Pleistocene volcanic basalt and deposits. The ground-water level is near the surface. A map of the two-dimensional (2D) subsurface stratigraphic cross-section of the AP1000 nuclear-island building site and the layout of boreholes are shown in Fig. 2. The mean S- and P-wave velocities of various layer soils measured using the PS suspension logging in the boreholes are given in Table 2. The underlying dense basalt with the S-wave velocity larger than 2400 m/s approximately below 60 m depth is regarded as a hard-rock (Westinghouse 2012). On the other hand, Yasuda and Gao (2021) suggested that the rock with the S-wave velocity of 2000 m/s or higher should be regarded as the preferable seismic bedrock. Thus, the top surface of the underlying dense basalt is determined as the “seismic” bedrock interface in this article.

3.2 Nonlinear constitutive model of soil

The constitutive model for describing the cyclic stress-strain behavior of soil under earthquake shaking plays a crucial role in the site response analyses. Existing studies indicated that proper characterization of nonlinear hysteretic behaviors of soil is essential in seismic wave propagation from bedrock (Chen et al. 2021). While advanced plasticity-based constitutive models are relatively sophisticated (Cubrinovski and Ishihara 1998; Boulanger and
Ziotopoulou, 2017&2018, their application in practice remains challenging. Only the measured modulus reduction and damping ratio curves are available for parameter calibration in most instances. To this end, proper and straightforward descriptions of the soil stress-strain behavior under irregular cyclic loadings, especially those within the family of hyperbolic formulations, are preferred in practice.

Chen et al. (2020, 2021) proposed a set of non-Masing rules for irregular cyclic loading sequences to model the 1D nonlinear hysteretic behavior of soil (see Fig. 3(a)), termed the DCZ model. The set of proposed non-Masing rules has the merits of simplicity and accuracy in capturing the strain reversal points: the stress-strain path under irregular cyclic loadings can be defined with only strain at the current reversal point and the historical maximum point. Thus, it minimizes the information that the model must “memorize.” Furthermore, by adopting a generalized algorithm of the equivalent shear strain ($\gamma_{eq}$) to capture the strain reversals in 2D and 3D stress conditions, the DCZ model is extended from 1D to 2D (and

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**Fig. 1** Plots of selected earthquake records with SL-1 level for (a) seismograms; (b) Fourier amplitude spectra; and (c) 5% damping spectral accelerations (log-log scale), as well as the 5% damping, AP1000 standard response spectra of 100 cm/s$^2$ level
3D) stress conditions. The extended DCZ model has been successfully implemented in ABAQUS/Explicit platform with a user-defined material subroutine VUMAT and has been tested and verified in the 2D or 3D nonlinear seismic response analyses (e.g., Miao et al. 2018, Ruan et al. 2019; Chen et al. 2020, 2021; Liu et al. 2021).

As shown in Fig. 3(a), the initial backbone curve in the DCZ model is expressed in a hyperbolic form:

\[
\tau = G_{\text{max}} \gamma \left[ 1 - \left\{ \frac{\gamma/\gamma_r}{1 + (\gamma/\gamma_r)^2B} \right\}^A \right] \tag{1}
\]

where \(\tau\) and \(\gamma\) are shear stress and strain, respectively; \(G_{\text{max}}\) is a small strain shear modulus; \(A\) and \(B\) are the dimensionless constants; \(\gamma_r\) is the reference shear strain.

The variation curves of shear modulus reduction \((G/G_{\text{max}})\) and the damping ratio \((\lambda)\) with shear strain are obtained from the resonant column tests for typical borehole undisturbed

| Soil layer structure | Sedimentary environment | Soil lithology | \(V_p\) (m/s) | \(V_s\) (m/s) | Density (kg/m\(^3\)) | DCZ model | \(A\) | \(B\) | \(\gamma_r\) (×10\(^{-4}\)) |
|---------------------|------------------------|----------------|-------------|-------------|---------------------|-----------|-----|-----|-------------|
| Top layer           | Qpd 4                  | Silty clay 1   | 1050        | 105         | 1810                | 1.03      | 0.47 | 5.28 |
| Middle layer        | Qmc 4                  | Silty clay with silty sand interbed | 1020 | 102 | 1900 | 1.01 | 0.45 | 6.04 |
|                     |                        | Mucky silty clay | 1520 | 239 | 1930 | 1.03 | 0.46 | 5.9  |
|                     |                        | Silty clay 2   | 1510        | 237         | 1890                | 1.03      | 0.45 | 6.01 |
| Lower layer         | Qmc 3                  | Silty clay 3   | 1504        | 254         | 1940                | 1.04      | 0.45 | 6.61 |
|                     |                        | Silty clay 4   | 1577        | 325         | 1960                | 1.06      | 0.44 | 7.41 |
|                     |                        | Silty sand     | 1580        | 346         | 2010                | 1.08      | 0.44 | 8.29 |
| Bottom layer        | \(\beta\)              | Vesicular basalt | 2525 | 1360 | 2620 | 1.15 | 0.40 | 47.13 |
|                     |                        | Dense basalt   | 4356        | 2417        | -                   | -         | -   | -   |

\(V_p=\) P-wave velocity; \(V_s=\) S-wave velocity.

Fig. 2 Stratigraphic section of the nuclear-island building site
soil samples in the nuclear-island building site, and are shown in Fig. 3(b). The parameters of soil were calibrated based on the element level laboratory tests and the in situ measurements of S- and P-wave velocities using the suspension S-P velocity logging method. The details of soils at the site and the parameters for the constitutive model of the soils are listed in Table 2.

4 Nonlinear seismic simulation method of soil-pile-mat -nuclear-island building system

4.1 Components of AP1000 nuclear island building

As shown in Fig. 4, the AP1000 nuclear power plant (Westinghouse 2012) consists of five principal building components: nuclear-island building, turbine building, auxiliary building, diesel generator building, and radioactive waste building. As the central part of the nuclear power plant structures, the nuclear-island building consists of the containment building (the steel containment vessel and the internal structures), the shield building, and the auxiliary building, all located on a thick concrete base mat. The top of the shield building is equipped with a cooling system water tank (see Fig. 4(b)), from which the water sprays to cool down the temperature of the containment vessel when the reactor is shut down. The volume of the tank is about 3000 m$^3$. Only the nuclear-island building is studied in the SSI analysis below.

4.2 Simulation model and method

Figure 5 illustrates the simulation model for the pile-mat-founded AP1000 nuclear-island building in soft soil deposits, in which the 3D finite-element model is established in the ABAQUS platform. The model includes three parts: the 3D lumped-mass stick model (hereafter termed the 3D LMS model) for the nuclear-island building (see Fig. 5(b), (d), and (e)), the pile-mat foundation (see Fig. 5(b) and (g)), and the soils in the computational domain (see Fig. 5(a) and (c)), extending the geotechnical cross-section in Fig. 2. The computational domain of soils at the site is 400 m · 300 m × 62 m, approximately five times that of the base.
Because of the extremely high-risk potential, nuclear-island buildings are designed to have a much lower failure probability compared to conventional structures. Thus, the conservative design calls for the pile-mat-nuclear-island building system to remain linearly elastic during intense earthquake shaking.

### 4.2.1 Nuclear-island building modeling and validation

The 3D LMS model is adopted to characterize the steel containment vessel, the internal containment structures, and the coupled shield and auxiliary buildings. As shown in Fig. 5(b) and (d), the overall 3D LMS model comprises lumped mass points, elastic structural sticks, and rigid beams. The lumped masses are located on the elevation of the main floor and at the locations of structural discontinuities. The structural eccentricities between the rigidity centers and the mass centers of the structures are simulated by connecting the lumped masses to the vertical elastic structural sticks with horizontal rigid beams. In addition, dynamic subsystems are integrated into the overall 3D LMS model to represent the coupling of the reactor coolant loop (Fig. 5(e)) with the internal containment structures and that of the polar crane model (Fig. 5(d)) with the containment vessel. The other subsystems and equipment are also assumed to be concentrated masses.

The parameters of the 3D LMS model are determined by extracting the structural sections from the 3D finite element model of the AP1000 nuclear-island building (Tunon-Sanjur et al. 2007; Westinghouse 2012). As the critical components in the nuclear-island building, the containment vessel and shield buildings are selected as the objects in this article. The element and node information of the containment vessel, the shield building, and the reactor coolant loop are given in Table 3; Fig. 5. The parameters of the critical nodes of the 3D LMS model are shown in Table 4.

As shown in Fig. 5(b), (d), and (e), the 3D two-node linear beam element (B31), the 3D two-node linear truss element (T3D2), and the 3D two-node linear pipe element (PIPE31) are selected for simulating the beam, the rod, and the pipe components in the nuclear-island building, respectively. The beam and rod components in the 3D LSM model are connected through the specific lumped mass points. Thus, the discrete structural components are integrated as a whole dynamic system consisting of 203 structural stick elements and 110 lumped mass points. Moreover, the interactions among the stick elements consist of 406...
constraint equations. In addition, the coupling constraint is used for ensuring the accurate definition of the degrees of freedom.

Due to the large size of the water tank sitting on the top of the shield building, the inertia and sloshing effect of water in the tank will significantly affect the safety of the nuclear-island building during strong earthquakes, which cannot be neglected in design. This dynamic influence of the water-tank system is mainly caused by the additional mass and hydrodynamic pressure. The extra mass will reduce the natural (modal) frequency of the nuclear-island building. The hydrodynamic pressure, including the impact pressure from the inertia of water that moves in unison with the tank wall and the sloshing pressure of water in the tank, will be transmitted to the shielding building through the water-tank interface. A simplified mass-spring model recommended in the guidelines (ASCE, 2017; Thomas et al. 1963) is commonly employed for seismic analysis of the tank-water system. This method assumes the essential nature of the seismic response of water in the tank can be represented by impulsive and convective modes. In other words, the water in the tank is regarded as the impact mass and the sloshing mass, respectively. In addition, the equivalent mass-spring model assumed that the water tank wall was rigid and the water in the tank was an incompressible ideal liquid (Housner 1957). This way, the equivalent mass-spring model is used to simulate the impact pressure and the sloshing pressure. Both can be regarded as the equivalent mass fixed at their respective equivalent heights above the bottom and top of the water tank. In the 3D LMS model based on the ABAQUS platform, the equivalent sloshing mass of the water is integrated into the lumped masses at the top (Node 312) and bottom (Node 311) of the water tank for considering fluid-structure interaction, implying the assumption of the 100% water level of the water tank volume in this article. On the other hand, the equivalent lumped mass Nodes 312 and 311 are separately connected to the shield building Nodes 310 (top) and 309 through a zero-length spring connector to simulate the effect of the sloshing water-tank system.

Given the complexity of the actual AP1000 nuclear-island structure, the modal analysis of the structure model should be first conducted to verify the modeling of nuclear-island building. Therefore, the 3D LMS models of the actual AP1000 nuclear-island building are separately built using ABAQUS and ANSYS platforms. Then, fixed-base computational modal analyses for the structure model are conducted. Figure 5(f) compares the modal frequencies of the first 15 modes of the 3D LMS model based on ABAQUS and ANSYS platforms. The modal frequencies in the two models are almost identical, indicating that the structural models in ABAQUS and ANSYS are equivalent. Besides, it should be noted that the first four modal frequencies represent the modal properties of the water sloshing in the tank, the 5th and 6th modal frequencies represent the modal properties of two non-structural components coupled to the subsystem (Fig. 5(e)). In contrast, the 7th to 15th modal frequencies are the modal properties of the primary structure of the AP1000 nuclear-island building (except the sloshing water and the two non-structural components).

In addition, a 3D finite element modal analysis of AP1000 nuclear-island building was also conducted by the ANSYS platform (Zhao and Chen 2014). In this analysis, the fundamental frequencies of the nuclear-island building with various water levels in the tank were 2.991−3.337 Hz, which are almost identical to the results obtained in this study (see Fig. 5(f)). Therefore, the 3D LMS modeling for the AP1000 nuclear-island building in this study is validated.
4.2.2 Pile-mat foundation modeling

The AP1000 nuclear-island building sits on a reinforced concrete base mat supported by 230 reinforced concrete piles. Figure 5(b) shows the overall 3D LMS model, including the
Table 3  Element and node information of the 3D LMS model for the AP1000 nuclear island building

| Structure (Element type) | Elastic modulus (GPa) | Poisson ratio | Damping ratio | Element ID | Node ID | Cross section area (m²) | Inertial moment $I_{XX} = I_{YY}$ (m⁴) | Shear deflection constant $S_X = S_Y$ |
|--------------------------|-----------------------|---------------|---------------|------------|---------|------------------------|--------------------------------------|-------------------------------|
| Shield building (Beam)   | 19.88                 | 0.17          | 0.07          | 31–38      | 80–160  | 124.30                 | 23304.00                            | 2.00                          |
|                          |                       |               |               | 301        | 160, 309| 4.69                   | 0.01                                | 0.00                          |
|                          |                       |               |               | 303        | 309, 310| 65.45                  | 4657.70                             | 2.00                          |
| Containment vessel (Beam)| 203.37                | 0.30          | 0.04          | 401        | 401, 402 | 1.35                   | 251.22                              | 0.53                          |
|                          |                       |               |               | 402–412    | 402–413  | 5.54                   | 1089.60                             | 2.00                          |
|                          |                       |               |               | 413        | 413, 414 | 1.22                   | 950.40                              | 0.49                          |
|                          |                       |               |               | 414        | 414, 415 | 0.43                   | 722.53                              | 0.19                          |
|                          |                       |               |               | 415        | 415, 416 | 0.16                   | 397.43                              | 0.09                          |
|                          |                       |               |               | 416        | 416, 417 | 0.05                   | 119.54                              | 0.06                          |
| Reactor coolant loop (Pipe)| 174.01–206.82         | 0.30          | 0.04          | 7028–7086  | 7034–7095 | -                      | 119.54                             | -                            |
|                          |                       |               |               | 7128–7186  | 7134–7195 |                        |                                     |                              |


pile-mat foundation. The base mat with 3 m thickness is modeled using 2140 eight-node linear brick elements (C3D8R) and 58 six-node linear triangular prism elements (C3D6) (see Fig. 5(g)). The element sizes are less than 1/10 of the shortest wavelength corresponding to the cutoff frequency of 25 Hz. Each pile is modeled using the 20 beam elements (B31) and fixed with the mat at the connecting node. The diameter and length of each pile are 1.5 and 36 m, respectively. The assumption of ignoring the slip between the underground structure and the surrounding soil is conservatively safe (Huo et al. 2005; Banerjee et al. 2014) indicated that the simulated results in terms of soil-pile bonding condition agree well with the centrifuge test results. In addition, the gapping and sliding between both soil-pile and soil-mat interface underneath the pile-mat foundation are very unlikely to occur due to a large number of piles and the deeply embedded base mat. Thus, the pile-mat foundation nodes are tied to surrounding soil nodes, and no sliding or gapping is allowed at the interface.

### 4.2.3 Spatial inhomogeneous soil domain modeling

The SPSI system should be represented as a fully coupled system, in which the soil is modeled as a continuum. To accurately capture the seismic wave propagation from the bedrock to the surface of a large soil domain, the element sizes that varies spatially along the direction of wave propagation should be smaller than 1/10–1/8 of the wavelength corresponding to the highest frequency (Kuhlemeyer and Lysmer, 1973), which is set as 25 Hz in this study. Thus, a non-uniform meshing scheme is adopted. The mesh sizes of the spatial inhomogeneous soils cover a width of 0.4 m~3.0 m in the two orthogonal horizontal directions and a length of 0.5 m~5 m in the vertical direction, respectively. The mesh of soils near the piles and mat is relatively finer. The global mesh for soils in the computational domain consists of 2,227,848 elements, including 2,185,518 C3D8R elements and 42,330 C3D6 elements. The mesh of soils surrounding the pile-mat foundation is shown in Fig. 5(a) and (c).

### 4.2.4 Artificial boundary condition and seismic input

The method of the seismic input determines the accuracy of the seismic simulation of the SPSI system. To reduce the scale of the computational domain, only the local site affected significantly by the seismic response of the SPSI system is mapped onto the computational domain. The rest is captured by an artificial boundary condition (Chen et al. 2015, 2020, ...
which allows the scattering waves to propagate through the cutoff boundaries toward infinity without reflection.

Zhang et al. (2016) proposed a viscous-spring artificial boundary with higher precision, which improved over the model proposed by Liu and Li (2005). The effectiveness of this enhanced viscous-spring artificial boundary has been verified in the results of the 2D and 3D nonlinear seismic response analyses (Ruan et al. 2019; Chen et al. 2020, 2021). In this study, the improved artificial boundary condition proposed by Zhang et al. (2016) is applied to the four lateral edges and the bottom (seismic bedrock surface) of the computational domain of soil. Note that for the multidirectional shaking analyses, the East-West (EW), North-South (NS), and Up-Down (UD) components of earthquake motion scenarios per event are applied to the X-axis, Y-axis, and Z-axis directions (see Fig. 5), respectively.

### 4.2.5 Solving the dynamic equilibrium equation

The seismic response analysis of the SPSI system is performed using the explicit parallel algorithm in ABAQUS/Explicit platform (DSSC 2014; Chen et al. 2011, 2021). The nonlinear time-domain solution of the motion equation for a dynamic system is expressed as:

\[
\dddot{u}^{(i)} = M^{-1}(F^{(i)} - I^{(i)})
\]

where \(\dddot{u}\) is the acceleration vector, \(M\) is the lumped mass matrix, \(F\) is the applied load vector, \(I\) is the internal force vector, the superscript \(i\) refers to the \(i\)th incremental step in an explicit dynamic analysis. In this study, the motion equation is solved using step by step integration to evaluate the nonlinearity of soils during earthquakes, and the step size is set to be on the order of \(10^{-5}\) s to \(10^{-6}\) s to achieve good convergence in the high nonlinearity of dynamic analysis. This parallel computational method has shown availability in 2D and 3D nonlinear seismic analyses (Chen et al. 2015, 2020, 2021; Ruan et al. 2019; Li and Chen, 2020).

The formulation of Rayleigh damping is implemented to simulate the energy dissipation of various materials during earthquake events in this article. The damping ratios for multiple materials in the model calculation are determined according to the AP1000 Design Control Document. Table 3 shows the damping ratios of the AP1000 nuclear-island building components. For the concrete material of the pile-mat foundation, the damping ratio is 3%. For the non-Masing hysteretic constitutive model, near-zero damping is encountered at very small strains. However, small energy dissipation for soils still exists even at minimal strain levels (Chen et al. 2016; Afshari and Stewart 2019; Tao and Rathje 2019). Therefore, for simplicity, a small strain damping of 0.1% is incorporated in the ABAQUS/Explicit to resolve the near-zero damping problem in this article.

### 4.2.6 Elaborate scheme of numerical simulation

Various numerical calculations are performed to analyze the seismic behavior of the soil-pile-mat-nuclear-island-building system to characterize the seismic response characteristics of the realistic AP1000 nuclear-island building with nonlinear SPSI. A total of 30 numerical cases considering bidirectional or tridirectional earthquake shaking scenarios with the SL-1 and SL-2 levels per event are presented in Table 5.
Two methods are employed in the seismic response time history analyses of the SPSI system. The first is a one-step method (so-called integrated simulation method): directly performing the 3D response analysis of the overall SPSI system using the expanded DCZ model. It can capture the SPSI effect considering both the primary and secondary nonlinearities of soil and provide a pseudo-exact solution to the dynamic SPSI problem for ignoring the influence of gapping and sliding with a conservative assumption. The second is a two-step method: a 3D fully nonlinear analysis of the free field site response using the expanded DCZ model is first performed, followed by a 3D linear response analysis of the SPSI system using the strain-dependent equivalent modulus corresponding to the 3D free field site responses. In the latter, the damping ratio is obtained from the free field site response analysis. This method has the advantage of significant time-saving in computation, which can be saved about one third of time consuming for each of calculation cases in this article. However, it considers only the primary nonlinearity of soil and does not provide the “exact solution” to the dynamic SPSI problem.

5 Results and discussions

5.1 Nonlinearity measure of site response

The surface amplification characteristics of earthquake motions can be estimated by the horizontal-to-vertical (H/V) spectral ratio method (Nakamura 2019; Noguchi and Sasatani 2011) defined an index, named DNL (the degree of nonlinearity), to quantify the nonlinear characteristics of soils for site response by the following equation:

![Table 5 Scheme of the numerical simulations in this article](image-url)
where \( R_{\text{strong}} \) and \( R_{\text{weak}} \) are the mean H/V spectral ratio for strong and weak motions, respectively; \( f_i \) is the \( i \)th frequency ranging from 0.5 to 20 Hz; \( N_1 \) is the first index corresponding to 0.5 Hz, and \( N_2 \) is the last index corresponding to 20 Hz. Based on Noguchi and Sasatani (2011), the DNL is more significant when the H/V spectral ratio for strong motion differs obviously from that for weak motion. The nonlinear site response occurs if the DNL exceeds 4. Thus, the DNL is an available proxy for quantifying soil nonlinearity (Wang et al. 2021) and is adopted here.

In this article, the Fourier amplitude spectrum is smoothed by the Parzen window of 0.4 Hz. Then, the acceleration responses at the mat surface Node 500 of the SPSI system obtained using the one-step method are extracted. To ensure the simulated weak-motion results are in the linear range of soil, we derive the responses at Node 500 using the bedrock motions of 5 cm/s² and 10 cm/s² by scaling three scenario earthquake records (see Table 1). Figure 6 compares the mean H/V spectral ratio against frequency curves at Node 500 for weak motions with those for strong motions (the SL-1 and SL-2 levels). Each grey line in Fig. 6 denotes the mean curve of the weak-motion H/V spectral ratio versus frequency for the six calculation cases in Table 5. Each of the blue (or red) lines in Fig. 6 denotes the curve of the strong-motion H/V spectral ratio versus frequency.

As shown in Fig. 6, a strong positive correlation exists between DNL and scenario bedrock motion level. The DNL values for tridirectional shakings at the SL-1 and SL-2 levels are greater than those for EW+UD and NS+UD bidirectional shakings. For both bidirectional and tridirectional shakings, the DNL for the OITH10 record is the largest, followed by the SUCHIL record and the FKSH21 record. In addition, the DNL for the NS+UD bidirectional shaking (the Y-Z cross-section) is greater than those for the EW+UD bidirectional shaking (the X-Z cross-section).

The simulation results imply that the site has a significant nonlinear soil response subjected to the three scenario earthquakes at the SL-1 and SL-2 levels. Furthermore, the non-
linear effect of site soil under the moderate-far field strong earthquake shakings is most significant, followed by the far-field large earthquake shakings and most insignificant under the near-field moderate-strong earthquake shakings.

5.2 Amplification of foundation input motion

The nonlinear site responses (site effect) due to strong earthquake shakings are mainly controlled by the stiffness and the hysteresis nonlinearity of near-surface soils. The responses at the mat surface Node 500 arising from the SPSI system subjected to tridirectional earthquake motion scenarios at “seismic” bedrock level may be regarded as the foundation input motions of the nuclear-island building, obtained via the one-step method. They are different from the free-field surface motions at the corresponding Node 500 under the tridirectional bedrock motion scenarios. Naturally, they are also different from the bedrock input motion scenarios. A term, surface-to-bedrock spectral acceleration ratio $SAR_{FFSM/BIM}$, is introduced to quantify the near-surface de-amplification and amplification of seismic wave propagation in the plant site:

$$SAR_{FFSM/BIM} = \frac{5\% \text{ damping spectral acceleration of the free-field surface motion}}{5\% \text{ damping spectral acceleration of the bedrock input motion}}$$

(4)

$SAR_{FIM/BIM}$ is introduced herein to quantify the difference between the foundation input motions and the bedrock input motions. This is to account for the coupling influences of the nonlinear site effect and the SPSI effect for tridirectional strong-motion scenarios due to frequency-dependent filtering/amplifying of upward earthquake waves propagated from bedrock motions:

$$SAR_{FIM/BIM} = \frac{5\% \text{ damping spectral acceleration of the foundation input motion}}{5\% \text{ damping spectral acceleration of the bedrock input motion}}$$

(5)

Similarly, $SAR_{FIM/FFSM}$ is introduced to quantify the difference between the foundation input motions and the free-field surface motions at the Node 500, which is the result of the nonlinear SPSI effect:

$$SAR_{FIM/FFSM} = \frac{5\% \text{ damping spectral acceleration of the foundation input motion}}{5\% \text{ damping spectral acceleration of the free-field surface motion}}$$

(6)

Figure 7 shows the seismograms of the foundation input motions simulated using the one-step method and those of the free-field surface motions at the SL-1 and SL-2 levels. A comparison of Figs. 1 and 7 shows that the horizontal and vertical peak acceleration amplifications or attenuations of the foundation input motions and free-field surface motions are closely related to the intensities and frequency characteristics of the bedrock input motions and the nonlinear levels of site soils. For the near-field earthquake (record FKSH21), which is rich in high-frequency contents, the horizontal peak accelerations are significantly attenuated. On the other hand, the vertical peak accelerations are moderately amplified for the foundation input motions and the free-field surface motions. For the moderate-far field earthquake (record OITH10) and the far-field earthquake (record SUCHIL), the horizontal and vertical peak accelerations of the foundation input motions and free-field surface motions are significantly amplified.
As a result, the horizontal and vertical peak accelerations of the foundation input motions and the free-field surface motions are all at the smallest under the SL-1 and SL-2 level near-field earthquakes. In contrast, the correlation between the horizontal/vertical peak accelerations of the foundation input motions and the free-field surface motions are rather complicated under the SL-1 and SL-2 level moderate-far field and far-field earthquakes. Note that the horizontal and vertical peak accelerations of the foundation input motions are always less than those of the free-field surface motions, except for the responses in the horizontal Y-axis direction corresponding to the SL-1 level of the OITH10 record.

Figure 8 shows the variations of the spectral ratios $SAR_{FFSM/BIM}$, $SAR_{FIM/BIM}$, and $SAR_{FIM/FFSM}$ with the period $T$ for the near-field, moderate-far field, and far-field earthquake shaking scenarios. It can be observed in Fig. 8 that the variation of $SAR_{FFSM/BIM}$, $SAR_{FIM/BIM}$, and $SAR_{FIM/FFSM}$ with the period $T$ are closely related to the intensities and frequency characteristics of bedrock input motions. As shown in Fig. 8(a), for the near-field shakings, the de-amplification of the short period components ($T<0.1$ s) and the amplification of the moderate-long period components ($T=0.2–1.5$ s) of horizontal strong motions in

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**Fig. 7** Comparison of the acceleration time-histories of the free-field surface motions and the foundation input motions at the surface Node 500 for three scenario earthquakes with tri-directional components at the SL-1 and SL-2 levels
the near-surface soils are significant, especially in the periods 0.82-0.95s. A resonance-like phenomena with the $SAR_{FFSM/BIM} = 6.8–9.3$ exists.

For the moderate-far field and far field shakings, the near-surface amplification of horizontal strong motions can be observed in the entire range of periods. Furthermore, for the moderate-far field shakings, the resonance-like phenomena exist near the periods $T=1.0$ and 1.6 s, with the $SAR_{FFSM/BIM} = 3.5–6.6$; and for the far-field shakings, the resonance-like phenomena exist near the periods $T=1.0$ s, 1.6 s, 3.0 s, and 5.5 s, with the $SAR_{FFSM/BIM} = 5.5–9.5$. In addition, the amplification of the components of the shorter periods ($T<1.0$ s) and the de-amplification of the components of long periods ($T>1.0$ s) of vertical strong seismic motions in the near-surface soils are generally significant. Specifically, the amplification is more significant in the near period 0.2s for the near- and far-field shakings and in the periods 0.04–0.4 s for the moderate-far field shakings.

Figure 8(b) shows the changes of horizontal $SAR_{FIM/BIM}$ versus $T$ at the longer periods ($T>0.30$ s) are very significant. Similarly, the changes of vertical $SAR_{FIM/BIM}$ versus $T$ at periods ranging from 0.06 to 0.5 s are also very substantial. The resonance-like phenomena in the horizontal and vertical $SAR_{FIM/BIM}$ versus $T$ curves are due to the nonlinear SPSI effect. Furthermore, the horizontal resonance-like frequencies at the near period 1.0 s (with the horizontal $SAR_{FIM/BIM} = 4.5–10$) are quite close to the horizontal fundamental periods of the SPSI system, which are 1.05 s in the X-axis direction and 1.19 s in the Y-axis direction, respectively. Meanwhile, the vertical resonance-like frequencies with the vertical $SAR_{FIM/BIM} = 3–4$ can be observed at the near period 0.14 s.

Figure 8(c) shows that the nonlinear SPSI effect is not monotonically filtering or amplifying, and its consequences exhibit complicated changes of frequency-dependent amplification and de-amplification. For different earthquake shaking scenarios, the differences of the horizontal and vertical $SAR_{FIM/FFSM}$ plotted against period $T$ curves are very significant; even for the same shaking scenarios, the horizontal $SAR_{FIM/FFSM}$ versus period $T$ curves in the X-axis and the Y-axis directions are also quite different due to the spatial asymmetry of nuclear-island building and the difference in bedrock input motions between the X-axis and Y-axis directions.

In summary, due to the difference between the stiffness of the pile-mat foundation and the surrounding soil, the pile-mat foundation will not obey the pattern of free-field motion deformations, resulting in differences in the foundation input motions from the free-field surface motions. Meanwhile, the flexibility of nonlinear soil and pile-mat foundation can make the dynamic characteristics of the SPSI system differ from the fixed-base superstructure, which alters the response of the SPSI system attribute to inertial interaction. Consequently, the SPSI effect can result in complex 3D wave propagation in the inhomogeneous soil site attributed to the disturbances generated by waves scattering and diffracting within and surrounding the pile-mat foundation. As a result, the wave interferences along propagation directions may cause the complex phenomena of either constructive or destructive at a single point; the wave intensity at this point will increase or decrease, correspondingly. Thus, the coupling nonlinear influence of the site effect and the SPSI effect results in significant deviations of the foundation input motions from both the free-field surface motions and the bedrock input motions.

Next, Fig. 9 shows the AP1000 standard design response spectra for various earthquake design levels (Westinghouse 2012) along with the 5% damped spectral accelerations of foundation input motions at the SL-1 and SL-2 levels and their upper envelopes. Note that
the upper envelope ordinates at periods longer than 1.0 s are mainly dominated by the moderate-far field strong earthquake scenario (OITH10 record). For the SL-1 level in Fig. 9(a), the horizontal and vertical AP1000 spectral ordinates of 100 cm/s² level are significantly lower than the upper envelope spectra. However, even the AP1000 spectral ordinates of 200 cm/s² level are still markedly lower than the upper envelope spectra at periods longer than 0.56 s (for the horizontal responses) and longer than 1.71 s (for the vertical responses). For the SL-2 level in Fig. 9(b), the AP1000 spectral ordinates of 200 cm/s² level are lower than the upper envelope spectra at periods longer than 0.15 s (for the horizontal responses) and the entire periods (for the vertical responses). Even the AP1000 spectral ordinates of 400 cm/s² level are still significantly lower than the upper envelope spectra at periods longer than 0.62 s (for the horizontal responses) and longer than 1.29 s (for the vertical responses). Compared to the site-independent AP1000 spectra, the significant conclusion is that the coupling of the nonlinear site effect and the nonlinear SPSI effect has a substantial negative influence on the seismic safety of the AP1000 nuclear-island building. The results address the importance of nonlinear SPSI analyses for evaluating the seismic behavior of a pile-mat-founded nuclear-island building.
5.3 Seismic response characteristics under tridirectional shaking

In this section, the peak horizontal acceleration and the peak relative horizontal displacement of a node of the nuclear-island building are the peak value of a vectorial combination of the two orthogonal horizontal components of the acceleration and relative horizontal displacement time series of the node, respectively.

5.3.1 Pile-mat foundation versus fixed base

For the fixed-base case, the tridirectional earthquake shaking is applied at the mat surface Node 500. In contrast, the tridirectional earthquake shaking is applied at “seismic” bedrock level for the pile-mat foundation case. The response analyses of the 3D LMS model siting on the pile-mat foundation and the fixed base are respectively performed for three scenario earthquakes with the SL-1 and SL-2 levels. For the pile-mat foundation case, the responses of the 3D LMS model are obtained via the one-step method. The response differences induced by the two seismic inputs attribute to the coupling influence of the flexibility and hysteresis nonlinearity of soil and the nonlinear SPSI.

The peak acceleration amplification factor (PAAF) of a node of the 3D LMS model is defined as the ratio of the peak acceleration of the node to the peak acceleration of the seismic input. For the fixed base case, the PAAFs of any node at the SL-1 and SL-2 levels are the same at the assumption of viscoelastic material behavior.
Figure 10 shows the differences in the peak profile responses, in terms of the horizontal and vertical PAAFs, along with the height in the pile-mat foundation and the fixed-base cases. The results indicate that the differences of PAAFs between the containment vessel and the shield building at the same height are quite small for various simulating cases. The distributions and magnitudes of peak profile responses exhibit significant differences in the pile-mat foundation and the fixed-base cases. These differences are strongly influenced by the intensities and frequency characteristics of scenario tridirectional earthquake shakings.

Compared to the fixed base cases, the PAAFs in the pile-mat foundation cases have the following features: (i) for the SL-1 and SL-2 level near-field earthquake shakings (FKSH21 record), the horizontal PAAFs are much smaller; (ii) for the SL-1 level moderate-far field (OITH10 record) and far-field (SUCHIL record) earthquake shakings, the horizontal PAAFs are much larger; (iii) the differences of horizontal PAAFs at the SL-2 level are insignificant for moderate-far field strong earthquake shaking, but they are cross-changing with height for far-field large earthquake shaking; (iv) except for the SL-2 level near-field earthquake shaking, the vertical PAAFs are always much larger. In addition, in the pile-mat foundation cases, it is worth noting that, regardless of the SL-1 or SL-2 level, both the horizontal and vertical PAAFs under the near-field earthquake shaking are the smallest, followed by the moderate-far field earthquake shaking, the largest for the far-field earthquake shaking. Therefore, the horizontal and vertical PAAFs in the pile-mat foundation cases are dominated by the frequency characteristics of scenario earthquake shakings. The correlation between the PAAFs in the pile-mat foundation and the fixed cases could be explained by the amplifying/filtering effect of the coupled influence of the nonlinear site effect and the nonlinear SPSI effect on different frequency characteristics of scenario earthquake shakings.

In addition, the relative displacement of any node of the 3D LMS model is defined as the peak value of the displacement time-history difference between the node and the mat surface Node 500. For the pile-mat foundation and the fixed cases, the peak relative vertical displacements of the containment vessel and the shield building at the SL-1 and SL-2 levels are all less than 1.45 mm and thus are not discussed here. Figure 11 compares the peak relative horizontal displacements (PRHDs) for the pile-mat foundation with those for the fixed base. It is worth noting that, either the pile-mat foundation case or the fixed base case, the PRHD differences at the similar height for the containment vessel and the shield building are insignificant. Whether the SL-1 or SL-2 level (or the pile-mat foundation case or the fixed base case) is considered, the PRHDs are always the smallest under the near-field earthquake and the largest for the far-field earthquake. Compared to the fixed base cases, the PRHDs in the pile-mat foundation case are much smaller for the near-field earthquake shaking, but they are close or significantly greater for moderate-far field and far-field earthquake shakings.

In brief, compared to the earthquake inputs based on the assumption of a fixed base, the above results highlight the significant amplifying influences that the nonlinear site effect and the nonlinear SPSI effect exert on the peak acceleration and peak relative displacement responses of the pile-mat-founded nuclear-island building subjected to the moderate-far field and far-field earthquake shaking scenarios.
Fig. 10 Comparison of the peak acceleration amplification factors (PAAFs) along the direction of the height of the nuclear-island building for the pile-mat foundation and the fixed base cases, under tri-directional earthquake shaking scenarios for: (a) FKSH21 record; (b) OITH10 record; and (c) SUCHIL record

5.3.2 One-step method versus two-step method

The response analyses of the 3D LMS model sitting on the pile-mat foundation under tri-directional earthquake scenarios are performed using both the one-step and two-step methods. The results are then compared to evaluate the influence of the secondary nonlinearity of soil.

The influence coefficient of peak acceleration (IRPA) induced by the secondary nonlinearity of soil is defined as follows:

$$ICPA = 10.0 \cdot \frac{\text{Peak acceleration calculated by the two-step method}}{\text{Peak acceleration calculated by the one-step method}}$$

Similarly, the influence coefficient of peak relative displacement (ICPRD) induced by the secondary nonlinearity of soil is defined as follows:

$$ICPRD = 10.0 \cdot \frac{\text{Peak relative displacement calculated by the two-step method}}{\text{Peak relative displacement calculated by the one-step method}}$$

Figures 12 and 13 show the variations of $ICPA$s and horizontal $ICPRD$s along the direction of the vertical height of the nuclear-island building for the three tri-directional earthquake
shaking scenarios with the SL-1 and SL-2 levels, respectively. As shown in Fig. 12, the horizontal ICPAs increase with the scenario earthquake shaking level, whereas the vertical ICPAs are just the opposite. Generally, the horizontal ICPAs at the SL-1 and SL-2 levels are always the largest under the near-field earthquake scenario, followed by the far-field earthquake scenario, and the smallest under the moderate-far field earthquake scenario. However, the opposite is true for the vertical ICPAs. Figure 13 shows that the horizontal ICPRDs decrease with the earthquake shaking level.
Moreover, the horizontal ICPRDs at the SL-1 and SL-2 levels are always the largest under the near-field earthquake and the smallest under the moderate-far field earthquake scenarios. In addition, the peak relative vertical displacements of the containment vessel and the shield building simulated at SL-1 and SL-2 levels calculated using the two-step method are small and less than 1.35 mm. Thus, its discussion is omitted in this article.

In brief, the values of both ICPA and horizontal ICPRD are all positive, and their respective maximum values may be as high as 30% or even higher. The implication is that the secondary nonlinearity of soil markedly aggravates the seismic responses of the pile-mat-founded nuclear-island building. This effect is very complicated and strongly affected by the intensity and frequency characteristics of scenario bedrock motions.

5.4 Seismic response differences under bidirectional and tridirectional shakings

The responses of the 3D LMS model sitting on the pile-mat foundation under bidirectional (horizontal and vertical) and tridirectional scenario earthquakes with the same intensity can be obtained via the one-step method. Typically, this difference is attributed to the out-of-plane earthquake shaking (corresponding to the bidirectional shaking) under the tridirectional shaking, which is the most significant. Hence, the focus is placed on the response difference in the out-of-plane direction of the bidirectional shaking.

The out-of-plane peak acceleration ratio (OPAR) is the ratio of the out-plane peak acceleration of a node of the 3D LMS model under the bidirectional earthquake shaking to the peak acceleration in the same horizontal direction under tridirectional earthquake shaking. Figure 14 shows the variation of OPARs at the X-axis and Y-axis directions with the height of nuclear-island building. Regardless of the intensities and frequency characteristics of scenario earthquake shakings, the OPARs at the Y-axis direction are always more significant than those at the X-axis direction due to the spatial asymmetry of the nuclear-island building (including the pile-mat foundation) and the difference of bedrock input motions at the X-axis and Y-axis directions. In addition, the differences of OPARs between the containment vessel and the shield building are insignificant, and both the OPARs at the X-axis and Y-axis directions increase with the level of scenario earthquake shakings.

Similarly, the out-of-plane peak relative displacement ratio (OPRDR) is the ratio of the out-plane peak relative displacement of a node of the 3D LMS model under the bidirectional earthquake shaking to the peak relative-displacement in the same horizontal direction under
tridirectional earthquake shaking. Figure 15 shows the variations of $OPRDR$s at the X-axis and the Y-axis directions with the height of nuclear-island building. As shown in Fig. 15, the distributions and magnitudes of $OPRDR$s along height are closely related to the intensities and frequency characteristics of scenario earthquake shakings and the $OPRDR$s of the containment vessel and the shield building at the similar height are roughly the same at the SL-1 and SL-2 levels.

In summary, a notable finding in Figs. 14 and 15 is that the $OPAR$s and $OPRDR$s at various heights are consistently much smaller than 1.0. The implication is that, compared to the tridirectional earthquake shaking scenarios, the seismic design of the pile-mat-founded nuclear-island building based on the bidirectional (horizontal and vertical) earthquake shaking scenarios exhibits a substantial negative influence on its seismic safety.

6 Summary and conclusions

This article presents the development, implementation, and application of an integrated simulation method for nonlinear soil-pile-superstructure interaction (SPSI) analyses of an AP1000 nuclear-island building on the ABAQUS/Explicit platform, subjected to multidirectional strong motions. The proposed model is applied to study the seismic responses of an actual pile-mat-founded AP1000 nuclear-island building. The influences of foundation flexibility, soil secondary nonlinearity, and the dimension differentiation of seismic input are investigated. The key conclusions are summarized as follows:

1. The coupling influence of nonlinear site effect and SPSI effect is complicated for scenario earthquakes due to the frequency-dependent filtering/amplifying effect of upward seismic waves from bedrock. Due to the nonlinear SPSI effect, the horizontal
and vertical components of the foundation input motion of the nuclear island-building exhibit the resonance-like phenomenon at the period of 1.0 s and 0.14 s, respectively.

2. Compared to the site-independent AP1000 spectra, the coupling influence of nonlinear site effect and nonlinear SPSI effect exhibits an amplification at longer periods and de-amplification at short periods for the spectra accelerations of the tridirectional foundation input motions, which significantly aggravates the seismic responses of a pile-mat-founded nuclear-island building under the moderate-far field and far-field strong-motion scenarios.

3. The coupling effects of nonlinear site effect and nonlinear SPSI on the seismic responses of the pile-mat-founded nuclear-island building are complex, which could be significantly affected by the characteristics of scenario earthquakes. Compared with the pile-mat foundation case, the seismic input based on the fixed-base assumption may substantially underestimate the seismic responses of pile-mat-founded nuclear-island buildings under scenario earthquakes with abundant long period components. For the peak responses in the case of pile-mat foundation, both the horizontal and vertical peak accelerations and peak relative horizontal displacements are the smallest for the scenario earthquakes with abundant short period components and shorter durations.

4. The influence of secondary nonlinearity of soil on the seismic responses of the pile-mat-founded nuclear-island building is complicated and closely related to the intensities and frequency characteristics of scenario bedrock motions. This influence increases the peak accelerations and peak relative displacements of the nuclear-island building by 30% or more.

5. The seismic design of the pile-mat-founded nuclear-island building based on the input of bidirectional (horizontal and vertical) strong motions has a substantial negative influence on its seismic safety compared to that based on the tridirectional
strong-motion scenarios, indicating that the input mode of tridirectional motions should be recommended.

Finally, it should be noted that the above conclusions were generalized via the study of three representative earthquake scenarios (array records) with their specific features in frequency characteristics and durations. Further studies under different strong motion records are warranted.

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**Data Availability** The data used in the manuscript are available upon request.

**Code Availability** Not applicable.

**Declarations**

**Conflict of interest** The authors declare that there is no conflict of interest.

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