

# Wave-induced seabed residual response and liquefaction around a mono-pile foundation with various embedded depth

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1	Wave-induced seabed residual response and liquefaction around a mono-pile
2	foundation with various embedded depth
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Abstract: Wave-induced seabed instability caused by the residual liquefaction of 19 seabed may threaten the safety of an offshore foundation. Most previous studies have 20 21 focused on the structure that sits on the seabed surface (e.g., breakwater and pipeline), a few studies investigate the structure embedded into the seabed (e.g. a mono-pile). In 22 23 this study, by considering the inertial terms of pore fluid and soil skeleton, a threedimensional (3D) integrated model for the wave-induced seabed residual response 24 around a mono-pile is developed. The model is validated with five experimental tests 25 available in the literature. The proposed model is then applied to investigate the spatial 26 and temporal pattern of pore pressure accumulation as well as the 3D liquefaction zone 27 around a mono-pile. The numerical simulation shows that the residual pore pressure in 28 front of a pile is larger than that at the rear, and the seabed residual response would be 29 30 underestimated if the inertial terms of pore fluid and soil skeleton are neglected. The result also shows that the maximum residual liquefaction depth will increase with the 31 increase of the embedded depth of the pile. 32

Keywords: Wave loading; seabed residual response; inertial terms; pile foundation;
embedded depth; liquefaction.

## 35 **1. Introduction**

Offshore marine structures are normally subjected to complex dynamic environmental 36 loadings during their service lifetime (Sumer, 2014). Their operational safety affected 37 by wave, current, tidal and seabed instability attracts the continuous attention of 38 offshore engineers and researchers (e.g., Mattioli et al., 2012; Fuhrman et al., 2014; 39 Zheng et al., 2014; Liang et al., 2015; Zhang et al. 2017; Zhang W. et al. 2018). A mono-40 pile has been widely used as the foundation of offshore wind power system, which 41 occupies approximately 80% of the commonly-used structural types (e.g., gravity, 42 43 tripods, and jacket) in the European market (EWEA, 2016). Regarding the failure of marine structure in an extreme hydrodynamic environment, it is believed that this may 44 be due to the wave-induced seabed instability around foundations, rather than the 45 46 construction deficiencies caused by wave impaction (Smith and Gordon, 1983). When a wave propagates over a seafloor, an excessive pore pressure within the seabed would 47 be generated, particularly under a poor drainage condition of soil. If the excess pore 48 49 pressure becomes greater than the overburden pressure, the resistance of soil skeleton will be fully lost owing to soil liquefaction, which threatens the stability of structures. 50 This implies an accurate evaluation of the wave-induced seabed response is important 51 in design of an offshore structural foundation. 52

Extensive studies have been conducted to investigate the wave-induced seabed response owing to its practical importance since the 1970s (Yamamoto et al., 1978). Based on the experiments and field observations (Zen and Yamazaki, 1990), two mechanisms for the wave-induced pore pressure variation can be identified, namely, an

oscillatory mechanism and a residual mechanism. An oscillatory mechanism is usually 57 found in the unsaturated sandy seabed with good drainage conditions, where a sharp 58 upwardly directed pressure gradient may lead to the momentary seabed liquefaction 59 (Madsen, 1978; Alcérreca-Huerta and Oumeraci, 2016; Zhou et al., 2018). In contrast, 60 a residual mechanism can be observed in a silt seabed with low permeability. For the 61 residual mechanism, the corresponding compaction of soil skeleton leads to an 62 accumulated pore pressure, which promotes the seabed residual liquefaction. As 63 reported in Jeng and Seymour (2007), residual mechanism will dominate the process of 64 65 seabed liquefaction for poorly-drainage conditions. Note that, this study would focus on the wave-induced residual liquefaction of a seabed around a mono-pile. 66

Among the previous works for wave-induced residual liquefaction of seabed, Seed 67 68 and Rahman (1978) may have been the first to investigate the pore pressure accumulation under wave loading. In their study, the cyclic shear stress was taken into 69 consideration using a non-linear source term for pore pressure accumulation. Using the 70 71 approximate linear-relation of the source term, McDougal et al. (1989) proposed analytical solutions for the pore pressure accumulation for a shallow, medium, and deep 72 seabed. The above solution was re-examined by Cheng et al. (2001) and some mistakes 73 of mathematical derivationed were reported. In their study, an analytical solution using 74 the Fourier transform was developed for the pore pressure accumulation. As pointed 75 out by Jeng et al. (2007), some mistakes were made in both McDougal (1989) and 76 Cheng et al. (2001). After correction of the previous work, the theoretical approach of 77 Jeng et al. (2007) is shown to provide a better prediction. In their approach, Laplace 78

Transportation was used for deep foundation, while Fourier Transformation was sued 79 for shallow foundation and a seabed of finite thickness. Based on the analytical solution, 80 81 a simplified model for prediction for residual liquefaction (so-called J-S curve) was proposed in Jeng and Seymour (2007). Furthermore, as reported in Jeng et al. (2007), 82 the predicated pore pressures based on both non-linear and linear-relations of the source 83 term are almost identical. Therefore, no any further research is needed for considering 84 the non-linear relations of the source term. Sumer et al. (2012) carried out a series of 85 experimental tests to determine several empirical factors for the residual response of 86 87 seabed. Using centrifuge flume tests, Sekiguchi et al. (1995) and Sassa and Sekiguchi (1999) examined the relationship between the elasto-plastic soil behaviour and pore 88 pressure build-up. Their studies considered the conditions under loadings of both 89 90 propagating and standing waves. Based on the residual model of Smits et al (1978), Meijers and Luger (2012), and Meigjers et al. (2014) proposed one numerical model 91 "DCYCLE" to investigate the effects of the pre-shearing and random waves on the 92 93 residual seabed response. Recently, Jeng and Zhao (2015) proposed a new definition of the source term and established a two-dimensional (2D) numerical model to consider 94 the time-phased shear stress of seabed. The pore pressure distribution with both 95 propagating and standing wave loading was investigated in their study. All the above 96 97 investigations didn't consider the presence of the marine structure.

98 The static loading of a marine structure due to its self-weight increases the initial 99 effective stress of soil, and then significantly affecting the residual response of seabed, 100 particularly for a heavy marine infrastructures (Jeng et al., 2013; Ye et al., 2015). Based

on an experimental study, Sumer et al. (1999; 2006) investigated the build-up of the 101 pore pressure and corresponding liquefaction around a pipeline. Their studies indicated 102 that liquefaction firstly occurred at the bottom of the pipe, and then developed upwards 103 to the top-soil along the pile surface. Recently, Ye et al. (2015) developed an integrated 104 numerical model for the residual response of seabed, which combines the Reynolds 105 Average Navier-Stokes (RANS) wave model and an elastoplastic seabed model. The 106 integrated model was applied to investigate the residual response of the sloping seabed 107 around a composed breakwater. Zhao and Jeng (2016) investigated the pore pressure 108 109 accumulation around the trenched pipeline in a half-buried seabed. They indicated that liquefaction might occur in the underlying trench layer if the backfill of the soil is 110 shallow. The above investigations focused on the residual response of the seabed around 111 112 breakwaters and buried pipelines, which are particularly limited to a 2D case without wave diffractions around the marine structure. For more discussions on the seabed 113 response and liquefaction phenomenon around marine structures, readers can refer to 114 115 de Groot et al. (2006a; 2006b).

The phenomenon of wave-seabed-mono-pile interaction is a typical 3D flow problem, in which the embedded depth of the pile and the wave reflection and diffraction significantly affect the residual response of seabed. Qi and Gao (2014) experimentally investigated the seabed response and scour around a mono-pile foundation in the lab, in which the pore pressures were measured by the pressure sensor buried in the seabed. Recently, several literatures are published for seabed instantaneous response around mono-pile and group pile foundations which are with QS (Quasi-static model) or PD (Partial dynamic) models (Duan and Jeng, 2018; Duan et al., 2018; Lin
et al., 2017; Tong et al., 2018; Zhang et al., 2017). Sui et al. (2017) and Zhang et al.
(2016) developed a more advanced fully dynamic (FD) seabed model to consider the
inertial terms of soil skeleton and pore water. The range of application of the QS, PD,
and FD models for the seabed oscillatory response can be found in studies by Ulker and
Rahman (2009).

The aforementioned investigations focused on the oscillatory mechanism of soil 129 response for mostly the sandy seabed with high permeability. Li et al. (2011) developed 130 131 a finite element model for the seabed residual response around an embedded pile foundation. They showed that the seabed liquefaction is more serious at the rear of a 132 pile. However, in their study, the wave reflection and diffraction were neglected, 133 134 because they only focus on a pile with small diameter. More recently, Zhao et al. (2017a) numerically investigated the seabed residual response around a single pile by 135 integrating a RANS wave model and a quasi-static soil model (QS model). Therefore, 136 137 the inertial terms of soil skeleton and pore water were neglected in their study. However, the inertial terms of both soil skeleton or/and pore pressure (considered in the PD and 138 FD model) can significantly affect the seabed response, particularly for the cases around 139 marine structures (Jeng and Cha, 2003; Ulker et al., 2010). To the best of the authors' 140 knowledge, the effect of the above-mentioned inertial terms on the residual response of 141 seabed has not been investigated. Despite this, such important effect of wave 142 transformation and the embedded depth of a pile on the residual response, was not 143 systematically discussed in Zhao et al. (2017a). 144

In this study, a 3D numerical model WINBED (version 2.0) for the wave-induced 145 residual response of seabed around a mono-pile foundation is developed. It should be 146 noted that the previous version of the model (WINBED 1.0) of Sui et al. (2017) and 147 Zhang et al. (2016) only deals with the oscillatory seabed response. The main 148 contributions of the present WINBED 2.0 model are: (1) the residual response module 149 of seabed has been added by using a 3D pore pressure source term (see Eqs. 9-16); and 150 (2) the present model (WINBED 2.0) may be the first one that considers the acceleration 151 of pore fluid and soil skeleton (inertial terms) in simulating the residual response of 152 153 seabed.

The present paper is organized as follows: the governing equations, boundary 154 conditions, numerical scheme, and operational process of the present model are 155 156 presented in Section 2. Model validations based on five sets of flume tests available in the literature are illustrated in Section 3. Based on the model application, the wave-157 induced 3D distributions of the accumulated pore pressure and corresponding 158 159 liquefaction around a mono-pile are discussed in Section 4. Through these discussions, the residual pore pressures owing to different simulation modes of the QS, PD, and FD 160 formulations (effects of the inertial terms) are examined. The effects of wave reflection 161 and diffraction on the residual response of seabed are analysed. The significance of the 162 above effects with different vertical locations, wave steepness, soil permeability, and 163 relative soil densities are studied. Seabed liquefaction around a mono-pile foundation 164 is also investigated. Finally, several concluding remarks are given in Section 5. 165

166

#### 167 **2. Numerical Model**

#### 168 **2.1 Seabed model**

Following the previous study (Jeng, 2013), the wave-induced pore pressure (p) can be divided into two parts, namely the oscillatory (instantaneous) pore pressure  $(p_{ins})$  and residual (accumulation) pore pressure  $(p_r)$  (see Fig. 1), which is expressed as

$$p = p_{ins} + p_r \tag{1}$$

The oscillatory pore pressure usually behaves cyclically in magnitude induced by each wave loading, and could be found with phase lag as well as the damping of amplitude in the vertical direction (Yamamoto et al., 1978). On the other hand, the residual pore pressure shows a progressive nature with time lasting. This is due to the volumetric contraction caused by the cyclic shear stress of soil (Seed and Rahman, 1978). In the following parts of section 2, both mechanisms for the wave-induced seabed response will be described in detail.

179

#### 180 **2.1.1 Oscillatory Mechanism**

In this study, the Biot's poro-elastic theory is used to investigate the oscillatory response of seabed. The basic assumptions of the model are: (1) the soil skeleton and pore fluid are compressible; (2) the flow in the porous bed obeys Darcy's law; (3) the deformation of the porous seabed obeys the Hooke's law and (4) the effect of gas diffusing through water and movement of water vapour is ignored. It should be noted that the tensile stress may occur in the simulation as there is no "yield" criteria in the elastic model. However, the elastic model is popularly used due to its simplicity and numerous successful validation cases in engineering practice (Alcérreca-Huerta and Oumeraci, 2016; Jeng

189 et al., 2013; Meijers and Luger, 2012). The governing equations in FD approximations

190 can be written as follows (Zienkiewicz et al., 1980):

$$\sigma_{ij,j} + \rho g_i = \rho \ddot{u}_i + \rho_f \ddot{w}_i \tag{2}$$

$$-p_{ins,j} + \rho_f g_i = \rho_f \ddot{u}_i + \frac{\rho_f \ddot{w}_i}{n} + \frac{\rho_f g_i}{k_i} \dot{w}_i \tag{3}$$

$$\dot{u}_{i,i} + \dot{w}_{i,i} = -n\beta \dot{p}_{ins} \tag{4}$$

where  $\sigma_{ij}$  is the total stress,  $\rho$  is the average density of the porous medium,  $\rho_f$  is the density of pore water,  $g_i$  is the gravitational acceleration in the *i*-direction,  $u_i$  is the displacement of the soil matrix in the *i*-direction,  $w_i$  is the average relative displacement of the fluid to the solid skeleton in the *i*-direction,  $k_i$  is the permeability of the porous medium in the *i*-direction, *n* is the porosity of the solid phase.

196 The equivalent compressibility of pore water and entrapped air  $\beta$  is defined as 197 (Verruijt, 1969):

$$\beta = \frac{1}{k_w} + \frac{1 - S_r}{\rho_f g d} \tag{5}$$

where *d* is the water depth,  $S_r$  is the saturation degree,  $k_w$  is the bulk modulus of the pure water which is taken as  $1.95 \times 10^9$  N/m<sup>2</sup> (Yamamoto et al., 1978). This expression takes the saturation degree ( $S_r$ ) into account for the deformation of porous medium. It is noted that this definition is only valid for a high saturation degree (e.g.  $S_r \ge 0.95$ ) (Pietruszczak and Pande, 1996).

The total stress ( $\sigma_{ij}$ ) can be expressed in terms of the effective stress ( $\sigma'_{ij}$ ) and pore pressure (*p*), and the effective stress-strain relation can be written as:

$$\sigma_{ij} = \sigma'_{ij} - \delta_{ij} p_{ins} \tag{6}$$

$$\sigma_{ij}' = \lambda \varepsilon_{kk} \delta_{ij} + 2G \varepsilon_{ij} \tag{7}$$

$$\mathcal{E}_{ij} = \frac{u_{i,j} + u_{j,i}}{2} \tag{8}$$

where  $\delta_{ij}$  is the Kronecker delta denotation,  $\sigma'_{ij}$  is the effective stress,  $\varepsilon_{ij}$  is the soil strain,  $\lambda = 2G\mu(1-2\mu)$ , *G* is the shear modulus,  $\mu$  is Poisson's radio. Note that the above definition implies a positive tensional stress.

208

# 209 2.1.2 Residual Mechanism

Following the previous investigations of Seed and Rahman (1978), Sumer et al. (2012) for 1D case and Jeng and Zhao (2015) for 2D case, the numerical simulation of waveinduced residual response of the seabed around a marine structure is conducted in 3D space by this study. The governing equation for the pore pressure accumulation in the present model is:

$$\frac{\partial p_r}{\partial t} = c_{v3} \left( \frac{\partial^2 p_r}{\partial x^2} + \frac{\partial^2 p_r}{\partial y^2} + \frac{\partial^2 p_r}{\partial z^2} \right) + f_3(x, y, z, t)$$
(9)

where  $c_{\nu 3}$  is the coefficient of the soil consolidation and  $f_3(x, y, z, t)$  is the source term of the pore pressure generation in 3D space, which can be defined as:

$$C_{\nu3} = \frac{kE}{3(1-2\mu)\gamma_w} \tag{10}$$

$$f_3(x, y, z, t) = \frac{\sigma'_{03}}{T} \left[ \frac{|\tau_{ins3}(x, y, z, t)|}{\alpha_r \sigma'_{03}} \right]^{-\frac{1}{\beta_r}}$$
(11)

where *E* is the Young's modulus of soil,  $\gamma_w$  is the unit weight of pore fluid, *T* is the period of wave loading;  $\alpha_r$  and  $\beta_r$  are the empirical parameters which are defined from the following expressions (Sumer et al., 2012):

$$\alpha_r = 0.34D_r + 0.08 \tag{12}$$

$$\beta_r = 0.37D_r - 0.46 \tag{13}$$

$$D_r = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \tag{14}$$

220 where  $D_r$  is the relative density of soil.

In Eq. (11),  $\sigma'_{03}$  is the initial soil effective stress at the final state of seabed consolidation,  $\tau_{ins3}(x,y,z,t)$  is the phase-resolved shear stress obtained from the oscillatory model (see Section 2.1.1), which are expressed as:

$$\sigma_{03}' = \frac{1}{3} (\sigma_{x0}' + \sigma_{y0}' + \sigma_{z0}')$$
(15)

$$\tau_{3}(x, y, z, t) = \sqrt{\tau_{xz}^{2}(x, y, z, t) + \tau_{yz}^{2}(x, y, z, t) + \tau_{xy}^{2}(x, y, z, t)}$$
(16)

It should be noted that the shear stress of soil ( $\tau$ ) is defined as the maximum  $\tau_{(max)}$ 224 225 within one wave period by Seed and Rahman (1978) in their 1-D model, and is defined as the instantaneous  $\tau_{xy}$  by Jeng and Zhao (2015) in their 2-D model. Li and Jeng (2008) 226 further set it as  $\tau_3 = (\tau_{xy} + \tau_{yz} + \tau_{xz})/3$  based on an averaged concept, and applied it for the 227 228 3D seabed residual response around a breakwater head. However, the above definition of Li and Jeng (2008) may significantly underestimate the amplitude of pore pressure 229 (Fig. 6). In this study, the shear stress is defined (Eq. 16) based on the resultant force 230 concept in 3D space. The comparison between the previous (Li and Jeng, 2008) and 231 present (Eq. 16) definition in simulating the residual response of seabed is presented in 232 chapter 3. 233

234

# 235 **2.2 Wave model**

The "FUNWAVE 2.0" open-source code is adopted to calculate the wave pressures at 236 the soil-water interface and soil-structure interface, which are used as the input in the 237 seabed model. "FUNWAVE" code was first developed at University of Delaware 238 (Kirby et al., 2003) based on the nonlinear Boussinesq equations of Wei et al. (1995), 239 and is now commonly used in simulating wave motion in the coastal area. Consequently, 240 Shi et al. (2001) further discretized the equations on the staggered grid in the 241 generalized curvilinear coordinates in order to better fit the complex configuration 242 boundary. In FUNWAVE, different levels of Boussinesq approximations can be chosen 243 244 by setting an equation ID in the input file. The main advantage of FUNWAVE is to simulate the wave transformations around marine structures for a relatively large 245 coastal area (comparing to the CFD model (Zhang et al., 2014)). However, the 246 247 limitation is that it is hard to deal with the complex wave breaking in front of a structure. The FUNWAVE model has also been successfully adopted in the previous study of Sui 248 et al. (2016) for the oscillatory response of seabed. For more detailed information 249 250 regarding the governing equations as well as the numerical techniques, readers can refer to Kirby et al. (2003). 251

252

# 253 2.3 Boundary Conditions

To solve the governing equations, appropriate boundary conditions are required. Fig. 2 shows a 3D sketch of the boundary conditions used in the present model. The seabed is assumed to be impermeable and rigid at the lateral and bottom boundaries. Therefore, the seabed displacements and the normal gradients of both oscillatory and residual pore 258 pressures are zero:

$$u_{soil} = 0, \quad \frac{\partial p_{ins}(p_r)}{\partial n} = 0 \tag{17}$$

At the seabed surface, effective normal stress vanish. The shear stress is also neglected as it is minor comparing to the maximum dynamic pore pressure in this study (Ye and Jeng, 2011; Liang et al., 2008; Zhang et al., 2015). The wave-induced oscillatory pore pressure is equal to dynamic wave pressure, and the residual pore pressure is zero without any contraction of soil skeleton:

$$p_{ins} = p_w, \quad p_r = 0, \quad \sigma'_{soil} = 0, \quad \tau_{soil} = 0$$
 (18)

At the structure-seabed interface, the normal gradient of pore pressures is zero, while the seabed displacement is equal to that of structure. This "no-slip" boundary is usually assumed in the previous studies, which is reasonable due to the minor displacements of marine structures (Mizutani et al., 1998).

$$\frac{\partial p_{ins}(p_r)}{\partial n}, \quad u_{soil} = u_{pile}, \quad \sigma'_{pile} = \sigma'_{soil} - p, \quad \tau_{pile} = \tau_{soil}$$
(19)

At the water-structure interface, the structure normal stress is equal to the wave pressure, the shear stress is assumed to be zero. Note that, this indicates a one-way coupling between water and structure.

$$\tau_{soil} = 0, \ \sigma'_{pile} = p_w \tag{20}$$

At the air-structure interface, all stresses are set to zero by assuming that the effects of the wind/aerodynamic is minor to be neglected:

$$\tau_{soil} = 0, \ \sigma'_{pile} = 0 \tag{21}$$

273

#### 274 **2.4 Integrating procedure**

The present "WINBED" model consists of two seabed modes, which are the oscillatory 275 276 mode and residual mode. Note that, two-way coupling for structure-soil (Eq. 19) while one-way coupling (Eq. 20) for wave-structure are applied in this model. At the 277 beginning of the simulation, model initialization and grid generation are conducted, and 278 the boundary conditions are assigned according to Eqs. (17-21). At one time-step, the 279 oscillatory mode is first solved to obtain the seabed oscillatory variables  $(u_x, u_y, u_z, w_x, u_z, w_y, u_z, w_$ 280  $w_v, w_z, p_{ins}$ ). The soil effective stress is then obtained based on the strain-stress relation 281 282 of soil, and these are the input for the residual mode. The simulation results show that the relative error of residual pore pressure significantly decreases with the increase of 283 the iteration steps. In this study, the threshold relative error between two successive 284 285 iteration steps is set as 0.0001 (usually realized after about 100 iteration steps). Computation will be terminated when this simulation accuracy is achieved. 286

287

#### 288 **3. Model validation**

Five cases are conducted against the previous experimental data to validate the present model. Case 1 (Fig. 4) is for the water wave elevation around a mono-pile foundation (Cong et al 2015). Case 2 (Fig. 5) is for the oscillatory seabed response under pure wave loading (Lu 2005). Cases 3 (Fig. 6) and 4 (Fig. 7) are for the residual seabed response under pure wave loading (Sumer et al. 2012; Kirca et al. 2013). Case 5 (Fig. 8) is for the wave-induced seabed response around a mono-pile foundation, where the oscillatory response dominates in the experiment (Qi and Gao 2014).

296	Cong et al. (2015) conducted a series of physical experiments to measure the water
297	wave surface elevations around $2 \times 2$ group of circular piles. The experiments were
298	carried out in a wave basin to measure the wave reflection and diffraction caused by the
299	wave-pile interaction. The parameters used in the experiments for this validation
300	include the water depth of $d = 0.5$ m and the pile diameter of $D = 0.4$ m. Fig. 4 shows
301	a comparison of the water surface elevation at three orientations corresponding to $\alpha =$
302	0, 22.5°, and 45° with respect to the wave incident direction. Wave period $T$ and wave
303	amplitude <i>A</i> are shown in Fig. 4. The wave surface elevations in G6 of $\alpha = 22.5^{\circ}$ (Fig.
304	4b) show an irregular and relatively flat sinuous formation, whereas the wave elevations
305	in G5 of $\alpha = 0^{\circ}$ (Fig. 4a) and G3 of $\alpha = 45^{\circ}$ (Fig. 4c) show an irregular fluctuation,
306	which are due to the interaction of the wave with the pile. As shown in Fig. 4, the present
307	model reproduces the experiment results well, indicating that the present model can
308	simulate the non-linear wave transformation around piles with high accuracy.
309	For the second validation, the present model is compared with the flume test
310	conducted by Lu (2005) on the seabed oscillatory response under linear/cnoidal wave
311	loading. In the experiments, several pressure sensors were installed within soil at
312	different depths of 0, -5, -10, and -15 cm from the soil surface. The parameters for the
313	wave and seabed are: seabed depth, $h = 20$ cm; permeability, $k = 1.4 \times 10^{-3}$ m/s; Young's
314	modulus, $E = 1.4 \times 10^7$ m/s; poison's ratio, $\mu = 0.33$ ; and porosity, $n = 0.39$ . The linear
315	wave case use period, $T = 1.2$ s; depth, $d = 0.5$ m; and height, $H = 0.12$ m, whereas the
316	cnoidal wave case uses period, $T = 1.4$ s; depth, $d = 0.3$ m; and height, $H = 0.14$ m. Fig.
317	5 shows a comparison of the simulated and measured dynamic pore pressures under the

linear wave (figures in the left-side column) and cnoidal wave (figures in the right-side column). As Fig. 5 indicates, the fluctuation of pore pressure under a cnoidal wave is sharper and thinner than that under a linear wave, whereas the amplitude of the pore pressure with a cnoidal wave is much larger. Fig. 5 also reveals that the dynamic pore pressure decreases with an increase in distance from the soil surface, which is because the pore water velocity dramatically decreases when going deep into the seabed.

Fig. 6 shows a comparison of the simulated (Li and Jeng, 2008; Sumer et al., 2012; 324 and the present study) and measured (Sumer et al., 2012) pore pressure accumulations 325 326 (residual response) within a silt seabed under progressive wave loading. The parameters for the wave and seabed are: seabed depth, h = 40 cm; permeability,  $k = 1.0 \times 10^{-5}$  m/s; 327 Shear modulus,  $G = 1.92 \times 10^6$  m/s; Poison's ratio,  $\mu = 0.29$ ; porosity, n = 0.51; 328 submerged specific weight of the soil,  $\gamma' = 8.14 \text{ kN/m}^3$ ; degree of saturation,  $S_r = 1$ ; 329 wave period, T = 1.6 s; wave height, H = 0.18 m; and water depth, d = 0.55 m. It should 330 be noted that, the pressure oscillations recorded in the physical model are mainly caused 331 332 by the oscillatory seabed response (Sumer, 2014). To better demonstrate the residual pore pressure, the oscillatory part has been manually removed, following the previous 333 studies of Sumer et al. (2012), Kirca et al. (2013) and Jeng and Zhao (2015), depicted 334 in Fig. 7. As is shown in Figs. 6a and 6b, the averaged shear stress ( $\tau_{average}$  = 335  $1(|\tau_{xy}|+|\tau_{yz}|+|\tau_{yz}|)/3)$  by Li and Jeng (2008) may significantly underestimates the 336 measured accumulated pore pressure, whereas the definition in eq. (16) shows an 337 338 overall agreement. The numerical results adopting the maximum 3D shear stress ( $\tau_{3max}$ ) of this study (Eq. 16), are very close to those of Sumer et al. (2012). An overall trend 339

of the residual pore pressure in the present study agrees well with the experiment data. It is noted that the maximum pressure ratio  $p/\sigma'_{\nu0}$  is 0.98; indicating that the seabed is between the partial liquefaction ( $p_r/\sigma'_{\nu0} = 50\%$ ) and full liquefaction (100%) (close to the full liquefaction). This validation also demonstrates that the present model improves the numerical accuracy of the 3D residual response over that found by Li and Jeng (2008).

The fourth validation case is the experiment by Kirca et al. (2013) who studied the 346 residual liquefaction under a standing wave in a detailed systematic manner in a 347 348 physical model investigation. Fig. 7 shows the comparison of a simulated and measured (Kirca et al., 2013) pore pressure build-up within the silt seabed under a standing wave. 349 The soil conditions for the experiments were the same as in the third validation (Sumer 350 351 et al., 2012). The wave parameters for the current case are: wave height, H=10.2 cm; wave period, T=1.09 s; water depth, d=0.3 m. It should be noted that, a sealed plate was 352 used to separate the silt seabed; This is different from the configuration utilized in 353 simulation domain, and this slightly affects the simulation results. Overall, the 354 numerical results capture the trend of the accumulated pore pressure as shown in Fig. 355 7, and this validates the capability of the present model in dealing with the accumulated 356 pore pressure under a standing wave. 357

The final validation case involves a mono-pile foundation. The experiment of Qi and Gao (2014) is selected. Qi and Gao (2014) carried out a series of flume tests to investigate the wave/current-induced seabed instability (scouring and liquefaction) around a pile. In the present study, the measured data with only wave loading prior to

scour are used to validate the present model. The parameters of the wave, seabed, and 362 mono-pile are: seabed depth, h = 0.5 m; permeability,  $k = 1.0 \times 10^{-3}$  m/s; Young's 363 modulus,  $E = 3.84 \times 10^7$  m/s; Poison's ratio,  $\mu = 0.33$ ; porosity, n = 0.435; wave period, 364 T = 1.0 s; height, H = 0.08 m; depth, d = 0.5 m; pile radius, R = 0.2 m; and pile embedded 365 depth,  $l_p = 0.3$  m. Fig. 8 shows a comparison of the simulated and measured vertical 366 pore pressure distribution in front of and behind the pile. As illustrated in Fig. 8, the 367 pore pressure in front of the mono-pile is greater than behind, but decreases more 368 rapidly. This is because the wave loading increases in front of the pile owing to the 369 370 wave reflection and diffraction. In general, the simulated pore pressure around the mono-pile agrees well with the measurements. 371

372

#### 373 4. Model applications

In this section, the present model is applied to investigate the wave-induced seabed 374 residual response around a mono-pile. Model sketch for the parametric study is shown 375 376 in Fig. 2. There are two application cases in this study. The first application (Figs. 9-19) is to investigate the effect of wave diffraction/reflection on the residual response around 377 mono-pile. The second application (Figs. 20-25) is to investigate the effect of the pile 378 embedded depth on the residual response and liquefaction of its surrounding seabed. 379 For the first application, the pile parameters  $(R_p, E_p, \mu_p, l, d_e)$ , water depth (d), water 380 density  $(\rho_f)$ , seabed porosity (n), submerged weight  $(\gamma_s)$  are fixed. For the second 381 application, only the parameter of pile inserted depth is varied. To investigate a 382 relatively obvious wave diffraction/reflection phenomenon, the wave steepness for the 383

first application are relatively small, which corresponds to a weak residual response.

385 Parameters of the wave, the seabed and the mono-pile for the first application are listed

in Table 1; while parameters for the second application are listed in Table 2.

387

# 388 4.1 Consolidation state

In natural environments, the seabed will be subjected to a long-time consolidation under 389 the weight of static water and the seabed itself. The basic assumption of this study is 390 that the seabed can be further compressed by the wave loading after the long-time 391 consolidation process under the static water and pile gravity. The assumption is 392 reasonable especially with large wave loading, as soil particles can become more 393 adjacent between each other due to the back-and-forth movement caused by the cyclic 394 395 shear stress. This assumption is commonly adopted in many previous studies (e.g. Seed and Rahman, 1978; Sumer 2014). In this situation, the final consolidation state is 396 important as it implies the initial resistance to liquefaction prior to wave loading (Sui 397 398 et al., 2017; Ye et al., 2015; Li et al., 2017).

Fig. 9 illustrates the final consolidation state of the seabed for (a) pore pressure, (b) effective stress and (c) subsidence in the vicinity of a mono-pile. After the long-time consolidation, the pore pressure (p) was found layered in the vertical direction (Fig. 9a). Fig. 9b shows the spatial distribution of effective stress ( $\sigma'_z$ ) around the mono-pile. It was seen that the effective stress below the pile is remarkably increased, which is due to pile gravity being completely supported by the soil skeleton after a long-time consolidation. In addition, the phenomenon of stress concentration was found at the 406 corners of the pile. This is probably caused by the sharp change of Young's modulus
407 between the seabed and pile. As a result, the seabed below the pile suffers the largest
408 subsidence (Fig. 9c) because of the large soil effective stress there.

409

# 410 **4.2 Wave transformation**

Fig. 10 shows the water surface elevations with wave reflection and diffraction around 411 the mono-pile. It was found that the presence of the mono-pile increases and decreases 412 the wave height in front of and behind the mono-pile, respectively. Specifically, when 413 414 the wave trough arrives at the head of the pile (t = 2.95 T), the largest negative wave pressure will be generated at the bottom of the water. For t = 3.45 T, the wave crest 415 arrives at the same location, generating positive pressure. Owing to the wave reflection 416 417 and diffraction, it was found (Fig. 10) that a specific zone in the front and rear sides (dashed line in Fig. 10a) relative to the pile has the largest variation in wave height (H). 418 This phenomenon can significantly affect the spatial features of the accumulated pore 419 420 pressure (see Fig. 11 in section 4.3). It should be noted that the wave reflection and diffraction provide a 3D wave loading at the interfaces of water-seabed and water-421 structure. 422

423

#### 424 **4.3 Cyclic stresses and pile displacements**

425 Cyclic wave loading would cause cyclic stresses and cyclic pile displacements, and they 426 come from the oscillatory part of the model in this study. Fig 11 illustrates the 427 distributions of pore pressure  $p_{ins}$  (coming from the oscillatory part of pore pressures

(Eq. 1)), effective stress  $\sigma'_x$  and  $\sigma'_z$ , and shear stress  $\tau_{xz}$  around the mono-pile foundation 428 Fig. 11 show that the positive pore pressure and negative effective normal stresses ( $\sigma'_x$ 429 and  $\sigma'_z$ ) are found under the wave crest. Relatively large shear stresses  $\tau_{xz}$  is found 430 between the wave trough and wave crest. It is seen that at t=0.5T, relatively large 431 positive shear stress is found at the head of the pile foundation, which would cause a 432 relatively large power (large source term f) to generate the residual pore pressure. As a 433 result, the cyclic motion of pile is found, as shown in Fig. 12. Horizontal displacement 434 of pile behaves cyclically under the dynamic wave loading. It should also be noted that, 435 436 different from most of the previous studies that set a fixed mono-pile, the pile displacements at the bottom are not zero (see Fig. 12). This is because pile movement 437 is allowed in this study by applying the two-way coupling of soil-pile interaction. 438

439

#### 440 **4.4 Pore pressure accumulation**

In above application cases, only the residual pore pressure is considered as the seabed has poor drainage. Numerical simulation shows that the amplitude of the oscillation in pore pressure is much smaller than the residual pore pressure in the application cases. Therefore, the peak value caused by the oscillatory pore pressure may not significantly affect the onset of liquefaction in this study.

Fig. 13 illustrates the 3D temporal and spatial features of the accumulated pore pressure  $(p_r)$  around the mono-pile for (a) t = 35 s in the *x*-*z* section  $(y/R_p=0)$ , (b) t = 35s in the *x*-*y* section  $(z/R_p=-0.75)$ , (c) t = 600 s in the *x*-*z* section  $(y/R_p=0)$ , and (d) t = 600s in the *x*-*y* section  $(z/R_p=-0.75)$ . At t = 35 s, the pore pressure is relatively small (Fig.

13a), then it increases gradually with time (Fig. 13c). A similar trend of the pore 450 pressure build-up in the x-y section can be seen in Figs. 13b and 13d. In addition, Figs. 451 452 13b and 13d indicate that the largest pore pressure  $(p_r)$  appears at the locations near the front ( $\pi/4$  with respect to the incident wave direction) and rear ( $3\pi/4$ ) sides of the pile. 453 This distribution pattern is due to the significant change in wave surface elevations 454 occurring there (see the dashed line in Fig. 10). A significant change in wave loading 455 (owing to the wave height) will increase the shear stress ( $\tau_{xz}$ ,  $\tau_{yz}$ , and  $\tau_{xy}$ ) of the soil 456 skeleton. This would lead to a large source term (f) which generates a large pore 457 458 pressure there.

Fig. 14 shows the effects of wave period (T) on the residual pore pressure  $p_r$  (in 459 the x-y plane) in the vicinity of the mono-pile foundation (after 150 wave cycles). The 460 concerned points have been selected along a half-circle with  $S/R_p = 1$  and  $z/R_p = -1$ 461 (where S is the distance from the mono-pile surface). The angle  $\alpha$  denotes the relative 462 position with respect to the mono-pile, which varies from zero (at the front of the mono-463 pile) to  $\pi$  (at the rear of the mono-pile). Three dashed lines are plotted in Fig. 14 464 indicating the iso-pressure ( $p_r = 60, 120, \text{ and } 180 \text{ Pa}$ ) of the pore fluid. It was found that 465 an increase in wave period greatly increases the amplitude of residual pore pressures 466 within the vicinity of the mono-pile. That is, due to large wave periods generating a 467 large wave loading at the seabed surface, thereby promoting compression of the soil 468 particles. In addition, the residual pore pressure is found to increase and decrease in 469 470 front of and behind the mono-pile, respectively, which is due to the wave transformation. The shape of the pore pressure distribution becomes more symmetric with respect to 471

the pile centre when the wave period *T* increases (from 4 to 8 s). It is noted that the
above effects may be only important in relatively shallow water, as the phenomenon of
wave diffraction and reflection is usually more pronounced in shallow water.

Fig. 15 illustrates the effects of the permeability k, degree of saturation  $S_r$ , Young's 475 modulus  $E_s$ , relative density  $D_r$ , residual coefficient ( $\alpha_r$  and  $\beta_r$ ) on the residual pore 476 pressure around the mono-pile. The vertical distribution of the residual pore pressures 477 in front of the pile ( $x/R_p = -3.5$  and  $y/R_p = 0$ ) is plotted at t = 600 s (150 wave cycles). 478 The relatively low seabed permeability k results in poor drainage conditions, which 479 480 hinders the pore pressure dissipation in the seabed. This further leads to a relatively high residual pore pressure (Fig. 15a). The decrease in the degree of saturation 481 corresponds to the increase of the residual pore pressure (Fig. 15b). This is because the 482 483 decrease of saturation leads to an increase in the seabed shear stress ( $\tau_{xz}$ ), which in turn strengthens the compression of the soil. It is noted that the difference of pore pressure 484 between cases having  $S_r=0.992$  and  $S_r=0.985$  is much smaller than between  $S_r=0.985$ 485 and  $S_r=0.98$ . This indicates that in a nearly saturated seabed (e.g.  $S_r>0.985$ ), the residual 486 pore pressure does not change much with the increase in  $S_r$ . Fig. 15c and Fig. 15d shows 487 the effects of Young's modulus  $(E_s)$  and relative density  $(D_r)$  on the residual pore 488 pressure around the mono-pile. It illustrates that, the increase of  $E_s$  and  $D_r$  would cause 489 the decrease of the amplitude of residual pore pressure. This is because the relative large 490 Young's modulus  $(E_s)$  and soil relative density  $(D_r)$  corresponds to a relatively "dense" 491 seabed; which would be more difficult to be compressed by the wave loading. Fig. 15e 492 and Fig. 15f examine the effects of the coefficients  $\alpha_r$  and  $\beta_r$ , respectively. It is found 493

that the residual pore pressure increases with the decrease of  $\alpha_r$  and  $\beta_r$ . This is in accordance with the change in eq. (11) for the pressure source term. The decrease of  $\alpha_r$ and  $\beta_r$  would cause an increase of the source term ( $f_3$ ) (negative correction), leading to an increase of the pore pressure. This is obtained providing that the  $\tau_{ins3}/(\alpha_r \sigma'_{03})$  of Eq. 11 is less than 1. We can deserve it would have a positive correction between  $\beta_r$  and source term ( $f_3$ ) if  $\tau_{ins3}/(\alpha_r \sigma'_{03})$  is greater than 1.

500

### 501 **4.5 Effects of inertial terms on the accumulated pore pressure**

502 Three different numerical models, namely the FD, PD, and QS models, for seabed oscillatory mechanism were proposed to investigate the effects of the inertial terms of 503 the soil skeleton/fluid (Zienkiewicz et al., 1980). The governing equations for the FD 504 505 model are shown in eqs. (2)-(4). Ignoring the accelerations from the pore fluid and/or soil motion simplifies these general formulations into a conventional PD or QS model. 506 Fig. 16 illustrates the vertical distribution of the residual pore pressure with FD, 507 508 PD, and QS models. Here,  $\Delta p_{rl}$  denotes the discrepancy in the residual pore pressure between the QS and PD models, and  $\Delta p_{r2}$  denotes this discrepancy between the PD and 509 FD models. The selected section is directly in front of the pile  $(x/R_p=-1.53, y/R_p=0)$ . Fig. 510 16 shows that almost no discrepancy  $(\Delta p_{r2})$  is found between the PD and FD models. 511 This is because the inertial terms effects of pore fluid on the seabed shear stresses is 512 minor for the case with wave loading (Ulker and Rahman, 2009), which leads to a small 513 514 discrepancy in residual pore pressure. As the comparison shows, the simulated residual pore pressure using the QS model is smaller than that using the FD or PD model. This 515

indicates that the seabed residual response will be underestimated if the inertial termsof the pore fluid and soil skeleton are neglected.

Fig. 17 shows the vertical distributions of the relative difference of pore pressure 518  $(\Delta p_{rl}/\max(p_{OS}))$  in front of (point A), at the side (point B), and at the rear (point C) of 519 the mono-pile. Note that, the  $max(p_{OS})$  indicates the maximum pore pressure with QS 520 model. It is found that the relative difference  $(\Delta p_{rl}/\max(p_{OS}))$  first increases with the 521 increase in the seabed depth at the top layer (i.e.  $-0.8 \le z/R_p \le 0$ ), and then decreases as 522 the seabed deepens  $(z/R_p < 0.8)$ . This indicates that the largest effect of the inertial terms 523 on residual pore pressure is found at the middle part of the seabed  $(z/R_p=-0.8$  in this 524 study). In addition, the relative difference  $\Delta p_{rl}/\max(p_{OS})$  is found to be the largest in 525 front of the mono-pile (point A) comparing to that at point B (at the side) and point C 526 527 (at the rear). This indicates that the effect of the inertial term is largest in front of pile. This is because the wave loading in front of pile is larger due to wave reflection and 528 refraction. It should be noted that, the largest relative difference  $(\Delta p_{rl}/\max(p_{OS}))$  can 529 530 reach 3.8% and  $\Delta p_{r2}/\max(p_{OS}) \approx 0$  (depicted in Fig. 16). This indicates that the PD model should better be used and is sufficient in simulation of wave-induced residual 531 response. This conclusion is in accordance with Ulker and Rahman (2009) which is for 532 the oscillatory response, and is extended to the residual response of seabed by the 533 present study. However, it should be noted that, the effect of the inertial terms is overall 534 small (the maximum difference is  $3.8\% p_0$  in this study); which could be neglected in 535 the engineering practice. 536

537

# 538 **4.6 Significance of the mono-pile foundation for the accumulated pore pressure**

Fig. 18 shows a comparison of the accumulated pore pressure with and without a mono-539 pile foundation. The concern point is in front of the pile with coordinates of  $x/R_p = -3.5$ , 540  $y/R_p = 0$ , and  $z/R_p = -0.83$ . As shown in Fig. 18, the increase in accumulated pore 541 pressure is relatively faster at the early stage of the wave loading (t < 200 s). This then 542 gradually decelerates until the residual pore pressure reaches a relative steady state (t =543 600 s). Fig. 18 also shows that the time for the residual pore pressure to reach a 544 relatively steady value is approximately the same (t = 600 s) for the cases with and 545 546 without the mono-pile. This indicates that such a build-up pattern of pore pressure is independent of the magnitude of the wave loading at seabed surface. 547

The discrepancy in the maximum pressure value  $(\Delta p_r)$  between the two cases (with 548 549 and without a mono-pile) is defined as the significance of the mono-pile on the residual response of seabed. Fig. 19 illustrates  $\Delta p_r$  with various vertical locations (z), wave 550 steepness (H/L), permeability (k), and relative seabed densities  $(D_r)$ . It was found that 551 this significance  $(\Delta p_r)$  increases with an increase in depth at the upper part of the seabed 552  $(-0.83 \le z/R_p \le 0)$ , and then decreases when the seabed deepens  $(z/R_p \le -0.83)$  (Fig. 19a). 553 Figs. 19b–19d illustrate the change in  $\Delta p_r$  with various wave and seabed parameters 554  $(x/R_p = -3.5, y/R_p = 0, \text{ and } z/R_p = -0.83)$ . It was found that the significance of a mono-555 pile for the residual response of seabed increases with the increase in wave steepness 556 (H/L) (Fig. 19b), and decreases with the seabed permeability (k) (Fig. 19c) and relative 557 seabed density  $(D_r)$  (Fig. 19d). This is due to the fact that the increase in wave steepness 558 (H/L) will increase the magnitude of the residual pore pressure. Increases in seabed 559

permeability (k) and relative seabed density  $(D_r)$  will decrease the residual pore pressure

because they improve the soil drainage conditions (for k) and restrain the compression

562 of soil particles (for  $D_r$ ).

563

# 564 **4.7 Residual liquefaction**

565 Zen and Yamazaki (1990) proposed the following 1D liquefaction criterion:

$$-(\gamma_s - \gamma_w)z \le p_0 - p_{b0} \tag{22}$$

where  $p_0$  is the wave-induced pore pressure,  $p_{b0}$  is the dynamic wave pressure at the seabed surface, and  $\gamma_s$  and  $\gamma_w$  are the specific bulk weight of the soil (not the grains) and water, respectively.

Jeng (1997) extended this criterion to a 3D situation by adopting the concept of average effective stress, namely,

$$-(\gamma_{s} - \gamma_{w})\frac{1 + 2k_{0}}{3}z \le p_{0} - p_{b0}$$
(23)

571 where  $k_0$  is the lateral compression coefficient of the soil.

572 The above criteria are only suitable for the cases without a marine structure. When a structure is present, the soil skeleton in the vicinity of the structure will be compressed, 573 which suppresses the occurrence of soil liquefaction (Jeng, 1997). In addition, Eqs. (22) 574 and (23) provide the criteria for an instantaneous liquefaction (Sumer, 2014), which is 575 likely to occur in a sandy seabed. For a silt seabed, the residual mechanism dominates 576 the seabed response. Therefore, liquefaction is mainly due to the excess residual pore 577 pressure  $(p_r - 0)$  caused by the compression of soil skeleton (Liao et al., 2015). 578 Following previous studies (Ye, 2012; Liao et al., 2015), the residual liquefaction 579

580 criterion that considers the weight of mono-pile can be expressed as follows:

$$\sigma_{z0}' \le p_r \tag{24}$$

where  $\sigma'_{z0}$  is the initial normal effective stress obtained from the final state of 581 consolidation (see section 4.1). The dynamic Biot equation is adopted in this study, 582 based on the assumption that seabed is seen as a porous elastic media (Sumer, 2014). It 583 has to be clarified that the present work only predicts the potential liquefaction depth 584 rather than simulating the real liquefaction process. In this section, the present model is 585 adopted to investigate the wave-induced residual liquefaction potential around a mono-586 587 pile, taking into consideration the state of the seabed consolidation. The parameters utilized in this simulation are listed in Table 2. 588

Fig. 20 illustrates the residual pore pressure  $(p_r)$  and liquefaction depth  $(l_d)$  around 589 590 the mono-pile at t = 40 s (a, c, and e) and t = 792 s (b, d, and f). Initially (t = 40 s), a relatively large residual pore pressure mainly appears within the vicinity of the mono-591 pile (-5 $<x/R_p<5$ , -3 $< y/R_p<3$ , zone A), whereas the location far from the pile ( $|x/R_p|>5$ , 592 593  $|y/R_p|>3$ , zone B) have a relatively small pore pressure (Fig. 20a). As a result, Figs. 20c and 20e illustrate that liquefaction only occurs close to the mono-pile foundation (zone 594 A). Comparing with Fig. 18a, the pore pressure at t = 792 s is largely increased 595 especially away from the pile (zone B) (Fig. 20b). Correspondingly, the liquefaction 596 depth ( $l_d$ ) significantly increases in zone B ( $l_d/R_p$  increases from 0 to 1.2 m) (Figs. 20d 597 and 20f). It was also found that the largest liquefaction depth  $(l_d)$  appears at the rear of 598 the pile instead of at the front, which is consistent with the main findings of Li et al. 599 (2011). Fig. 20f also shows that the liquefaction zone affected by the presence of the 600

mono-pile (shown as the red dashed line) is approximately three-times the pile diameters (one pile diameter in front and two times the diameter in the rear) in length along the wave propagation direction (*x*-direction), and one pile diameter in width at the sides of the pile (*y*-direction). This demonstrates that the seabed in this area is prone to be liquefied, which therefore requires a special concern in engineering practice.

Fig. 21 illustrates the liquefaction depth  $(l_d)$  around the mono-pile with various 606 embedded depths of the pile  $(de/R_p)$ . The case without a pile foundation is also included 607 for comparison. It was found from Fig. 21a that, for the case without the mono-pile, the 608 609 liquefaction depth remains as  $z/R_p$ =-1.2 in the x-direction, which is consistent with the main conclusion of Jeng and Zhao (2015) that a 1D pattern of the liquefaction depth 610 exists at the final state. The previous studies mostly investigate the seabed response 611 around composite breakwater (de = 0) (Zhao et al., 2017b), and this study further 612 reveals the liquefaction depth when a structure (e.g. a mono-pile) is embedded into the 613 seabed (de > 0). It is found that, the liquefaction depth is small adjacent to the pile 614 615 foundation due to the fact that the large resistance of soil skeleton exists there. In addition, the liquefaction depth  $(l_d)$  would decrease as the inserted depth of the pile is 616 increased. This is mainly due to the decrease in the initial effective stress ( $\sigma'_{z0}$ ) with 617 increasing inserted depth (Sui et al., 2017), thereby leading to a significant decrease in 618 soil overburden pressure. 619

The failure of seabed around the structure may also occur even with the a partially liquefied seabed. Such a partially liquefied seabed is usually referred to as Partial Fluidized Sediments (PFS) (Sumer, 2014; Zhang S. et al., 2018). The fluidization 623 degree  $(f_d)$  (the ratio of excess pore pressure to the initial effective stress) is used to 624 describe how much of sediment is fluidized, given by:

$$f_d = u/\sigma'_{\nu 0} \tag{25}$$

In which, u is the excess pore pressure and is equal to  $p_r$  in the present study.

Full liquefaction is seen as corresponding to 100% fluidization ( $f_d$ ) as indicated in 626 Equation (24) (Jeng, 2013; Kirca et al., 2013; Liao et al., 2018; Sumer, 2014; Zen and 627 Yamazaki, 1990). As indicated above, full liquefaction is seen as the extreme state of 628 the sediment fluidization (again, 100% fluidization) process, corresponding to the 629 630 complete loss in resistance of the soil skeleton. This is a useful indicator to describe the most dangerous situation of the seabed. The relative cyclic shear stress ratio  $\tau_c/\sigma'_{\nu 0}$  (CSR, 631 contributing to the source part  $f_3$  in Eq. 11 plays an important role as it generates the 632 633 excess pore pressure.

Fig. 22 shows the fluidization degree  $p/\sigma'_{\nu0}$  as well as the relative cyclic shear 634 stress ratio (CSR) in front of and at the rear of the pile (see below). It is seen that the 635 636 fluidization degree is zero at the seabed surface, and increases with time (Fig. 22a and Fig. 22b). Relatively large fluidization degree is found at the rear of the mono-pile. This 637 indicates that liquefaction would easily happen there  $(u/\sigma'_{\nu 0} \text{ exceeds 1})$ . This is mainly 638 because the cyclic shear stress ratio (CSR) is larger at the rear of the pile, which 639 promotes more compression of the soil (Fig. 22c and Fig. 22d). The above conclusion 640 is consistent with the previous study of Li et al. (2011) and Fig. 20 of the present study. 641 Fig 22 a and b shows that the ratio  $u/\sigma'_{\nu 0}$  is greater than 1 at the seabed surface, 642 indicating that the soil particles have a potential trend to be separated (liquefaction 643

already happens there) (Liu et al., 2015). This  $(u/\sigma'_{v0}>1)$  occurs because of the use of the elastic model, and cannot be rigorously avoided as there is no yielding criteria. In fact, the maximum liquefaction depth predicted with elastic model may be larger than that with a plastic model, due to the fact that the pore pressure is difficult to release with the elastic model (Qi and Gao, 2018). Such conservative approach benefits the foundation design in engineering practice.

Fig. 23 illustrates the effects of the relative density  $D_r$  on the liquefaction depth 650 around mono-pile. It is found that the liquefaction depth decreases with the increase of 651 652 the relative density  $D_r$ . This is because the relatively large  $D_r$  means a much more sand densification with a relatively low residual pore pressure generation. Noted that, there 653 may be pre-shearing effects during the wave loading process (de Groot et al., 2006a; de 654 655 Groot et al., 2006b; Meijers and Luge, 2012). The pre-shearing effect will increase the soil relative density  $(D_r)$  due to the sand densification. Therefore, Fig. 23c indicates that 656 the pre-shearing effects can decrease the maximum liquefaction depth, which is in 657 accordance with the main finding of Meijers and Luger (2012). This can provide the 658 readers a basic understanding on the pre-shearing effects. 659

Fig. 24 illustrates the parametric studies for the effects of coefficients  $\alpha_r$  and  $\beta_r$  on the liquefaction depth around mono-pile foundation. Fig. 24 shows that, the liquefaction depth decreases with the increase of  $\alpha_r$  while it decreases with the decrease of  $\beta_r$ . This figure also shows, the liquefaction depth decreases with the increase of  $\alpha_r$  while it decreases with the decrease of  $\beta_r$ . It is also found, effects of coefficient  $\alpha_r$  on the liquefaction depth far away the pile is much larger than that near the pile. While for the

coefficient  $\beta_r$ , the location at the vicinity of the pile owns the larger effects. With 666 comparison, the effects of  $\beta_r$  on the liquefaction depth is obviously smaller than that of 667 the  $\alpha_r$ . Noted that, the scale effect may occur when the results are extended to the 668 application in the real environment. The empirical formulas (Eq. 12 and Eq. 13) 669 involving  $\alpha_r$  and  $\beta_r$ , are obtained from a curve-fit exercise to the large-scale simple shear 670 test data (De Alba, 1976; Sumer et al., 2012). It should be noted that, in this sense, the 671 present simulations have already considered the scale effects based on the full-scale 672 coefficients used. 673

674 Partial liquefaction is also often found in the real environment, which would cause a large decrease in soil effective stress and thus leading to the instability of the 675 foundation. Partial liquefaction may happen if the fluidization degree  $p_r/\sigma' z$  is greater 676 677 than e.g. 0.5 (it is 1 for the full liquefaction, see Eq. 24) (de Groot et al., 2006a; de Groot et al., 2006b). Fig. 25 shows the comparison of liquefaction depth between the 678 full liquefaction criteria and partial liquefaction criteria. It illustrates that the estimated 679 680 liquefaction depth with partial liquefaction criteria is much larger than that with the full liquefaction. This indicates that the design strategy with a partial liquefaction criteria 681 should be much safer in the practical offshore engineering. It is also found that, with 682 the partial liquefaction criteria, the liquefaction depth near the pile does not change 683 much comparing to that far away from the pile. This indicates that the effect of the 684 presence of pile on the liquefaction depth is much weaker with the partial liquefaction 685 686 criteria.

687

# 688 **5.** Conclusion

In this study, based on a non-linear Boussinesq wave model and FD seabed model, a 689 690 3D integrated numerical model was developed to investigate the wave-induced residual response of the seabed around a mono-pile foundation. Experimental data from five 691 flume tests were used to validate the present model. Good agreement between the 692 measured data and numerical simulations was obtained. The validated model was then 693 applied to investigate the pore pressure accumulation around a mono-pile foundation. 694 Considering the self-gravity of the pile, the wave-induced 3D liquefaction zone around 695 696 an embedded pile foundation was investigated. The following conclusions were drawn: (1) The present numerical model adopting the definition of the 3D source term  $f_3$ 697 can provide reliable results with regard to pore pressure accumulation around a marine 698 699 structure.

(2) Wave diffraction and reflection increase and decrease the residual pore pressure in front and at the rear of a mono-pile, respectively. Effects of wave diffraction/reflection increase with an increase in wave height (H) and a decrease in wave period (T), seabed permeability (k), and relative density ( $D_r$ ).

(3) The increase of the residual pore pressure is relatively faster during the early
stage of wave loading, then gradually decelerates until the pore pressure reaches a
relatively high value. Such the build-up pattern of pore pressure is independent of the
magnitude of wave loading.

(4) This study presents a direct comparison among the FD, PD, and QS models. Itis found that the wave-induced residual response would be underestimated if the inertial

terms of pore fluid and soil skeleton are neglected. The above effect from the inertial
terms is overall minor which may be neglected in the engineering practice. The PD
model is recommend to use if a high simulation accuracy is needed for e.g. scientific
research.

- (5) The presence of pile restrains the residual liquefaction adjacent to the pile
  surface, and the maximum liquefaction depth increases with an increase in the inserted
  depth of pile.
- In this study, a new 3D residual model is established and the effects of the wave reflection/diffraction, inertial terms and various inserted depth of pile on the seabed residual response are investigated. Other factors, such as current, random waves (Meijers et al., 2014) and pre-shearing (Meijers and Luger, 2012) may affect the liquefaction and will be examined in future study.
- 722

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## 932 Table lists:

Parameters		Notations	Magnitudes	Units
	Radius	$R_p$	2.5	m
Pile foundation	Young's modulus	$E_p$	10	GPa
	Poisson's ratio	$\mu_p$	0.25	-
	Pile length	l	14	m
	Embedded depth	de	8	m
	Depth	d	4	m
Wave	Density	$ ho_f$	1000	Kg/m <sup>3</sup>
	Wave height	Н	0.2-0.4	m
	Wave period	Т	4-8	S
	Permeability	k	2×10 <sup>-4</sup> -4×10 <sup>-5</sup>	m/s
	Porosity	п	0.425	-
Seabed	Relative density	$D_r$	0.28-0.32	-
	Saturation degree	$S_r$	0.98-0.992	-
	Poisson's ratio	$\mu_s$	0.35	-
	Young's modulus	$E_s$	5×10 <sup>6-</sup> 7×10 <sup>7</sup>	Pa

## Table 1. Parameters used in the first application case

934

Parameters		Notations	Magnitudes	Units
	Radius	$R_p$	1.5	m
Pile foundation	Young's modulus	$E_p$	10	GPa
	Poisson's ratio	$\mu_p$	0.3	-
	Pile length	l	24	m
	Embedded depth	de	0-12	m
	Depth	d	8	m
Wave	Density	$ ho_f$	1000	Kg/m <sup>3</sup>
	Wave height	Н	3	m
	Wave period	Т	8	S
	Permeability	k	1×10 <sup>-5</sup>	m/s
	Porosity	п	0.3	-
Seabed	Relative density	$D_r$	0.28	-
	Saturation degree	$S_r$	0.992	-
	Poisson's ratio	$\mu_s$	0.3	-
	Young's modulus	$E_s$	1.6×10 <sup>8</sup>	Pa

Table 2. Parameters used in the second application case



Fig. 1 Mechanisms of wave-induced pore pressures (not to scale)



Fig. 2 (a) 3D Sketch and (b) boundary conditions of the present model in which d is the water depth, de is the embedded depth of the pile,  $R_p$  is the pile radius.



Fig. 3 Integrating procedure of the present numerical model.



Fig. 4 Comparison of simulated (this study) and measured wave elevations (Cong et al. 2015) considering wave-pile interactions



Fig. 5 Comparison of simulated (this study) and measured (Lu, 2005) wave induced dynamic pore water pressure at different soil depth for (a, c, e, g) linear wave and (b, d, f, h) cnoidal wave.



Fig. 6 Comparisons of simulated (Li and Jeng (2008), Sumer et al., (2012), this study) and measured (Sumer et al., (2012)) pore pressure build-up within a silt seabed (the oscillatory part of pore pressure is manually removed here).



Fig. 7 Comparisons of simulated (this study) and measured (Kirca, 2013) pore pressure build-up within a silt seabed with a standing wave (the oscillatory part of pore pressure is manually removed here).



Fig. 8 Comparisons of simulated (this study) and measured (Qi and Gao, 2014) pore pressure vertical distributions around a mono-pile foundation.



Fig. 9 The consolidation state of seabed for (a) pore pressure, (b) effective stress and (c) subsidence in the vicinity of a mono-pile (H=0 m,  $k=2\times10^{-5}$  m/s, de=8 m).



Fig. 10 3D distribution of wave surface elevation around a mono-pile for (a) t=2.95T, (b) t=3.2T, (c) t=3.45T and t=3.70T. (H=0.2 m)



Fig. 11 Distribution of (a) oscillatory pore pressure ( $p_{ins}$ ), (b) horizontal effective stress ( $\sigma'_x$ ), (c) vertical effective stress ( $\sigma'_x$ ) and (d) shear stress ( $\tau_{xz}$ ) around a mono-pile for *t*=0.5T (*H*=0.4 m)



Fig. 12 Cyclic horizontal displacements of the pile  $(u_x)$  under the dynamic wave loading for one wave period (*H*=0.4 m)



Fig. 13 3D temporal and spatial features of the accumulated pore pressure  $(p_r)$  around a mono-pile for (a)  $p_r$  at t=35 s in x-z section  $(y/R_p=0)$ , (b)  $p_r$  at t=35 s in x-y section  $(z/R_p=-0.75)$ , (c)  $p_r$  at t=600s in x-z section  $(y/R_p=0)$  and (d)  $p_r$  at t=600 s in x-y section  $(z/R_p=-0.75)$ .  $(H=0.2 \text{ m}, k=1\times10^{-5} \text{ m/s}, D_r=0.28, de=8 \text{ m})$ 



Fig. 14 Effects of wave periods (*T*) on residual pore pressure (*x-y* plane) in the vicinity of a monopile foundation. (*H*=0.2 m,  $k=1\times10^{-5}$  m/s,  $D_r=0.28$ ,  $z/R_p=-1$ ,  $S/R_p=1$ , de=8 m, t=600 s)



Fig. 15 Vertical distribution of the residual pore pressures (*t*=600 s, *T*=4 s, *H*=0.2 m, *d*=4 m) for (a) different seabed permeability *k* (*S<sub>r</sub>*=0.992, *E<sub>s</sub>*=5×10<sup>7</sup> N/m<sup>2</sup>, *D<sub>r</sub>*=0.28), (b) Saturation degree *S<sub>r</sub>* (*k* = 2×10<sup>-5</sup> m/s, *E<sub>s</sub>*=5×10<sup>7</sup> N/m<sup>2</sup>, *D<sub>r</sub>*=0.28), (c) Young's modulus *E<sub>s</sub>* (*k* = 2×10<sup>-5</sup> m/s, *S<sub>r</sub>*=0.992, *D<sub>r</sub>*=0.28), (d) relative density *D<sub>r</sub>* (*k*=2×10<sup>-5</sup> m/s, *S<sub>r</sub>*=0.992, *E<sub>s</sub>*=5×10<sup>7</sup> N/m<sup>2</sup>), (e) coefficient  $\alpha_r$ (*S<sub>r</sub>*=0.992, *E<sub>s</sub>*=5×10<sup>7</sup> N/m<sup>2</sup>,  $\beta_r$ =-0.356) and (f) coefficient  $\beta_r$ =-0.356 (*S<sub>r</sub>*=0.992, *E<sub>s</sub>*=5×10<sup>7</sup> N/m<sup>2</sup>,  $\alpha_r$ =0.175).



Fig. 16 Vertical distributions of the residual pore pressure simulated by Fully-Dynamic model, Partial-Dynamic model and Quasi-Static model. (*H*=0.4 m,  $k=2\times10^{-4}$  m/s,  $D_r=0.28$ , de=8 m,  $E_s=5\times10^6$  N/m<sup>2</sup>)



Fig. 17 Vertical distribution of pore pressure discrepancy ( $\Delta p_{rI}/\max(p_{QS})$ ) in front (Point A), at sides (Point B) and at rear (Point C) of a mono-pile foundation ( $\max(p_{QS})$ ) is the maximum pore pressure with Quasi-static model) (H=0.4 m, k=2×10<sup>-4</sup> m/s,  $D_r$ =0.28, de=8 m,  $E_s$ =5×10<sup>6</sup> N/m<sup>2</sup>).



Fig. 18 Comparisons of the accumulated pore pressure with and without a mono-pile foundation at the location of  $x/R_p$ =-3.5,  $y/R_p$ =0,  $z/R_p$ =-0.83. (*H*=0.2 m, *k*=2×10<sup>-5</sup> m/s, *D<sub>r</sub>*=0.28, *de*=8 m)



Fig. 19 Significance of mono-pile foundation against various (a) vertical locations (z) (*H/L*=0.01,  $k=2\times10^{-5}$  m/s,  $D_r=0.28$ ), (b) wave steepness (*H/L*) ( $k=2\times10^{-5}$  m/s,  $D_r=0.28$ ), (c) seabed permeability (k) (*H/L*=0.01,  $D_r=0.28$ ) and (d) seabed relative density ( $D_r$ ) (*H/L*=0.01,  $k=2\times10^{-5}$  m/s). (de=8 m)



Fig. 20 Residual pore pressure  $(p_r)$  and liquefaction depth (*ld* in the *x*-*z* plane and *x*-*y* plane) in the vicinity of a mono-pile at *t*=40 s (a, c and e) and *t*=792 s (b, d and f) (*de*/*R*<sub>*p*</sub>=8).



Fig. 21 Liquefaction depth (*ld*) around a mono-pile for different pile embedded depth (*de*).



Fig. 22 The ratio of excess pore pressure  $u/\sigma'_{v0}$  (the fluidization degree) (a, b) as well as the relative cyclic shear stress amplitude (CSR) (c, d) in front ( $x/R_p$ =-2.5) and at the rear ( $x/R_p$ =2.5) of the mono-pile ( $de/R_p$ =8).



Fig. 23 Effects of soil relative density  $D_r$  on the liquefaction depth around the pile ( $de/R_p=8$ ).



Fig. 24 Effects of (a) coefficients  $\alpha_r$  ( $\beta_r$ =-0.356) and (b) coefficients  $\beta_r$  ( $\alpha_r$ =0.175) on the liquefaction depth around the pile ( $de/R_p$ =8).


Fig. 25 Comparison of liquefaction depth with full liquefaction criteria and partial liquefaction criteria around the pile foundation ( $de/R_p=8$ ).