Numerical study of pile group effect on wave-induced seabed response

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1 **Abstract:** Wave-induced seabed response around pile foundations affects the seabed
2 stability, thereby threatening the structure safety, and numerous studies have been
3 performed to explore the wave-structure-seabed interaction (WSSI) around mono-pile.
4 However, little attention has been paid to WSSI around pile groups. In the present study,
5 based on an integrated model, WSSI around three-pile group is numerically examined,
6 and the effects of wave obliquity, pile diameter, pile spacing and embedment depth are
7 parametrically explored. In this model, Reynolds-Averaged Navier-Stokes (RANS)
8 equations with k - ε turbulence model and Biot's quasi-static (QS) model for poro-elastic
9 medium are adopted to govern the wave motion and seabed response, respectively. The
10 results reveal that the effects of wave obliquity and pile diameter are significant, whereas
11 the influences of pile spacing and embedment depth are negligible.
12
13 *Keywords:* Pile group effect; Three-pile group; WSSI; RANS equations; QS Biot's
14 model; Oscillatory pore pressure
15

1. Introduction

Pile foundations are widely applied in coastal engineering practice (e.g., offshore wind turbines [1–3], sea bridges [4] and petroleum platforms [5]). Unlike the inland ones, offshore pile foundations are more vulnerable due to various oceanic loads [6], among which wave loading may generate excessive pore water pressure in seabed, leading to seabed liquefaction or shear failure [7], and thereby affecting the structure safety.

There are two mechanism of wave-induced pore pressure in seabed, i.e. the oscillatory mechanism [8,9] and residual mechanism [10,11]. In most cases, the wave-induced seabed response is oscillatory, except for some special cases of non-cohesive sediments with loose to medium density [12,13]. For the wave-induced oscillatory seabed response, numerous numerical models were established based on the framework of Biot's theories for poro-elastic medium [14–16], including the quasi-static (QS) model, partly dynamic (PD) model and fully dynamic (FD) model. In QS model, the inertia terms of fluid and soil particles are both ignored, while only that of fluid is neglected in FD model. To identify the applicability of these three models, Ulker et al. [17] developed a set of analytical solutions to free seabed response under plane strain condition. Based on their results, Sumer [18] summarized that for low permeability soil (permeability $k = 1 \times 10^{-5}$ m/s), no inertia effect exists, and even for a large permeability soil ($k = 1 \times 10^{-2}$ m/s, gravel actually), the difference between the results of FD (PD) and QS model is less than 5%. Therefore, for most engineering problems, the inertia effects can be ignored as revealed by Cheng and Liu [19], and QS model could be served as a cost-effective tool in numerical simulation.

In the past decade, numerous numerical studies have been carried out to examine the wave-induced oscillatory seabed response around mono-pile foundations. Based on boundary element method (BEM), Lu and Jeng [20] developed a coupled model to study the linear wave-induced seabed response and mono-pile deformation with seawater described by Helmholtz equation. Li et al. [21] proposed a three-dimensional (3D) seabed-pile coupled model using finite element method (FEM) to simulate the wave-induced oscillatory and residual pore pressure response, wherein analytical wave pressure is applied on seabed surface with Stokes wave theory, and effects of soil

1 permeability and pile diameter are parametrically studied. Sui et al. [22] developed
2 another one-way coupled model to simulate the dynamic oscillatory seabed response
3 with wave motion described by nonlinear Boussinesq equations, and found that
4 maximum pile displacement is less than 4×10^{-6} times the pile radius. Zhang et al. [23]
5 carried out 3D one-way coupling studies to examine the effect of mono-pile on wave-
6 induced oscillatory seabed response. In their model, RANS equations solved by finite
7 volume method (FVM) are adopted to govern wave motion and it is revealed that
8 existence of mono-pile foundation would decrease the seabed response around mono-
9 pile. Then Zhang, based on fully dynamic (FD) Biot's equations and linear wave theory,
10 Zhang et al. [24] studied the non-homogenous seabed response around mono-pile.
11 Recently, Lin et al. [25] developed an integrated 3D model to explore strong nonlinear
12 wave-induced seabed response around mono-pile with the framework of OpenFOAM,
13 and parametric study of embedded depth shows that increasing embedded depth
14 significantly reduces the magnitude of pore pressure along mono-pile foundation. Then
15 Zhao et al. [26] extended this study to enclose the residual seabed response with the
16 formulation proposed by Sassa et al. [27].

17 However, exploration on wave-induced seabed response around pile-group foundation
18 commenced relatively later, and is significantly scarce up to date. Sui et al. [28] provided
19 a preliminary work on the wave-induced seabed response around four-square-pile group
20 and studied the influence of degree of saturation and permeability on seabed response.
21 Chang and Jeng [29] further performed a case study to explore the wave-induced seabed
22 response around high-rising structure foundation of Donghai Offshore Wind Farm, which
23 is composed of eight inclined steel piles and a concrete cap, and it is demonstrated that
24 replacing seabed soils around pile foundation with coarser ones offers effective
25 protection against liquefaction. Recently, Zhang et al. [30] carried out a 3D numerical
26 simulation to examine the wave-induced seabed response around a four-pile platform
27 considering structure response. In this study, parametric studies were conducted to
28 explore the effects of pile diameter and pile insertion ratio on seabed and structure
29 response.

30 It is well known that pile group may have numerous configurations, and pile group

effect on wave-induced seabed response is a primary concern, which involves the pile diameter, pile spacing, embedment depth, wave obliquity, etc. For purpose of optimizing pile group configuration, it is vital to explore the effects of these factors on seabed response.

Due to the diversity of pile group formation, it is unrealistic to carry out study for each configuration, and it is expedient to study several typical configurations, among which three-pile group is the simplest and a common type of pile group in offshore practice [2,31,32]. Numerous studies have been performed to investigate the nearby local scour development [33–35] and the wave force acting on three-pile groups and the superstructures [36–38]. However, little attention has been paid to the wave-induced seabed response around three-pile group.

In the present study, parametric study around three-pile groups will be numerically conducted to explore the pile group effect on wave-induced seabed response with regard to wave obliquity, pile diameter, pile spacing and embedment depth. RANS equations with $k-\varepsilon$ turbulence closure are employed to describe the wave-induced fluid motion. Following the instructions of Sumer [18] as aforementioned, Biot's QS theory is adopted to capture the seabed response. The wave and seabed modules are solved by finite difference method (FDM) and FEM, respectively.

2. Numerical model

The present numerical model is composed of wave sub-model and seabed sub-model, which are governed by RANS equations and QS (quasi-static) Biot's equations for poro-elastic medium [14], respectively. In the previous studies, these two sets of governing equations have been adopted in the examination of wave-induced seabed response around pile foundations [25,26,29], breakwaters [39], and pipelines [40,41]. The integration method of these two sub-models will be also demonstrated in this section. Fig. 1 illustrates the definition of WSSI in the vicinity of three-pile group. Ye and Jeng [42] suggested that the seabed model length should be at least two times the incident wavelength to diminish the effect of fixed boundaries in case of free seabed under wave and current combined loading. In the present model, the model size is set as $4L_w \times 2L_w$, where L_w is the incident wavelength. The origin of Cartesian coordinates, O, is located

at the centre of the pile group on the seabed surface. The wave incident angle, θ , ranges from 30° to 90° .

[Fig. 1 is here.]

2.1. Wave sub-model

For incompressible fluid motion, mass and momentum conservations are shown as follows, respectively:

$$\frac{\partial \langle u_{fi} \rangle}{\partial x_{fi}} = 0 \quad (1)$$

$$\begin{aligned} \frac{\partial \rho_f \langle u_{fi} \rangle}{\partial t} + \frac{\partial \rho_f \langle u_{fi} \rangle \langle u_{fj} \rangle}{\partial x_j} = & -\frac{\partial \langle p_f \rangle}{\partial x_i} + \frac{\partial}{\partial x_j} \left[\mu \left(\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right) \right] \\ & + \frac{\partial}{\partial x_j} (-\rho_f \langle u'_{fi} u'_{fj} \rangle) + \rho_f g_i \end{aligned} \quad (2)$$

where $\langle u_{fi} \rangle$ ($i = 1, 2, 3$) is the ensemble mean fluid velocity in x , y and z direction, respectively; u'_{fi} is the fluctuating velocity; $\langle p_f \rangle$ is the fluid pressure; μ is the dynamic viscous; x_{fi} ($i = 1, 2, 3$) is the coordinate x , y and z , respectively; ρ_f is the fluid density; t is the time and g_i ($i = 1, 2, 3$) is the body force in x -, y - and z -direction, respectively. In the present study, the only body force is gravity, $g = 9.80665 \text{ m/s}^2$.

The term $-\rho_f \langle u'_{fi} u'_{fj} \rangle$ in Eq. (2) is the so-called Reynolds stress, which can be estimated by eddy viscosity model:

$$-\rho_f \langle u'_{fi} u'_{fj} \rangle = \mu_t \left[\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right] - \frac{2}{3} \left(\rho_f k + \mu_t \frac{\partial \langle u_i \rangle}{\partial x_i} \right) \delta_{ij} \quad (3)$$

where μ_t is the turbulent viscosity, $k = \frac{1}{2} \langle u'_{fi} u'_{fi} \rangle$ is the turbulent kinematic energy and δ_{ij} is the Kronecker delta:

$$\delta_{ij} = \begin{cases} 1, & i = j \\ 0, & i \neq j \end{cases} \quad (4)$$

By substituting Eq. (3) into Eq. (2), Eq. (2) could be expressed by

$$\begin{aligned} \frac{\partial \rho_f \langle u_{fi} \rangle}{\partial t} + \frac{\partial \rho_f \langle u_{fi} \rangle \langle u_{fj} \rangle}{\partial x_j} = & -\frac{\partial \langle p_f + \frac{2}{3} \rho_f k \rangle}{\partial x_i} + \frac{\partial}{\partial x_j} \left[\mu_{eff} \left(\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right) \right] \\ & + \rho_f g_i - \frac{2}{3} \mu_t \frac{\partial}{\partial x_j} \left[\frac{\partial \langle u_i \rangle}{\partial x_i} \delta_{ij} \right] \end{aligned} \quad (5)$$

where $\mu_{eff} = \mu + \mu_t$ is the total effective viscosity.

In order to solve Eq. (1) and Eq. (5), k - ε turbulence model [43] is adopted. Firstly, a variable called turbulent dissipation rate is introduced into the model:

$$\varepsilon = \frac{\mu}{\rho_f} \overline{\left(\frac{\partial u'_{fi}}{\partial x_k} \right) \left(\frac{\partial u'_{fi}}{\partial x_k} \right)} \quad \P(6)$$

Hence a connection between ε and turbulent kinematic energy k could be established as

$$\varepsilon = C_D \frac{k^{\frac{2}{3}}}{l} \quad \P(7)$$

in which C_D is a constant, l is a length representing the macroscale of turbulence.

Eventually, the k - ε turbulence model is derived as

$$\frac{\partial(\rho_f k)}{\partial t} + \frac{\partial(\rho_f k \langle u_{fi} \rangle)}{\partial x_i} = \frac{\partial}{\partial x_j} \left[\left(\mu + \frac{\mu_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} \right] + G_k - \rho_f \varepsilon \quad \P(8)$$

$$\frac{\partial(\rho_f \varepsilon)}{\partial t} + \frac{\partial(\rho_f \varepsilon \langle u_{fi} \rangle)}{\partial x_i} = \frac{\partial}{\partial x_j} \left[\left(\mu + \frac{\mu_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_j} \right] + \frac{C_{1\varepsilon}}{k} G_k - C_{2\varepsilon} \rho_f \frac{\varepsilon^2}{k} \quad \P(9)$$

$$\mu_t = \frac{C_\mu \rho_f k^2}{\varepsilon} \quad \P(10)$$

$$G_k = \mu_t \left(\frac{\partial \langle u_i \rangle}{\partial x_j} + \frac{\partial \langle u_j \rangle}{\partial x_i} \right) \frac{\partial \langle u_i \rangle}{\partial x_j} \quad \P(11)$$

where the values of constants $\sigma_k, \sigma_\varepsilon, C_{1\varepsilon}, C_{2\varepsilon}, C_\mu$ are as follows: $\sigma_k = 1.00, \sigma_\varepsilon = 1.30, C_{1\varepsilon} = 1.44, C_{2\varepsilon} = 1.92, C_\mu = 0.09$ [44].

To diminish the wave reflection from the outflow boundary, a sponge layer is adopted to absorb waves in front of the outlet boundary, in which the Navier-Stokes equation is modified as

$$\frac{\partial \vec{u}}{\partial t} + \vec{u} \cdot \nabla \vec{u} = -\frac{1}{\rho} \nabla p + \nabla \cdot \nabla (\mu \vec{u}) - k_d (\vec{u} - \vec{u}_{str}) \quad \P(12)$$

where $-k_d (\vec{u} - \vec{u}_{str})$ is the artificial damping force that dissipates the wave motion, k_d is the damping coefficient in units of (time)⁻¹, and $\vec{u}_{str}$ is the background stream velocity that is exempted from damping. The coefficient k_d is estimated by

$$k_d = k_{d0} + l_d \cdot \frac{k_{d1} - k_{d0}}{d_{sl}} \quad \P(13)$$

in which k_{d0} and k_{d1} ($k_{d1} \geq k_{d0}$) are the values of k_d at the starting side of the sponge layer and the open boundary, respectively. The distance l_d is measured from the starting side of the wave-absorbing layer toward the open boundary. Finally, d_{sl} is the length of the sponge layer. In the present study, $k_{d0} = 0$, $k_{d1} = 1$, and $l_d = d_{sl} = 2L_w$, where L_w is the

1 incident wavelength.

2 The free surface elevation is captured using Volume-of-Fluid (VOF) method [45], in
3 which a function F is defined to characterize the fractional volume water fluid in a cell

$$4 \quad \frac{\partial F}{\partial t} + u_{fi} \frac{\partial F}{\partial x_i} = 0 \quad \P(14)$$

5 where $F = 0$ indicates that the cell is full of air, and $F = 1$ means that the cell is full of
6 water. Otherwise, the piecewise linear interface calculation method is applied to
7 reconstruct the air water interface [46].

8 Linear wave is applied at the inlet boundary to generate the incident wave as well as
9 outflow boundary with Sommerfeld radiation method is applied at the outlet boundary.
10 At the seafloor, no-slip boundary is employed at the boundary with a zero velocity normal
11 to the boundary. Specified fluid elevation is applied as the initial condition.

12 The wave sub-model is realized in FLOW-3D, and is solved by FDM. The whole wave
13 domain is divided into a series of hexahedron cells in the vicinity of the pile group. All
14 variables are located at the centers of the cells except for velocities, which are located at
15 cell-faces. The mesh size is 0.2 m in x - and y -direction, and is 0.12 m in z -direction, as
16 shown in Fig. 2(a). Total cell number is 7, 850, 040.

17 *[Fig. 2 is here.]*

18 2.2 Seabed sub-model

19 In line with most previous studies, the seabed soil are assumed to be isotropic
20 homogeneous with identical permeability in all directions. Hence, the mass conservation
21 is expressed as

$$22 \quad \nabla^2 p_s - \frac{n_s \beta_s \gamma_w}{k_s} \frac{\partial p_s}{\partial t} - \frac{\gamma_w}{k_s} \frac{\partial \varepsilon_s}{\partial t} = 0 \quad \P(15)$$

23 in which $\nabla^2 = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$ is the Laplace operator, p_s is the pore pressure, n_s is the
24 soil porosity, β_s is the compressibility of pore fluid, γ_w is the unit weight of pore water,
25 and ε_s is the volume strain. β_s and ε_s are defined as follows, respectively:

$$26 \quad \varepsilon_s = \frac{\partial u_s}{\partial x} + \frac{\partial v_s}{\partial y} + \frac{\partial w_s}{\partial z} \quad \P(16)$$

$$27 \quad \beta_s = \frac{1}{K_w} + \frac{1-S_r}{P_{w0}} \quad \P(17)$$

28 where (u_s, v_s, w_s) are the soil displacements in x -, y -, and z -direction respectively, K_w is

1 the true elastic modulus of water (taken as 2.0×10^9 Pa) and keeps as a constant, S_r is the
 2 seabed degree of saturation, and P_{w0} is the absolute water pressure.

3 According to Terzaghi's principle of effective stress, the total soil stress (σ_{ij}) is
 4 composed of effective stress (σ'_{ij}) and pore pressure (p_s)

$$5 \quad \sigma_{ij} = \sigma'_{ij} + \delta_{ij} p_s \quad \P(18)$$

6 where δ_{ij} is the Kronecker delta.

7 Leaving out the body force and inertia terms, the equilibrium conditions could be
 8 expressed as

$$9 \quad G \nabla^2 u_s + \frac{G}{(1-2\nu)} \frac{\partial \varepsilon_s}{\partial x} = \frac{\partial p_s}{\partial x} \quad \P(19)$$

$$10 \quad G \nabla^2 v_s + \frac{G}{(1-2\nu)} \frac{\partial \varepsilon_s}{\partial y} = \frac{\partial p_s}{\partial y} \quad \P(20)$$

$$11 \quad G \nabla^2 w_s + \frac{G}{(1-2\nu)} \frac{\partial \varepsilon_s}{\partial z} = \frac{\partial p_s}{\partial z} \quad \P(21)$$

12 in x -, y - and z -direction, respectively, in which G is the shear modulus and ν is the
 13 Poisson's ratio.

14 In the present study, the poro-elastic constitutive relationship is applied to describe the
 15 seabed soil behavior. Hence, the effective normal stresses and shear stresses can be
 16 expressed in terms of soil displacements as

$$17 \quad \sigma'_x = 2G \left[\frac{\partial u_s}{\partial x} + \frac{\nu \varepsilon_s}{1-2\nu} \right], \quad \tau_{xy} = \tau_{yx} = G \left[\frac{\partial u_s}{\partial y} + \frac{\partial v_s}{\partial x} \right] \quad \P(22)$$

$$18 \quad \sigma'_y = 2G \left[\frac{\partial v_s}{\partial x} + \frac{\nu \varepsilon_s}{1-2\nu} \right], \quad \tau_{yz} = \tau_{zy} = G \left[\frac{\partial v_s}{\partial z} + \frac{\partial w_s}{\partial y} \right] \quad \P(23)$$

$$19 \quad \sigma'_z = 2G \left[\frac{\partial w_s}{\partial z} + \frac{\nu \varepsilon_s}{1-2\nu} \right], \quad \tau_{zx} = \tau_{xz} = G \left[\frac{\partial w_s}{\partial x} + \frac{\partial u_s}{\partial z} \right] \quad \P(24)$$

20 Equations (15), (19-24) with appropriate boundary conditions, are capable of
 21 determining the wave-induced pore pressure, displacements and effective stresses in a
 22 porous seabed.

23 Wave-induced pressure (p_{wv}) is calculated from the wave sub-model:

$$24 \quad p_{wv} = p_f - \gamma_w d_w \quad \P(25)$$

25 where d_w is the water depth. Then wave pressure is applied on seabed surface to exert
 26 wave loading:

$$27 \quad p_s = p_{wv} \quad \P(26)$$

1 The four lateral sides and seabed bottom are set as impermeable fixed boundaries:

$$2 \quad u_s = v_s = w_s = \frac{\partial p_s}{\partial n} = 0 \quad \P(27)$$

3 where $\mathbf{n}$ denotes the normal vector of the boundaries. In the present study, the three piles
4 are considered to be impermeable rigid bodies. Therefore, the interfaces between three
5 piles and seabed are impermeable boundaries:

$$6 \quad \frac{\partial p_s}{\partial n} = 0 \quad \P(28)$$

7 The seabed sub-model is established in COMSOL Multiphysics and is solved with
8 FEM. The seabed domain is discretised into 90653 tetrahedral elements with refined
9 mesh in the vicinity of three piles for configuration of $\theta = 30^\circ$, as shown in Fig. 2(b). The
10 element size is 0.334-4.6 m with maximum element growth rate 1.4, the curvature factor
11 is 0.4, and the resolution of narrow regions is 0.7. For the other configurations, the
12 amount of element is slightly different.

13 *2.3. Integration process of sub-models*

14 As have been pointed out by Sui et al. [47], the movement of pile is very small. Hence,
15 it has little effect on wave propagation, and correspondingly this study adopts the so-
16 called one-way coupling method [23] to integrate the wave sub-model and seabed sub-
17 model. At each time step, FDM is employed to solve the RANS equations in wave sub-
18 model and to capture the wave pressure acting on seabed. Then the wave pressure is
19 introduced into the seabed sub-model to apply the wave loading on seabed, as shown in
20 Eq. (26). Finally, FEM is employed to solve the Biot's QS equations in the seabed sub-
21 model with boundary conditions mentioned above, and seabed response including
22 oscillatory pore pressure and momentary seabed liquefaction is obtained.

23 **3. Model validation**

24 The wave sub-model and seabed sub-model will be validated against available
25 experiment data in the literature, respectively.

26 *3.1. Wave sub-model validation*

27 The wave sub-model is validated against experiment data of Zang et al. [3], which
28 have been used to validate the wave sub-model of Lin et al. [25] built in OpenFOAM. In
29 the experiment, a mono-pile with a diameter of 0.25 m is set in a wave flume of 35 m $\times$
30 25 m. The mono-pile is 7.52 m from the wave paddle with a water depth of $d_w = 0.505$

m, more details could be found in Zang et al. [3] and Chen et al. [48]. Two types of incident wave are used in the experiment, among which the incident wave of 1.22 s period and 0.14 m wave height is used in the present study to validate the wave sub-model.

Fig. 3 displays the simulated time series of free surface elevation at two positions against the experiment data. Wave gauge 1 and 2 are located 0.77 m from the wave paddle and 0.002 m in front of the mono-pile along the centreline, respectively. It is shown that at both locations, the simulated free surface elevation reaches a good agreement with the experiment data. This demonstrates that the present model has the capacity to simulate the behaviour of wave-structure interaction.

[Fig. 3 is here.]

3.2. Seabed sub-model validation

The seabed sub-model is validated against the experiment data of Tsui and Helfrich [49] and Liu et al. [50].

In the experiment of Tsui and Helfrich [49], a concrete wave tank ($24.7\text{ m} \times 0.6\text{ m} \times 0.92\text{ m}$) was employed with a bottom-fixed wave paddle and a flared section. The wave paddle top could oscillate to create 25.4 mm-89 mm high wave of 0.7-4.5 sec period. A box of $1.7\text{ m} \times 0.61\text{ m}$ is filled with Chattahoochee sand and located 13 m from the wave generator and its height is variable to enable simulation of various seabed depth. Pore pressure variation in the sand was measured using two pressure transducers. The experiment data of this study have been adopted to verify existing numerical models such as Sui et al. [22] and Zhang et al. [51] with appropriate values assigned to corresponding seabed parameters. Fig. 4 shows the vertical distribution of simulated pore pressure against the measured data. It is shown that the present model reaches a good agreement with the experiment data.

[Fig. 4 is here.]

Liu et al. [50] conducted a series of one-dimensional (1D) experiments to monitor wave-induced excessive pore pressure in sandy deposits. In the experiment, a cylindrical soil model facility of 205 mm inner diameter and 1.8 m height is installed with 0.2 m water above it. Cyclic air pressure is applied on the water to provide the wave pressure with a rubber airbag. Fig. 5 displays the time series of wave-induced pore pressure at two

depths against the experiment results. It is observed that the seabed sub-model presents an excellent agreement with the experiment.

[Fig. 5 is here.]

The above validation demonstrates that the seabed sub-model has the capacity for simulating seabed response under wave loading.

4. Results and discussion

The effects of wave obliquity, pile diameter, pile spacing and embedment depth will be explored in this section. Zhang et al. [23] have concluded that the existence of mono-pile would increase the wave-induced pore pressure around pile foundation, and would alleviate the seabed liquefaction around mono-pile foundation. In the present study, the existence of the pile group would likewise generate effect on seabed. However, the effect of the piles cannot be simply superposed due to the interaction between three piles via water wave and seabed soil.

The pile group effect influences the seabed response between three piles. Due to the simultaneous effect of side-by-side interaction and blockage of the front pile(s) on the lee-side, the pile group effect between the three piles (e.g., location O in Fig. 1) should be the most significant. Therefore, wave pressure and seabed response between three piles are supposed to serve as probes to examine the pile group effect. In the present study, the wave pressure and seabed response at location O are displayed in the following parametric studies (Subsection 4.2-4.5).

In this section, the wave pressure (p_{wv}) is the difference of total hydraulic pressure (p_f) and hydrostatic pressure ($\gamma_w d_w$), as defined in Eq. (25). The seabed response presented (i.e. pore pressure p_s , vertical effective stress σ'_z and shear stress τ_{xz}) in this section corresponds to the wave pressure p_{wv} .

The input data of the present study are tabulated in Table 1.

[Table 1 is here.]

4.1. Basic features of WSSI around three-pile group

In the present study, the configuration of wave incident angle $\theta = 30^\circ$ is taken as the standard case with a pile diameter $D = 2$ m, and the pile spacing $d = 3D = 6$ m. In this subsection, some basic characteristics of WSSI around three piles will be firstly

presented. The wave loading gives rise to seabed response, thereby would be presented first, followed by the results of seabed response.

Fig. 6 shows the time series of wave pressure (p_{wv}) in the vicinity of three piles, including the frontal side of the upstream pile (Location A), the lateral sides of three piles (Locations B and E), region between three piles (Location O) and the lee side of the downstream piles (Location F). It is seen that, during the propagation, the amplitude of wave pressure moderates along the centrelines of the piles (ACO, DF). Besides, magnitude of wave pressure at the lateral sides of the upstream pile (B) is greater than the downstream piles (E), but both are smaller than wave pressure along the centrelines (ACO and DF, respectively). This implies that wave energy dissipates during propagation due to fluid-structure interaction, and the pile group renders significant effect on wave motion. It is also shown that the magnitude of negative wave pressure at O tends to be the largest. With respect to oscillatory seabed response, it is well-recognized that, under the wave troughs, pore water pressure generates an uplift force on the soil grains in the vertical direction[18]. Thus, it could be inferred that, with the simultaneous effect of the blockage from the upstream pile and the side-by-side effect of the downstream piles, the zones between three piles suffers the most severe threat of liquefaction.

[Fig. 6 is here.]

The seabed response, mainly pore pressure (p_s), soil displacements (u_s , w_s), vertical effective stress (σ'_z) and shear stress τ_{xz} , are widely studied in previous studies on WSSI [52–54] due to their significant effects on seabed stability. Fig. 7 displays the spatial distribution of p_s , u_s , w_s , σ'_z and τ_{xz} on x - z plane ($y = 0$) when wave pressure at location O reaches its minimum ($t/T = 6.250$). It is shown that, the magnitudes of p_s and σ'_z is significantly larger than that of τ_{xz} . The magnitude of horizontal displacement (u_s) is quite smaller than that of the vertical displacement, and both are less than 2 mm. The maximum vertical displacement (w_s) takes place at the seabed surface, whereas the maximum horizontal displacement (u_s) occurs inside the seabed. It is known that a mild vertical pore pressure gradient ($\partial p_s / \partial z$) in shallow seabed depth provides an effective resistance to wave-induced liquefaction. In Fig. 7(a), it is clearly shown that pore pressure moderates rapidly with increase of seabed depth. This indicates that liquefaction

might occur with high-energy wave loading.

[Fig. 7 is here.]

Fig. 8 illustrates the vertical distribution of wave-induced pore pressure (p_s), effective normal stresses ($\sigma'_x, \sigma'_y, \sigma'_z$) and shear stresses ($\tau_{xy}, \tau_{xz}, \tau_{yz}$) at location O when $t/T = 6.250$. It is shown that the magnitude of pore pressure decreases dramatically near the seabed surface ($z/h \leq 0.05$), and almost keeps as zero in the deep soil depth ($z/h \geq 0.6$). The effective normal stresses presents a different trend, i.e. increasing first and then decreasing in the shallow soil range ($z/h < 0.1$). It is shown that the shear stress τ_{xz} have a similar feature, however with a rather smaller magnitude. Specially, the magnitudes of τ_{xy} , τ_{yz} almost keep as zero throughout the whole seabed depth. This indicates that the seabed undergoes shear deformation merely in x - z plane in line with the propagation direction of wave train. Besides, it is noted that the magnitude of σ'_z is the largest among the effective normal stresses. Hence, in the following parametric studies, the results of p_s , σ'_z , and τ_{xz} are presented.

[Fig. 8 is here.]

4.2. Effect of wave obliquity

Wave obliquity determines the magnitudes of blockage influence of the upstream pile on the downstream piles and the side-by-side interference, and both effects would vanish when pile spacing reaches a critical value. When incident angle of water wave, θ (see Fig. 1), alters from 30° , 60° to 90° , the pile group configuration shifts correspondingly as shown in Fig. 10. When $\theta = 30^\circ$, there are one pile upstream and two piles downstream. When $\theta = 60^\circ$, there are two piles on a line in the direction of wave propagation and the third pile at the lateral side. When $\theta = 90^\circ$, there are two piles upstream and one pile downstream.

Fig. 9 displays the time series of wave pressure at O with various wave obliquities. It is shown that, with θ increasing from 30° to 90° , the magnitude of positive wave pressure increases while the magnitude of negative wave pressure (suction) decreases. This indicates that the wave obliquity plays an important role. When θ shifts from 30° to 90° , the blockage effect of the upstream pile(s) on location O decreases, meanwhile the wave reflection from the downstream pile(s) increases. Hence, it could be concluded that

stronger blockage effect leads to higher magnitude of negative wave pressure, and meanwhile the increasing wave reflection from the downstream pile(s) could enlarge the magnitude of positive wave pressure.

[Fig. 9 is here.]

The wave pressure (p_{wv}) acting on the seabed surface is shown in Fig. 10 when wave pressure at O reaches its minimum. It can be observed that the alteration of wave obliquity changes the spatial distribution of wave pressure in the proximity of three piles. The amplitude of wave pressure moderates with the wave propagation. At the lee side of the pile group, the significant change of wave pressure distribution could be observed due to the wave-structure interaction. In the vicinity of three piles, it could be seen that the minimum wave pressure is located between the three piles when $\theta = 30^\circ$, 60° , and is located in front of the pile group when $\theta = 90^\circ$. This indicates that, under the action of blockage effect, the region between three piles endures the most severe threat of seabed liquefaction.

[Fig. 10 is here.]

Wave obliquity should influence seabed response as it affects the wave loading. Fig. 11 displays the vertical distribution of wave-induced pore pressure (p_s), vertical normal effective stress (σ'_z) and shear stress τ_{xz} under wave trough (location O) for various wave incident angle (θ). It is seen that with the increase of θ , p_s , σ'_z and τ_{xz} decreases in whole seabed depth. It is noted that the decrement of p_s is quite smaller than those of σ'_z and τ_{xz} . It is well known that the seabed liquefaction may take place when the excessive pore pressure overcomes the mean initial effective stresses. Meanwhile, shear failure would occur when the wave-induced shear stress exceeds the shear resistance of the seabed [7]. Thus, the influence of wave obliquity might have more significant impact on seabed anti-shear stability in comparison with anti-liquefaction stability.

[Fig. 11 is here.]

4.3. Effect of pile diameter

In this subsection, the effect of pile diameter (D) will be examined in case of wave obliquity angle $\theta = 30^\circ$. The pile spacing (d) (see Fig. 1) equals to and changes with pile

1 diameter (D), i.e. $d = D$.

2 Variation of pile diameter (D) alters the flow regime around three piles. When D
3 increases, the blockage effect of three piles on wave propagation grows and leads to
4 higher magnitude of wave pressure (p_{wv}) on seabed, as shown in Fig. 12, in which $k =$
5 $2\pi/L_w$ is the wave number. It can be also observed that increase of pile diameter (kD)
6 gives rise to phase lag of wave pressure cycling. This indicates that the increase of pile
7 diameter delays the wave propagation due to the elongation of diffraction path around
8 the upstream pile. Besides, the magnitude of negative wave pressure is enlarged when
9 kD increases from 0.6 to 0.9, and however decreases when kD increases from 0.9 to 1.2.
10 This indicates that, when kD increases, the blockage effect of the upstream pile grows,
11 however the side-by-side effect of the downstream piles is weakened as the pile spacing
12 increases (d/D).

13 *[Fig. 12 is here.]*

14 Fig. 13 presents the spatial distribution wave pressure on seabed when pore pressure
15 at location O reaches its maximum. It can be seen that, with increasing of pile diameter
16 (kD), the distribution of wave pressure is disturbed not only at the lee-side of the pile
17 group, but also in front of the pile group due to the growing effect of wave reflection
18 with the increase of pile diameter. In the whole domain, the increase of pile diameter
19 leads to the increase of minimum wave pressure (from -4.83kPa, -5.51kPa to -6.35 kPa).
20 When $kD = 0.6, 0.9$, the minimum wave pressure is located between three piles. However,
21 when $kD = 1.2$, its location is shifted to the forthcoming wave trough in front of the pile
22 group.

23 *[Fig. 13 is here.]*

24 Fig. 14 illustrates the minimum wave-induced pore pressure (p_s), effective normal
25 stress σ'_z and shear stress τ_{xz} in seabed for various pile diameter (kD). It is shown that
26 the seabed response changes due to the variation of wave loading with the increase of
27 pile diameter. It is also observed that the variation of pore pressure (p_s) is less than those
28 of the effective stress σ'_z and the shear stress τ_{xz} .

29 *[Fig. 14 is here.]*

30 4.4. Effect of pile spacing

Pile spacing influences the interaction of diffracted wave from the upstream pile(s) and reflected wave from the downstream pile(s), leading to wave pressure variation on seabed. As a matter of fact, the pile group effect would be diminished when pile spacing increases to a critical value. However, when pile spacing is relatively small, they could be treated as a mono-pile, as have been pointed out by Sumer and Fredsøe [55]. In this subsection, the pile spacing used are $d = 5$ m, 6 m, and 7 m, which are 2.5, 3.0, and 3.5 times of pile diameter ($D = 2$ m), respectively.

Fig. 15 depicts the time series of wave pressure (p_w) at point O for various pile spacing (d/D). It is shown that increasing the pile spacing leads to slight phase lag and negligible decrease of crest value of wave pressure, while the trough value almost remains a constant. Hence, it could be deduced that when pile spacing $d/D \geq 2.5$, the influence of pile spacing on seabed response could be neglected. This indicates that the effect of pile spacing is insignificant as the pile spacing is usually greater than $2.5D$ in offshore engineering to overcome the overturning moment of the lateral loadings (wind, wave, current, etc.).

[Fig. 15 is here.]

Fig. 16 shows the wave pressure distribution on seabed surface when wave pressure at location O reaches its minimum for various pile spacing. It can be observed that increase of pile spacing hardly alters the spatial distribution of wave pressure, however affecting the magnitude of wave pressure around pile group. When pile spacing $d = 2.5D = 5$ m (Fig. 16a), the minimum wave pressure is -4.85 kPa, and is located before the pile group rather than O. When pile spacing extends to $3D$ (Fig. 16b), the magnitude of minimum wave pressure declines to -4.83 kPa. When further extending d to $3.5D$ (Fig. 16c), the magnitude of minimum wave pressure decreases to -4.67 kPa.

[Fig. 16 is here.]

Fig. 17 displays the vertical distribution of wave-induced pore pressure (p_s), effective normal stress σ'_z and shear stress τ_{xz} at O under wave trough. It is shown that, with the variation of pile spacing, p_s , σ'_z , and τ_{xz} almost remained the same except for the drop down of τ_{xz} when d/D increase from 3.0 to 3.5. Generally, the result of seabed response corresponds to the variation of wave loading.

[Fig. 17 is here.]

4.5. Effect of embedment depth

The variation of embedment depth (d_e) of pile foundations may not affect the wave-structure interaction when embedment depth fulfils the requirement of bearing capacity. However, it will alter the soil-structure boundary and lead to variation of seabed response. The soils between three piles are restricted and the horizontal transmission of pore pressure would be blocked, which might lead to higher pore pressure. However, the pore pressure below the embed depth could be freely transmitted in the horizontal plane. Hence, the wave-induced seabed response might be changed.

Fig. 18 illustrates the wave-induced pore pressure p_s , effective normal stress σ'_z and shear stress τ_{xz} at location O when magnitude of negative wave-pressure at O reaches its maximum for various embedment depth (d_e/h , h is the seabed thickness). It is seen that, with the variation of embedment depth, the seabed response almost keeps unchanged above the pile tip. Under the pile tip, there is a slight difference between the cases of partially penetrated pile group and the fully penetrated pile group. As matter of fact, the embedment depth of a pile foundation should be at least several times the pile diameter to provide bearing capacity for upper structure, and hence the influence of embedment depth on seabed liquefaction around three piles should be negligible.

[Fig. 18 is here.]

5. Conclusion

This paper parametrically studied the pile group effect on wave-induced seabed response with a 3D numerical model. Three-pile group is adopted to explore the effects of wave obliquity, pile diameter, pile spacing, and embedment depth, respectively. The main conclusions can be drawn as follows:

1. Magnitudes of wave pressure and seabed response moderate with wave propagation due to wave interaction with three-pile group. The simultaneous effect of side-by-side interaction and blockage effect tends to generate a larger magnitude of negative wave pressure on seabed between the three piles, which would further create a severe risk of seabed liquefaction. The wave-induced shear response is in the vertical plane of wave propagation (x - z plane in this study).

- 1 2. Wave obliquity affects the wave pressure and seabed response between three piles.
2 The alteration of wave obliquity angle from 30° to 90° attenuates the blockage effect
3 of upstream pile(s) on the region between piles, leading to increases of negative
4 wave pressure and seabed response between three piles.
- 5 3. Increase of pile diameter tends to aggravate the blockage effect on wave propagation.
6 Increasing pile diameter causes larger magnitude of negative wave pressure in front
7 of the pile group, and elongates the diffraction path around three piles, which leads
8 to stronger depletion of wave energy and generates wave phase lag. With the
9 increase of pile diameter, the magnitude of negative wave pressure and
10 corresponding seabed response amid the piles increase first then decline.
- 11 4. Effects of pile spacing and embedment depth are insignificant in practice. With the
12 increase of pile spacing, the minimum wave pressure and corresponding seabed
13 response around pile group decrease slightly. With the decrease of pile embedment
14 depth, merely a negligible variation of seabed response under the pile tip occurs.

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18 **References**

- 19 [1] G.T. Houlsby, Interactions in offshore foundation design, *Géotechnique*. 66 (2016)
20 791–825. doi:10.1680/jgeot.15.RL.001.
- 21 [2] V. Negro, J.-S. López-Gutiérrez, M.D. Esteban, P. Alberdi, M. Imaz, J.-M. Serracarla,
22 Monopiles in offshore wind: Preliminary estimate of main dimensions, *Ocean Eng.*
23 133 (2017) 253–261. doi:10.1016/j.oceaneng.2017.02.011.
- 24 [3] J. Zang, P.H. Taylor, Steep wave and breaking wave impacts on offshore structures
25 wind turbine columns, *Proc. 3rd Int. Conf. Appl. Phys. Model. to Port Coast. Prot.*
26 (2010) 1–4.
- 27 [4] S. LIU, Y. LI, G. LI, Wave Current Forces on the Pile Group Of Base Foundation for
28 the East Sea Bridge, China, *J. Hydrodyn. Ser. B*. 19 (2007) 661–670.
29 doi:10.1016/S1001-6058(08)60001-3.

- 1 [5] J. de Kuiter, F.L.L. Beringen, Pile foundations for large North Sea structures, *Mar.*
2 *Geotechnol.* 3 (1979) 267–314. doi:10.1080/10641197909379805.
- 3 [6] G.H. Sterling, G.E. Strohbeck, The Failure of the South Pass 70 Platform B in
4 Hurricane Camille, *J. Pet. Technol.* 27 (1975) 263–268. doi:10.2118/4475-PA.
- 5 [7] D.-S. Jeng, *Porous Models for Wave-seabed Interactions*, Springer Berlin Heidelberg,
6 Berlin, Heidelberg, 2013. doi:10.1007/978-3-642-33593-8.
- 7 [8] K. ZEN, H. YAMAZAKI, Mechanism of wave-induced liquefaction and
8 densification in seabed., *SOILS Found.* 30 (1990) 90–104.
9 doi:10.3208/sandf1972.30.4_90.
- 10 [9] K. ZEN, H. YAMAZAKI, Oscillatory pore pressure and liquefaction in seabed
11 induced by ocean waves., *Soils Found.* 30 (1990) 147–161.
12 doi:10.3208/sandf1972.30.4_147.
- 13 [10] H. Nago, M. Shiro, T. Matsumoto, Y. Hachiman, Liquefaction and densification of
14 loosely deposited sand bed under water pressure variation, in: *Proceeding 3rd Int.*
15 *Offshore Polar Eng. Conf. Vol. 1.*, The International Society of Offshore and Polar
16 Engineers, Singapore, 1993: pp. 578–584. [https://www.onepetro.org/conference-](https://www.onepetro.org/conference-paper/ISOPE-I-93-083)
17 [paper/ISOPE-I-93-083](https://www.onepetro.org/conference-paper/ISOPE-I-93-083).
- 18 [11] C.C. Liao, H. Zhao, D.-S. Jeng, Poro-Elasto-Plastic Model for the Wave-Induced
19 Liquefaction 1, *J. Offshore Mech. Arct. Eng.* 137 (2015) 042001.
20 doi:10.1115/1.4030201.
- 21 [12] H.B. Seed, M.S. Rahman, Wave-induced pore pressure in relation to ocean floor
22 stability of cohesionless soils, *Mar. Geotechnol.* 3 (1978) 123–150.
23 doi:10.1080/10641197809379798.
- 24 [13] S. Sassa, H. Sekiguchi, Wave-induced liquefaction of beds of sand in a centrifuge,
25 *Géotechnique*. 49 (1999) 621–638. doi:10.1680/geot.1999.49.5.621.
- 26 [14] M.A. Biot, General Theory of Three-Dimensional Consolidation, *J. Appl. Phys.* 12
27 (1941) 155–164. doi:10.1063/1.1712886.
- 28 [15] M.A. Biot, Theory of Elasticity and Consolidation for a Porous Anisotropic Solid, *J.*
29 *Appl. Phys.* 26 (1955) 182–185. doi:10.1063/1.1721956.

- 1 [16]M.A. Biot, Mechanics of Deformation and Acoustic Propagation in Porous Media,
2 J. Appl. Phys. 33 (1962) 1482–1498. doi:10.1063/1.1728759.
- 3 [17]M.B.C. Ulker, M.S. Rahman, D.S. Jeng, Wave-induced response of seabed: Various
4 formulations and their applicability, Appl. Ocean Res. 31 (2009) 12–24.
5 doi:10.1016/j.apor.2009.03.003.
- 6 [18]B.M. Sumer, Liquefaction Around Marine Structures, WORLD SCIENTIFIC, 2014.
7 doi:10.1142/7986.
- 8 [19]A.H.-D. Cheng, P.L.-F. Liu, Seepage force on a pipeline buried in a poroelastic
9 seabed under wave loadings, Appl. Ocean Res. 8 (1986) 22–32. doi:10.1016/S0141-
10 1187(86)80027-X.
- 11 [20]J. Lu, D. Jeng, A Coupled Model for an Offshore Pile , Seabed and Seawater
12 Interaction, J. Coast. Res. SI 50 (2007) 389–393.
- 13 [21]X.-J. Li, F.-P. Gao, B. Yang, J. Zang, Wave-induced pore pressure responses and
14 soil liquefaction around pile foundation, Int. J. Offshore Polar Eng. 21 (2011) 233–
15 239.
- 16 [22]T. Sui, C. Zhang, Y. Guo, J. Zheng, D. Jeng, J. Zhang, W. Zhang, Three-dimensional
17 numerical model for wave-induced seabed response around mono-pile, Ships
18 Offshore Struct. 11 (2016) 667–678. doi:10.1080/17445302.2015.1051312.
- 19 [23]C. Zhang, Q. Zhang, Z. Wu, J. Zhang, T. Sui, Y. Wen, Numerical Study on Effects
20 of the Embedded Monopile Foundation on Local Wave-Induced Porous Seabed
21 Response, Math. Probl. Eng. 2015 (2015) 1–13. doi:10.1155/2015/184621.
- 22 [24]C. Zhang, T. Sui, J. Zheng, M. Xie, V.T. Nguyen, Modelling wave-induced 3D non-
23 homogeneous seabed response, Appl. Ocean Res. 61 (2016) 101–114.
24 doi:10.1016/j.apor.2016.10.008.
- 25 [25]Z. Lin, D. Pokrajac, Y. Guo, D. sheng Jeng, T. Tang, N. Rey, J. Zheng, J. Zhang,
26 Investigation of nonlinear wave-induced seabed response around mono-pile
27 foundation, Coast. Eng. 121 (2017) 197–211. doi:10.1016/j.coastaleng.2017.01.002.
- 28 [26]H.Y. Zhao, D. Jeng, C.C. Liao, J.F. Zhu, Three-dimensional modeling of wave-
29 induced residual seabed response around a mono-pile foundation, Coast. Eng. 128
30 (2017) 1–21. doi:10.1016/j.coastaleng.2017.07.002.

- 1 [27] S. Sassa, H. Sekiguchi, J. Miyamoto, Analysis of progressive liquefaction as a
2 moving-boundary problem, *Géotechnique*. 51 (2001) 847–857.
3 doi:10.1680/geot.2001.51.10.847.
- 4 [28] T. Sui, J. Zheng, C. Zhang, J. Zhang, G. Wang, Wave-Induced Porous Seabed
5 Response around Pile-Group Foundations, in: *Proc. Twenty-Third Int. Offshore*
6 *Polar Eng.*, ISOPE, Anchorage, Alaska, USA, 2013: pp. 1188–1192.
- 7 [29] K.T. Chang, D.S. Jeng, Numerical study for wave-induced seabed response around
8 offshore wind turbine foundation in Donghai offshore wind farm, Shanghai, China,
9 *Ocean Eng.* 85 (2014) 32–43. doi:10.1016/j.oceaneng.2014.04.020.
- 10 [30] Q. Zhang, X. Zhou, J. Wang, J. Guo, Wave-induced seabed response around an
11 offshore pile foundation platform, *Ocean Eng.* 130 (2017) 567–582.
12 doi:10.1016/j.oceaneng.2016.12.016.
- 13 [31] K. Chatziioannou, V. Katsardi, A. Koukouselis, E. Mistakidis, The effect of
14 nonlinear wave-structure and soil-structure interactions in the design of an offshore
15 structure, *Mar. Struct.* 52 (2017) 126–152. doi:10.1016/j.marstruc.2016.11.003.
- 16 [32] J.-Y. Chen, R.B. Gilbert, F.J. Puskar, S. Verret, Case Study of Offshore Pile System
17 Failure in Hurricane Ike, *J. Geotech. Geoenvironmental Eng.* 139 (2013) 1699–1708.
18 doi:10.1061/(ASCE)GT.1943-5606.0000894.
- 19 [33] B. Stuyts, D. Cathie, Y. Xie, Scour Assessment and Measurements for Pile-
20 Supported Wind Turbine Foundations, in: *Vol. 6 Polar Arct. Sci. Technol. Offshore*
21 *Geotech. Pet. Technol. Symp.*, ASME, 2013: p. V006T10A025.
22 doi:10.1115/OMAE2013-10660.
- 23 [34] C. Yuan, B.W. Melville, K.N. Adams, Scour at wind turbine tripod foundation under
24 steady flow, *Ocean Eng.* 141 (2017) 277–282. doi:10.1016/j.oceaneng.2017.06.038.
- 25 [35] Q. Zhang, X.-L. Zhou, J.-H. Wang, Numerical investigation of local scour around
26 three adjacent piles with different arrangements under current, *Ocean Eng.* 142 (2017)
27 625–638. doi:10.1016/j.oceaneng.2017.07.045.
- 28 [36] A.G. Hanssen, A. Tørum, Breaking Wave Forces on Tripod Concrete Structure on
29 Shoal, *J. Waterw. Port, Coastal, Ocean Eng.* 125 (1999) 304–310.
30 doi:10.1061/(ASCE)0733-950X(1999)125:6(304).

- 1 [37] A. Hildebrandt, V. Sriram, T. Schlurmann, Simulation of Focusing Waves and Local
2 Line Forces due to Wave Impacts on a Tripod Structure, in: Proc. Twenty-Third Int.
3 Offshore Polar Eng., ISOPE, Anchorage, Alaska, USA, 2013: pp. 575–581.
4 <https://www.onepetro.org/conference-paper/ISOPE-I-13-359>.
- 5 [38] R. Mo, M. Li, H. Kang, Wave Interaction with a Tripile Foundation Using SBFEM,
6 in: Proc. Twelfth Pacific-Asia Offshore Mech. Symp., ISOPE, Gold Coast, Australia,
7 2016: pp. 525–532.
- 8 [39] H.Y. Zhao, D.S. Jeng, Numerical study of wave-induced soil response in a sloping
9 seabed in the vicinity of a breakwater, Appl. Ocean Res. 51 (2015) 204–221.
10 doi:10.1016/j.apor.2015.04.008.
- 11 [40] H.Y. Zhao, D.S. Jeng, C.C. Liao, Parametric study of the wave-induced residual
12 liquefaction around an embedded pipeline, Appl. Ocean Res. 55 (2016) 163–180.
13 doi:10.1016/j.apor.2015.12.005.
- 14 [41] Z. Lin, Y. Guo, D. Jeng, C. Liao, N. Rey, An integrated numerical model for wave–
15 soil–pipeline interactions, Coast. Eng. 108 (2016) 25–35.
16 doi:10.1016/j.coastaleng.2015.11.003.
- 17 [42] J.H. Ye, D.-S. Jeng, Response of Porous Seabed to Nature Loadings: Waves and
18 Currents, J. Eng. Mech. 138 (2012) 601–613. doi:10.1061/(ASCE)EM.1943-
19 7889.0000356.
- 20 [43] B.E. Launder, D.B. Spalding, The numerical computation of turbulent flows,
21 Comput. Methods Appl. Mech. Eng. 3 (1974) 269–289. doi:10.1016/0045-
22 7825(74)90029-2.
- 23 [44] W. RODI, Turbulence models and their applications in hydraulics, AIRH, Delft,
24 1984.
- 25 [45] C.W. Hirt, B.D. Nichols, Volume of fluid (VOF) method for the dynamics of free
26 boundaries, J. Comput. Phys. 39 (1981) 201–225. doi:10.1016/0021-9991(81)90145-
27 5.
- 28 [46] W.J. Rider, D.B. Kothe, Reconstructing Volume Tracking, J. Comput. Phys. 141
29 (1998) 112–152. doi:10.1006/jcph.1998.5906.

- 1 [47] T. Sui, J. Zhang, J. Zheng, C. Zhang, Modeling of Wave-Induced Seabed Response
2 and Liquefaction Potential Around Pile Foundation, in: Vol. 6 Polar Arct. Sci.
3 Technol. Offshore Geotech. Pet. Technol. Symp., ASME, 2013: p. V006T10A013.
4 doi:10.1115/OMAE2013-10230.
- 5 [48] L.F. Chen, J. Zang, A.J. Hillis, G.C.J. Morgan, A.R. Plummer, Numerical
6 investigation of wave–structure interaction using OpenFOAM, Ocean Eng. 88 (2014)
7 91–109. doi:10.1016/j.oceaneng.2014.06.003.
- 8 [49] Y. Tsui, S.C. Helfrich, Wave-Induced Pore Pressures in Submerged Sand Layer, J.
9 Geotech. Eng. 109 (1983) 603–618. doi:10.1061/(ASCE)0733-
10 9410(1983)109:4(603).
- 11 [50] B. Liu, D.-S. Jeng, G.L. Ye, B. Yang, Laboratory study for pore pressures in sandy
12 deposit under wave loading, Ocean Eng. 106 (2015) 207–219.
13 doi:10.1016/j.oceaneng.2015.06.029.
- 14 [51] L.L. Zhang, Y. Cheng, J.H. Li, X.L. Zhou, D.S. Jeng, X.Y. Peng, Wave-induced
15 oscillatory response in a randomly heterogeneous porous seabed, Ocean Eng. 111
16 (2016) 116–127. doi:10.1016/j.oceaneng.2015.10.016.
- 17 [52] M.B.C. Ulker, M.S. Rahman, M.N. Guddati, Wave-induced dynamic response and
18 instability of seabed around caisson breakwater, Ocean Eng. 37 (2010) 1522–1545.
19 doi:10.1016/j.oceaneng.2010.09.004.
- 20 [53] D.-S. Jeng, J.-H. Ye, J.-S. Zhang, P.L.-F. Liu, An integrated model for the wave-
21 induced seabed response around marine structures: Model verifications and
22 applications, Coast. Eng. 72 (2013) 1–19. doi:10.1016/j.coastaleng.2012.08.006.
- 23 [54] J. Ye, D. Jeng, R. Wang, C. Zhu, Numerical simulation of the wave-induced dynamic
24 response of poro-elastoplastic seabed foundations and a composite breakwater, Appl.
25 Math. Model. 39 (2015) 322–347. doi:10.1016/j.apm.2014.05.031.
- 26 [55] B.M. Sumer, J. Fredsøe, Wave Scour around Group of Vertical Piles, J. Waterw.
27 Port, Coastal, Ocean Eng. 124 (1998) 248–256. doi:10.1061/(ASCE)0733-
28 950X(1998)124:5(248).

1 **List of tables**

2 *Table 1 Input data of the present study.*

Sub module	Parameter	Notation	Value	Unit
Wave	Period	T	4	s
	Wave height	H	2	m
	Wavelength	L_w	20.9	m
	Water depth	d_w	4	m
	Incident angle	θ	30°, 60°, 90°	
Seabed	Seabed thickness	h	20.9	m
	Seabed length	L	83.4	m
	Seabed width	B	41.7	m
	Permeability	k_s	1×10^{-4} (Coarse sand)	m/s
	Degree of saturation	S_r	0.98	
	Porosity	n_s	0.3	
	Poisson's ratio	ν	0.4	
	Shear modulus	G	10^7	Pa
Pile	Pile diameter	D	2, 3, 4	m
	Pile spacing	d	5, 6, 7	m
	Embedment depth	d_e	5.2 (0.25 h)	m
			10.4 (0.5 h)	
			15.6 (0.75 h)	
			20.9 (h)	

3

4

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- 18 Fig. 11. Wave-induced normalized minimum pore pressure and vertical effective stress
- 19 at O for various wave obliquity
- 20 Fig. 12. Comparison of wave pressure at O for various pile diameter.
- 21 Fig. 13. Wave pressure around three piles when wave pressure at O reaches its minimum
- 22 for various pile diameters.
- 23 Fig. 14. Wave-induced normalized minimum pore pressure and vertical effective stress
- 24 at O for various pile diameters.
- 25 Fig. 15. Comparison of wave pressure at O for various pile spacing (d).
- 26 Fig. 16. Wave pressure around three piles when wave pressure at O reaches its minimum
- 27 for various pile spacing.
- 28 Fig. 17. Wave-induced normalized minimum pore pressure and vertical effective stress
- 29 at O for various pile spacing.
- 30 Fig. 18. Wave-induced normalized minimum pore pressure and vertical effective stress

1 at O for various embedment depth.

Fig. 1. Definition of WSSI around three-pile group.

(a) FDM mesh of wave sub-model

(b) FEM mesh of seabed sub-model

Fig. 2. Illustration of mesh size of the integrated model.

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Fig. 3. Time series of free surface elevation against experiment data of Zang et al. [3].
 (a) Wave gauge 1, 0.77 m from the inlet; (b) Wave gauge 2, 0.002 m in front of the
 mono-pile surface along the centerline.

Fig. 4. Vertical distribution of pore pressure against experiment data of Tsui and Helfrich [49]. Symbols denote the experiment data, and lines denotes the numerical results.

Fig. 5. Time series of pore pressure at various depth against experiment data of Liu et al. [50].

Fig. 6. Time series of wave pressure at typical locations around three-pile group.

Fig. 7. Pore pressure, soil displacements and soil stresses on x-z plane ($y=0$) when $t/T = 6.250$.

Fig. 8. Seabed response at location O when pore pressure reaches its minimum.

Fig. 9. Comparison of wave pressure at O for various wave obliquity.

Fig. 10. Wave pressure around three piles when wave pressure at O reaches its minimum for various wave obliquities.

1
2 Fig. 11. Wave-induced normalized minimum pore pressure and vertical effective stress
3 at O for various wave obliquity.
4

Fig. 12. Comparison of wave pressure at O for various pile diameter.

Fig. 13. Wave pressure around three piles when wave pressure at O reaches its minimum for various pile diameters.

1
2 Fig. 14. Wave-induced normalized minimum pore pressure and vertical effective stress
3 at O for various pile diameters.
4

Fig. 15. Comparison of wave pressure at O for various pile spacing.

Fig. 16. Wave pressure around three piles when wave pressure at O reaches its minimum for various pile spacing.

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2 *Fig. 17. Wave-induced normalized minimum pore pressure and vertical effective stress*
 3 *at O for various pile spacing.*

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Fig. 18. Wave-induced normalized minimum pore pressure and vertical effective stress at O for various embedment depth.

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