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Investigation of nonlinear wave-induced seabed response around mono-pile foundation

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Abstract: Stability and safety of offshore wind turbines with mono-pile foundations, affected by nonlinear wave effect and dynamic seabed response, are the primary concerns in offshore foundation design. In order to address these problems, the nonlinear wave effect on dynamic seabed response in the vicinity of mono-pile foundation is investigated using an integrated model, developed using OpenFOAM, which incorporates both wave model (waves2Foam) and Biot's poro-elastic model. The present model was validated against several laboratory experiments and promising agreements were obtained. Special attention was paid to the systematic analysis of pore water pressure as well as the momentary liquefaction in the proximity of mono-pile induced by nonlinear wave effects. Various embedment depths of mono-pile relevant for practical engineering design were studied in order to attain the insights into nonlinear wave effect around and underneath the mono-pile foundation. By comparing time-series of water surface elevation, inline force, and wave-induced pore water pressure at the front, lateral, and lee side of mono-pile,

29 the distinct nonlinear wave effect on pore water pressure was shown. Simulated results confirmed
30 that the presence of mono-pile foundation in a porous seabed had evident blocking effect on the
31 vertical and horizontal development of pore water pressure. Increasing embedment depth
32 enhances the blockage of vertical pore pressure development and hence results in somewhat
33 reduced momentary liquefaction depth of the soil around the mono-pile foundation.

34
35 **Key words:** wave-structure-seabed interaction (WSSI); dynamic seabed response; mono-pile
36 foundation; blockage effect; momentary liquefaction

37 38 **1. Introduction**

39 Demand for green energy in response to climate change has driven a substantial increase of
40 construction of offshore wind farms in the past decades, which is likely to continue in the
41 forthcoming years. Large diameter mono-pile is the preferred foundation for offshore wind turbines
42 located in shallow or intermediate water depths. Mono-pile foundation supporting offshore wind
43 turbine may suffer the damage from strongly nonlinear, and even breaking waves. The soil near a
44 mono-pile foundation may be liquefied under wave loading and in turn aggravate the vibration of the
45 offshore wind turbine. Understanding these mechanisms and accurate prediction of their influences
46 on mono-pile foundations are therefore particularly important in engineering design.

47
48 In recent decades, wave-induced hydrodynamic loads acting on the cylindrical structure have been
49 extensively studied since they are of primary concern in offshore engineering. The costly and
50 time-consuming laboratory experiments cannot provide a complete set of results on wave-structure
51 interaction. Consequently, the numerical models of wave-structure interaction have been increasingly
52 used. Based on potential theory and the assumption that flow is inviscid and irrotational, various
53 numerical analyses of linear and weakly non-linear wave-structure interactions have been presented.
54 To study the three-dimensional (3D) wave-structure interaction, Ma et al. (2001a, 2001b)
55 numerically solved the fully nonlinear potential flow with Finite Element Method (FEM)
56 incorporating recovery technique to obtain better solution. The same approach was used by Kim et al.
57 (2006) to investigate wave run-up around cylinders with steeper Stokes waves. The technique of

58 domain decomposition with enforcing continuity of the interface between neighbour subdomains was
59 implemented by Bai and Taylor (2007, 2009) to examine fully nonlinear wave interaction with
60 vertical cylinder. However, the potential flow theory is limited to non-breaking and small steepness
61 waves (small H/L , where H is the wave height, and L is the wave length). The alternative that is
62 becoming increasingly popular is to use Computational Fluid Dynamics (CFD) for investigating high
63 steepness wave interacting with offshore structures, including breaking wave effect and higher-order
64 harmonic forces. Recent CFD computations within the framework of OpenFOAM based on Finite
65 Volume Method (FVM), a free access source C++ library for various fluid flow and solid mechanics
66 problems, have been performed to obtain the insights into fully nonlinear wave-structure interactions.
67 Using the wave generation tool waves2Foam (Jacobsen et al., 2012), Paulsen et al. (2014b)
68 investigated the capacity of OpenFOAM for modelling nonlinear wave motion interacting with
69 mono-pile foundation for a range of Keulegan–Carpenter (KC) numbers, $KC = U_m T/D$, where U_m is
70 the maximum velocity, T is wave period and D is the diameter of cylinder (Sumer and Fredsøe, 2006),
71 and concluded that the dominant physics of wave-pile interactions was well predicted, despite the
72 simplification of cylinder wall and the seabed surface boundary conditions. Paulsen et al. (2014a)
73 introduced an innovative domain decomposition approach to integrate potential flow theory model
74 (OceanWave3D) developed by Engsig-Karup et al. (2009) and waves2Foam library (Jacobsen et al.,
75 2012) based on Navier-Stokes (NS) equations and volume of fluid method (VOF). Good agreement
76 between numerical and experimental results has been obtained for several sensitivity tests of wave
77 loads on a cylindrical pile foundation. A comprehensive investigation of the potential of OpenFOAM
78 for accurately predicting the interactions between wave and vertical cylinder was elaborated by Chen
79 et al. (2014) for a variety of wave conditions, including regular and focused waves. Higuera et al.
80 (2013a) developed an advanced wave generation tool and the active wave absorption boundary
81 condition (IHFOAM) for predicting wave interaction with coastal structures in coastal engineering
82 (Higuera et al., 2013b; Higuera et al., 2014a; Higuera et al., 2014b). A moving boundary condition
83 with multi-paddles for wave generation is further incorporated into IHFOAM (Higuera et al., 2015)
84 together with an improved active wave absorption boundary. Nevertheless, the research solely
85 concerning the mechanism of wave interacting with offshore structure does not fully cover the
86 complexity of realistic design issues.

87

88 Another important issue in offshore engineering is the risk associated with formation of liquefied
89 zone of seabed as a consequence of wave-induced dynamic seabed response in the vicinity of
90 offshore structures (Sumer, 2014; Sumer and Fredsøe, 2002; Ye et al., 2016; Ye et al., 2015).
91 Liquefaction can be caused by two different mechanisms which occur at different time-scales, so we
92 distinguish between residual and momentary liquefaction. Residual liquefaction typically occurs in
93 undrained soils, when the pore water pressure accumulated over time exceeds overburden pressure
94 (Sumer, 2014). A much shorter-lived phenomenon, termed momentary liquefaction, occurs in an
95 unsaturated seabed, due to the direct effect of wave pressure imposed on seabed surface under wave
96 trough. The resulting fast decrease of pore water pressure in the unsaturated seabed generates large
97 upwards pressure gradients. If the lift induced by upward gradient of pore water pressure surpasses
98 the submerged weight of soil, effective stress vanishes and the soil is liquefied. From geotechnical
99 aspect, the occurrence of liquefaction under extreme wave impact during storm conditions may result
100 in the failure of the supporting foundation of an offshore structure, as well as foundation protection.
101 The relationship between momentary liquefaction and extreme wave interaction with mono-pile
102 foundation is the primary focus of present study.

103

104 In past decades, the analytical studies of wave-induced seabed response have also been extensively
105 carried out. Madsen (1978) and Yamamoto et al. (1978) extended the poro-elastic Biot's theory (Biot,
106 1941) to a close-form analytical solution for the examination of wave-induced seabed response.
107 Afterwards the investigation of wave-induced response for both coarse and fine sand, using a
108 boundary-layer approximation, was conducted by Mei and Foda (1981). They pointed out that their
109 approach can be used to economically solve poro-elastic boundary value problem with a free surface.
110 Using a simpler analytical solution, Okusa (1985) studied wave-induced stability of completely or
111 partially saturated seabed with a conclusion that Skempton's pore pressure coefficient played a key
112 role in predicting wave-induced seabed response. Hsu and Jeng (1994) analytically derived a
113 closed-form solution to investigate wave-induced soil response within the case of a finite thickness
114 seabed. A good agreement was found between their results and semi-analytical solution (Yamamoto
115 et al., 1978). After then, a thorough review on research of wave-induced dynamic seabed response

116 was described by Jeng (2003), where both theoretical and physical studies are included and examined
117 in detail. Most recently, with the fully dynamic soil behaviour considered, Liao et al. (2013)
118 presented an analytical study of combined effect of wave and current over an infinite seabed. It was
119 noted that the effect of currents on the seabed response was significant only in the upper area closed
120 to seabed surface (about 10% of wave length). Nevertheless, the aforementioned analytical
121 investigations are limited to given assumptions and scenarios.

122
123 To improve understanding of the entire wave-induced seabed response multiple physical experiments
124 were conducted with/without structures. Based on the laboratory experiments in a wave flume,
125 Sumer et al. (2006) elaborated the mechanism of wave-induced liquefaction and consecutive
126 compaction of a flat seabed without structures, and suggested that the completion of compaction and
127 final equilibrium with continuing waves produces ripples. The laboratory experiments of Sumer et al.
128 (2007) confirmed that when the progressive wave was greater than critical wave height, the soil
129 around a pile, that was freshly settled without liquefaction history, may experience liquefaction after
130 installation. In the dense-silt scour tests, it was also demonstrated that the scour around the pile may
131 occur after liquefaction and compaction. Liu et al. (2015) conducted one-dimensional (1D) soil
132 column experiments to investigate wave-induced pore water pressure in various sandy soil conditions.
133 The soil thickness was found to decrease due to the dynamic loading. Though the realistic
134 mechanism of wave-induced seabed response is easily captured by using natural materials, physical
135 experiments are relatively expensive to carry out and restricted to the limited-scale cases.

136
137 Numerical modelling has been broadly employed as a cost-effective method for investigating seabed
138 response induced by various wave conditions. Li et al. (2011) used FEM approach to numerically
139 solve the 3D Biot's equations without considering wave diffraction in their model. Wave-induced
140 seabed response around pile foundation, including transient and residual pore water pressure, was
141 examined for different pile diameters. However, in this study, the incident wave was simplified as an
142 analytical solution, so that the complicated wave-structure interaction was not taken into
143 consideration. The rapid development of computing resources enables researchers to couple flow
144 model with seabed model into an integrated model, which enables them to systematically investigate

145 the mechanisms of seabed response to waves in the vicinity of offshore structures, such as pipelines
146 (Lin et al., 2016; Zhao et al., 2016a; Zhao et al., 2016b; Zhao et al., 2014) and breakwaters (Jeng et
147 al., 2013; Jianhong et al., 2014; Jianhong et al., 2013; Ye et al., 2013a; Ye et al., 2013b). In the
148 previous studies, the equations governing fluid and soil behaviour were solved by different methods,
149 namely flow field by FVM and soil model by FEM. A monolithic approach to both models was used
150 in Lin et al. (2016), who developed an integrated FEM Wave-Seabed-Structure Interaction (WSSI)
151 model to explore the wave-induced liquefaction potential in the vicinity of a partially/fully buried
152 pipeline in an open trench. As an alternative approach, Liu et al. (2007) first discretized the Biot's
153 equations in a FVM manner within OpenFOAM, and then investigated the wave-induced response
154 around the submerged object without parallel computing. Tang et al. (2015) and Tang (2014)
155 extended and modified the poro-elastic Biot's model to poro-elasto-plasticity soil model. However,
156 so far majority of integrated models have focused on the investigation of
157 wave-pipeline/breakwater-seabed interaction. For the wave-pile-seabed interaction, a numerical
158 study based on FVM-FEM approach carried out by Chang and Jeng (2014) showed that replacing the
159 soil around a high-rising structure foundation was an effective protection against liquefaction. The
160 only available numerical model of WSSI focuses solely on the dynamic seabed response induced by
161 weakly nonlinear waves or regular non-breaking waves. Recently, Sui et al. (2015) integrated
162 FUNWAVE (Kirby et al., 2003; Shi et al., 2001; Wei and Kirby, 1995) and fully dynamic (FD) form
163 of Biot's equations to investigate the small steepness wave-induced seabed response around
164 mono-pile without considering fully nonlinear wave-pile interaction. In their study, dynamic
165 response of porous seabed, structural dynamics of mono-pile, and their interactions were all solved
166 by FD form of Biot's equations. However, the nonlinear wave-pile interaction has a significant effect
167 on porous seabed response. This complex process is not fully studied in the aforementioned studies.
168 Consequently, an integrated WSSI numerical model capable of accurately estimating strongly
169 nonlinear wave load and the corresponding dynamic seabed response provides an efficient tool for
170 the design of offshore wind turbine foundations.

171

172 This paper presents a sophisticated WSSI numerical model developed in order to aid the design for
173 offshore wind turbine foundations. A segregated FVM solver is implemented within the framework

174 of OpenFOAM, incorporating waves2Foam and Biot's equations, to address the issue of nonlinear
 175 wave-induced dynamic seabed response surrounding mono-pile foundation. The description of wave
 176 and seabed model is outlined in Section 2. Section 3 presents the validation of present model against
 177 several available experimental data sets. In Section 4 the calibrated model is used to investigate the
 178 nonlinear wave-induced dynamic seabed response, as well as the liquefaction potential, around
 179 mono-pile foundation. The main conclusions are listed in Section 5.

180

181 2. Numerical model

182 Figure 1 shows a sketch of simulation domain for the WSSI numerical model developed in this study.
 183 The domain includes two sub-domains: the sea water (including the air above the free surface) and
 184 the porous bed. The two corresponding sub-models, namely waves2Foam (Jacobsen et al., 2012) and
 185 QS (quasi-static) Biot's model, are integrated into the present WSSI model. The flow field is
 186 described by the incompressible Navier-Stokes equations with water-air interface traced by Volume
 187 of Fluid method (Berberović et al., 2009; Hirt and Nichols, 1981). The dynamic behaviour of a
 188 porous seabed is governed by QS Biot's equations, which contain both the pore water pressure and
 189 soil displacement. The process of integration is implemented by extended general grid interpolation
 190 (GGI), which interpolates the face and point from zone to zone for non-conformal meshes at the
 191 wave-seabed interface (Tukovic et al., 2014).

192

193 2.1 Wave model

194 The governing equations for simulating two-phase incompressible flow dynamics are

$$\nabla \cdot \mathbf{u} = 0 \quad (1)$$

$$\frac{\partial \rho \mathbf{u}}{\partial t} + \nabla \cdot (\rho \mathbf{u}) \mathbf{u}^T = -\nabla p^* - (\mathbf{g} \cdot \mathbf{x}) \nabla \rho + \nabla \cdot (\mu \nabla \mathbf{u}) \quad (2)$$

$$\frac{\partial \alpha}{\partial t} + \nabla \cdot \mathbf{u} \alpha + \nabla \cdot \mathbf{u}_r \alpha (1 - \alpha) = 0 \quad (3)$$

195 where \mathbf{u} is velocity field; ρ is fluid density; t is time; $p^* = p - \rho \mathbf{g} \cdot \mathbf{x}$ is the modified pressure
 196 which removes the effect of static pressure from the momentum equation (2); \mathbf{g} and \mathbf{x} are gravity
 197 acceleration and Cartesian coordinate vector, respectively; p is total pressure; μ is dynamic
 198 viscosity; \mathbf{u}_r is relative velocity field (Berberović et al., 2009); α is scalar field of volume fraction

199 function. α is equivalent to 1 when the computational cell indicates water field, while $\alpha = 0$
 200 indicates the simulated field to be air, and the water-air mixture field is denoted by $0 < \alpha < 1$. The
 201 momentary flow density and dynamic viscosity are computed by following equations:

$$\rho = \alpha\rho_w + \rho_a(1 - \alpha) \quad (4)$$

$$\mu = \alpha\mu_w + \mu_a(1 - \alpha) \quad (5)$$

202 where the sub-indices w and a represent water and air, respectively.

203

204 Consistently with the investigation by Paulsen et al. (2014b), where boundary layer effects were not
 205 taken into consideration, slip boundary condition is specified on the seabed, mono-pile surface, and
 206 lateral boundaries of the numerical wave flume. The atmospheric boundary at the upper boundary of
 207 flow domain is selected as a pressure outlet condition. The more comprehensive description of wave
 208 generation (inlet boundary) and wave absorption (outlet boundary) zone can be found in Jacobsen et
 209 al. (2012).

210

211 2.2 Seabed model

212 In present study, QS Biot's equations (Biot, 1941) are adopted as the governing equations for
 213 describing wave-induced dynamic soil response in a hydraulically isotropic porous seabed. The
 214 combined continuity and motion equation for the pore water is:

$$\nabla^2 p_p - \frac{\gamma_w n_s \beta_s}{k} \frac{\partial p_p}{\partial t} = \frac{\gamma_w}{k} \frac{\partial \varepsilon_s}{\partial t} \quad (6)$$

215 where p_p is wave-induced pore water pressure (i.e. pore water pressure in excess of the static
 216 pressure due to mean seawater level); γ_w is the unit weight of pore water; n_s is soil porosity; k is
 217 the Darcy's permeability assumed to be the same in all directions. The compressibility of pore fluid
 218 β_s and the volume strain ε_s are defined by

$$\beta_s = \frac{1}{K_w} + \frac{1 - S_r}{P_{w0}} \quad (7)$$

$$\varepsilon_s = \nabla \cdot \mathbf{v} = \frac{\partial u_s}{\partial x} + \frac{\partial v_s}{\partial y} + \frac{\partial w_s}{\partial z} \quad (8)$$

219 where K_w is the bulk modulus of pore water (adopted as 2×10^9 N/m² in Section 3.2, Yamamoto et
 220 al., 1978, and 2.3×10^9 N/m² in Section 4, Hansen, 2012); S_r is soil saturation degree; P_{w0} is
 221 absolute static water pressure; $\mathbf{v} = (u_s, v_s, w_s)$ is soil displacement vector.

222

223 The force equilibrium in a poro-elastic seabed can be calculated via following equation:

$$G\nabla^2\mathbf{v} + \frac{G}{1-2\nu}\nabla\varepsilon_s = \nabla p_p \quad (9)$$

224 where G is the shear modulus of soil and can be obtained through Young's modulus (E) and
 225 Poisson's ratio (ν):

$$G = \frac{E}{2(1+\nu)} \quad (10)$$

226 Hansen (2012) suggested that Young's modulus (E) for the soil at large depth within a seabed can be
 227 determined by

$$E = E_{ref} \left(\frac{\sigma'_3}{\sigma'_{3,ref}} \right)^\alpha \quad (11)$$

228 where E_{ref} is reference Young's modulus of soil, σ'_3 and $\sigma'_{3,ref}$ are confining pressure and
 229 reference confining pressure, respectively, α is a constant ranging from 0.5 to 0.7 for sand.

230

231 In accordance with the generalized Hooke's law, effective normal stress, σ'_i , and shear stress, τ_{ij} ,
 232 where the subscripts $i,j=x,y,z$ indicate the direction of Cartesian coordinate, can be determined by

$$\sigma'_x = 2G \left(\frac{\partial u_s}{\partial x} + \frac{\nu}{1-2\nu} \varepsilon_s \right), \quad \sigma'_y = 2G \left(\frac{\partial v_s}{\partial y} + \frac{\nu}{1-2\nu} \varepsilon_s \right) \quad (12)$$

$$\sigma'_z = 2G \left(\frac{\partial w_s}{\partial z} + \frac{\nu}{1-2\nu} \varepsilon_s \right), \quad \tau_{xy} = \tau_{yx} = G \left(\frac{\partial u_s}{\partial y} + \frac{\partial v_s}{\partial x} \right) \quad (13)$$

$$\tau_{xz} = \tau_{zx} = G \left(\frac{\partial u_s}{\partial z} + \frac{\partial w_s}{\partial x} \right), \quad \tau_{yz} = \tau_{zy} = G \left(\frac{\partial v_s}{\partial z} + \frac{\partial w_s}{\partial y} \right) \quad (14)$$

233 Several boundary conditions have to be specified at the boundary of seabed domain and the
 234 pile-seabed interface for an accurate prediction of WSSI. At seabed surface, $y=0$ (Fig. 1), the
 235 wave-induced pore water pressure, p_p , is set equal to p^* obtained from the wave model, and vertical
 236 effective normal stress and shear stresses are considered to be 0,

$$\sigma'_z = \tau_{xy} = \tau_{yz} = 0, \quad p_p = p^* \quad \text{at } y = 0 \quad (15)$$

237 At the bottom of seabed ($y = -h_s$, where h_s is soil depth, Fig. 1), an impermeable rigid boundary
 238 condition is applied, where soil displacement is zero and there is no vertical flow:

$$u_s = v_s = w_s = \frac{\partial p_p}{\partial y} = 0 \quad \text{at } y = -h_s \quad (16)$$

239 The same no flow (zeroGradient) and zero soil displacement boundary condition is applied at the
 240 lateral boundaries (Chang and Jeng, 2014):

$$u_s = v_s = w_s = \frac{\partial p_p}{\partial x} = 0 \text{ at } x = 0 \text{ and } x = L_s \quad (17)$$

$$u_s = v_s = w_s = 0, \frac{\partial p_p}{\partial z} = 0 \text{ at } z = -W_s/2 \text{ and } z = W_s/2 \quad (18)$$

241 In order to eliminate the influence of lateral boundaries, the length, L_s , and the width, W_s , of
 242 simulation domain (Fig. 1), were taken as four times the wavelength, L_w , and sixteen times the
 243 mono-pile diameter D . This domain size was used in Chen et al. (2014) to investigate wave-structure
 244 interaction. It is reported in Ye and Jeng (2012) that the soil domain length (L_s) larger than double
 245 wavelength is sufficient to eliminate the impact from fixed lateral boundaries. Thus, the mono-pile is
 246 located at the centre of computing domain and the lateral boundary of soil domain does not affect the
 247 simulated results around mono-pile foundation. Additionally, mono-pile is simulated as a rigid
 248 impermeable object so that at its surface the no-flow boundary condition applies, i.e. the gradient of
 249 pore water pressure vanishes:

$$\frac{\partial p_p}{\partial n} = 0 \quad (19)$$

250 where n denotes the normal to mono-pile surface. This boundary condition is acceptable for the rigid
 251 object located within a porous seabed (Chang and Jeng, 2014; Lin et al., 2016; Zhao et al., 2016a).

252

253 2.3 Integration process between wave and seabed model

254 Unlike the previous two-dimensional (2-D) monolithically integrated model in COMSOL
 255 Multiphysics using FEM (Lin et al., 2016), the three-dimensional (3-D) one-way integrated WSSI
 256 model is proposed in OpenFOAM with FVM. The present integrated model is able to simulate the
 257 wave-structure interaction more accurately, with low-cost of computer memory, and with high mesh
 258 density in the 3-D case. It solves the wave and soil model by two steps within one time step as
 259 illustrated in Fig. 2. First of all, in accordance with input wave parameters and the adjustable time
 260 step calculated by Courant–Friedrichs–Lewy (CFL) condition (adopted as 0.5 in this study), the
 261 wave model solves the Navier-Stokes and Volume of Fluid equations by the combined algorithm
 262 (PISO-SIMPLE, namely PIMPLE) for pressure-velocity coupling. Secondly, the dynamic wave
 263 pressure is extracted from wave model and applied to seabed surface through extended general grid

264 interpolation (GGI) (Tukovic et al., 2014), which allows the integrated model to run WSSI
265 computation in parallel within a time step compared to the serial WSSI simulation in Liu et al.
266 (2007). The soil model then computes the wave-induced dynamic seabed response by solving QS
267 Biot's equations using FVM method (Tang and Hededal, 2014). After the completion of two
268 sub-models simulations within a time step, the integrated model exports the simulated results based
269 on pre-set writing time interval and then continues to the next time step simulation until the
270 prescribed total simulation time is reached.

271

272 **3. Validation**

273 In this section, we validate both wave and seabed components of the integrated WSSI model against
274 the available published laboratory experimental results. The lateral and plan views of numerical
275 domains are shown in Fig. 1. The wave characteristics and soil properties used for validation are
276 listed in Table 1.

277

278 3.1. Wave model

279 Before applying the present WSSI to practical engineering, the ability of model to accurately
280 simulate wave nonlinearity when interacting with a mono-pile needs to be investigated. The
281 experimental data presented in Chen et al. (2014) and Zang et al. (2010) are adopted to validate
282 present wave model. Two types of regular wave, one with the wave height $H = 0.14$ m, and the wave
283 period $T = 1.22$ s, and another one with $H = 0.12$ m, $T = 1.63$ s, are used to study the nonlinear
284 wave-structure interaction. To reproduce the laboratory experiment a 3-D numerical wave tank was
285 established, as shown in Fig. 1, but without seabed sub-domain. In laboratory experiment, the
286 diameter of mono-pile, D , is 0.25 m, while mean water depth, h_w , is 0.505 m. On the basis of the
287 investigation of mesh sensitivity by Paulsen et al. (2014b), the refined mesh with a resolution of 15
288 points per wave height is adopted in the validation.

289

290 Fig. 1 also shows several wave gauges and pore water pressure sensors locations for model
291 validations and further applications in the numerical wave-seabed tank. Wave gauge 1 at 0.77 m from
292 the inlet, and Wave gauge 2 at 0.002 m distance from the upstream mono-pile surface along the

293 centreline are used to measure free surface elevation, η . Fig. 3 (a) shows the comparison of simulated
294 and experimental free surface elevation for the incident wave, i.e. at Wave gauge 1. The simulated
295 incident wave is in a good agreement with the experimental results. The time series of simulated and
296 experimental free surface level close to the mono-pile (at Wave gauge 2) for two different regular
297 waves are shown in Fig. 3(b) and 3(c). Excellent agreement between numerical and experimental
298 results denote that present wave model has the capacity to simulate the strongly nonlinear behaviour
299 of waves interacting with mono-pile, including the small jump after wave troughs.

300

301 The simulated wave-induced inline force on the surface of mono-pile, F_x , is also compared with
302 experimental results in Fig. 4. The simulated inline force is calculated by spatial integration of the
303 total pressure, p , over the surface of the mono-pile exposed to sea water (the water sub-domain in
304 Fig. 1). Despite minor discrepancy at wave nodes the agreement between computed and experimental
305 results is generally good, hence showing that the application of present wave model to practical
306 engineering is promising. The aforementioned validations show that nonlinearity of wave-pile
307 interaction is accurately predicted in the numerical wave tank in both cases. It can be concluded that
308 present wave model (waves2Foam) is capable of capturing the nonlinear wave-pile interactions,
309 including free surface elevation and wave load on the mono-pile.

310

311 3.2. Wave-seabed interaction model

312 Wave-induced dynamic seabed response was validated by comparison of simulation results with the
313 laboratory experiment of Liu et al. (2015). The laboratory experiment was carried out in a
314 one-dimensional column filled with sand saturated with water, and exposed to a periodic variation of
315 pressure at the cylinder top. The time series of the resulting variation of pore water pressures was
316 measured at several locations along the column. The soil properties used for validation are listed in
317 Table 1 and the reader is referred to Liu et al. (2015) for more details. In order to eliminate the
318 potential effect from lateral boundaries, the soil domain for validating soil model is designed as a
319 2-D case, in which the lateral and bottom boundary conditions are selected as demonstrated in
320 section 2.2, and at seabed surface, analytical wave pressure based on laboratory experiment is

321 imposed. The soil properties tabulated in Table 1 are measured in Liu et al. (2015), and then used as
322 input parameters in the validation of soil model.

323

324 Vertical distribution of wave-induced pore water pressure from the experiment shown in Liu et al.
325 (2015) is compared with numerical simulation in Fig. 5. Results are scaled with the maximum pore
326 water pressure at seabed surface, P_0 . The simulated results generally agree with the experiment and
327 the analytical result (Hsu and Jeng, 1994) except for an obvious discrepancy at the position close to
328 seabed bottom ($y/h_s=-0.8$). A possible explanation, given in Liu et al. (2015), is that the soil in the
329 physical test was not perfectly homogeneous, i.e. soil properties could have been different close to
330 the bottom, while in numerical model soil properties are constant. The time series of wave-induced
331 pore water pressure at the depth $y = -0.067$ m ($y/h_s=-0.037$) against experimental data is shown in Fig.
332 6, in which ω is wave frequency. The numerical prediction agrees well with the experimental results.
333 In conclusion we are confident that the present seabed model in OpenFOAM has the capacity to
334 accurately model the wave-induced dynamic seabed response.

335

336 4. Application

337 In reality, the foundations of offshore mono-piles are protected by granular filters in order to prevent
338 scour which may result in the failure of the offshore structures. As pointed out by Kirca (2013), the
339 seabed beneath granular filters may experience liquefaction in the seabed below. Following the
340 satisfactory validations present coupled WSSI model is further applied to investigate dynamic seabed
341 response in the proximity of mono-pile foundation due to nonlinear effect of wave-pile interaction at
342 intermediate water depth. In this example, the wave from the Danish ‘Wave loads’ project (Paulsen et
343 al., 2014b) is considered, and the wave field interacts with a mono-pile of 6 m diameter (D). The
344 mean water depth is constant, $h_w = 20$ m. The detailed wave and seabed parameters for investigation
345 of nonlinear wave-induced seabed response around mono-pile are listed in Table 2. To determine the
346 distribution of Young’s modulus (E) in seabed, $E_{ref} = 177$ MPa, $\sigma'_{3,ref} = 150$ kPa, and $\alpha = 0.62$
347 are used in accordance with the medium sand in Eskesen et al. (2010). In reality the vibration of
348 mono-pile due to the action of violent wave may compact granular soil and urge the air out, leading
349 to a denser and more saturated soil around mono-pile foundation during pile vibration. In present

350 study this phenomenon is not simulated - mono-pile is assumed to be very rigid and the seabed
351 saturation is adopted as a constant (Table 2). The focus of present study is therefore solely on
352 dynamic behaviour of porous seabed and associated potential liquefaction around mono-pile
353 foundation caused by the interaction of extreme wave and mono-pile foundation.

354
355 The initial investigation is performed for a mono-pile that is embedded into seabed until the depth
356 equal triple pile diameter. We first examine the connection between nonlinear wave and dynamic
357 seabed response due to the blockage effect of mono-pile. According to the available momentary
358 liquefaction criterion, the potential momentary liquefaction zone around mono-pile is studied in
359 detail. The final part of this study investigates the influence of the embedment depth of mono-pile
360 foundation, ranging from three to seven times pile diameter, on the dynamic seabed response to the
361 action of high steepness waves.

362

363 4.1 Vertical distribution of pore water pressure in the vicinity of mono-pile

364 The vertical distribution of pore water pressure around pile is recorded at a series of vertical profiles
365 located 0.05 m away from the surface of mono-pile with θ ranging from 0° to 180° with 45°
366 increment (wave gauges 2-6 in Fig. 1), and at position 7 located in the centre of mono-pile. The
367 corresponding vertical profiles of pore water pressures are shown in Fig. 7 with t/T varying from
368 5.04 to 6.07, i.e. over one period. In general, the vertical distribution of pore water pressure has the
369 greatest amplitudes at front face of mono-pile foundation, $\theta = 0^\circ$, and the smallest amplitudes at $\theta =$
370 90° . Between $\theta=0^\circ$ and $\theta=90^\circ$, the overall pore water pressures along embedment depth reduce,
371 while beneath the pile there is only a slight decrease. For θ between 90° and 180° , the trend reverses,
372 resulting in peak pressures at $\theta =180^\circ$. The reason for these trends may be a consequence of free
373 surface elevation variation together with the variation of wave pressure around mono-pile. The
374 comparison and analysis of relationship between wave-pile interaction and pore water pressure
375 distribution are elaborated in next section.

376

377 As shown by Zhang et al. (2015), the presence of mono-pile in seabed increases the pore water
378 pressure along mono-pile foundation compared to that without mono-pile foundation penetrated into

379 seabed. Fig. 7(a)-(e) shows that the magnitude of pore water pressure declines rapidly from the
380 seabed surface to approximately $y = -1.8$ m, and then slightly decreases until the depth of about $y =$
381 -17.46 m, close to the pile bottom. Between $y = -17.46$ m and $y = -19$ m, an evident fall of pore water
382 pressure magnitude can be noticed. The explanation of this is that the soil below pile bottom may be
383 shielded from the pore water pressure induced by propagating wave above. Fig. 7 (f) presents the
384 pore water pressure along the central line of mono-pile bottom. In comparison with the pore water
385 pressure around mono-pile circumference at $y = -18$ m, the pore water pressure underneath pile
386 bottom is relatively small and has limited variation. The limited impact of the wave pressure on the
387 dynamic soil response under pile bottom at different θ -locations also confirms the shielding effect of
388 pile foundation.

389

390 4.2 Wave-induced seabed response around mono-pile

391 The wave model validation has shown (Fig. 3) that high steepness wave has an evident nonlinearity
392 when interacting with mono-pile. Wave crest and wave trough, as well as pore water pressure,
393 develop nonlinearly due to interaction with mono-pile, compared to the case without mono-pile. This
394 is primarily due to the blockage effect of mono-pile in the wave and pore water pressure propagating
395 direction.

396

397 In order to further examine the notable variation of pressure at several vertical locations, $y = 0$ m,
398 -1.8 m, -17.46 m, and -18 m, the time histories of pore water pressure at these locations, as well as
399 the time history of free surface elevation are presented in Fig. 8, at the same locations 0.05 m away
400 from mono-pile surface (wave gauges 2-6 in Fig. 1). The t/T from 4 to 7, when the interaction of
401 wave and mono-pile has attained the dynamic equilibrium, is considered. It can be noticed that the
402 interaction between wave and mono-pile produces strong nonlinearity of free surface elevation, even
403 wave-breaking at WG4 and WG5. This in turn affects pore water pressure, which shows similar
404 albeit development history. By comparing free surface elevation at various wave gauges, it is implied
405 that the maximum free surface elevation declines gradually with θ increasing from 0° to 135° and, at
406 WG6 ($\theta = 180^\circ$), the maximum free surface elevation raises due to the merge of incident wave crest
407 propagated separately from both lateral sides of pile (Swan and Sheikh, 2015). Pore water pressure

408 presents similar decrease when θ grows from 0° to 90° , but different development at $\theta = 135^\circ$. It can
409 be inferred that, when the free surface elevation is changing rapidly, the water pressure at the seabed,
410 and hence also pore water pressure within the bed, do not respond simultaneously. The precise
411 simulation of wave pressures around the pile is therefore required in order to accurately model the
412 dynamic seabed response.

413

414 The second column of Fig. 8, shows that, while pore water pressure at $y = -1.8$ m still shows similar
415 development history as that at seabed surface, the effect of wave-pile interaction on pore water
416 pressure becomes weaker as the observation point moves from -1.8 m to -18 m. The comparison of
417 maximum pore water pressure at different θ in Fig. 8 shows once more that the pore water pressure at
418 $\theta = 90^\circ$ reaches its minimum.

419

420 4.3 Wave-induced liquefaction around pile

421 Liquefaction around offshore structures is considered as one of the primary threats to operational
422 lifetime of these structures (Sumer, 2014), so it is a major concern in the engineering practice. Based
423 on the liquefaction criterion suggested in Jeng (2013) and Sumer (2014), the potential liquefaction
424 zone can be determined by

$$-(\gamma_s - \gamma_w)y \leq (p_{ps} - P_b) \quad (20)$$

425 where γ_s and γ_w are the unit weight of seabed and water, respectively ($\gamma_s = 1.9 \gamma_w$ was used in
426 this study); P_b is the pore water pressure on the seabed surface; p_{ps} is the pore water pressure
427 within porous seabed. Liquefaction may occur in a porous seabed when the net excessive pore water
428 pressure, equals to the difference between the pressure at seabed surface and pressure at a point
429 beneath the surface, surpasses overburden soil pressure and soil matrix begins to lose its capacity for
430 undertaking any load.

431

432 Using the aforementioned liquefaction criterion, maximum liquefied depth was evaluated and its
433 time series is shown in Fig. 9, along with free surface elevation and inline force. Comparison
434 between Fig. 9 (a) and (c) shows that the momentary liquefaction close to mono-pile surface takes
435 place periodically at the moment when free surface elevation at WG2 is smaller than 0 and inline

436 force has its minimum (see Fig. 9). As a consequence of the propagation of wave trough, liquefied
437 depth reaches its maximum. Maximum liquefaction depth drops and disappears due to the arrival of
438 wave crest and rapid increase of free surface elevation and excess pressure on seabed surface from
439 negative to positive, which in turn leads to decrease of the difference of pore water pressure at
440 seabed surface and within seabed, which can be observed at $t/T = 5.33$ to 5.92 in Fig. 7.

441

442 Comparison of Fig. 9 (b) and (c) in the case with KC number being 8.85 and D/L being 0.032 , shows
443 that during the potential liquefaction period, very close to maximum depth, there is also negative
444 inline force directed upstream ($F_x < 0$). As a result of this, the liquefied soil in the closest vicinity of
445 mono-pile loses its support and then may enlarge mono-pile vibration, which is induced by periodic
446 inline force. As mentioned earlier this periodic vibration of mono-pile foundation may pressurize
447 adjacent soil in the vibration direction, and force the air out. As a consequence this process tend to
448 harden surrounding soil and alter soil properties. For pile-seabed interaction, the reader is referred to
449 Hansen (2012) for more details. To avoid the threat from potential liquefaction around foundation,
450 Chang and Jeng (2014) suggested that momentary liquefaction may be prevented by replacing the
451 existing soil layer with coarse sand layer with greater permeability.

452

453 Further presentation of the extent of liquefaction potential is shown in Fig. 10 at $t/T = 5.66$, when
454 liquefaction depth is the largest (highlighted by black hollow circle in Fig. 9 (c). As shown in Fig. 10
455 (a) and (b), momentary liquefaction potential arises broadly while wave trough is approaching
456 mono-pile over porous seabed. Compared with the liquefaction zone without mono-pile in the far
457 field, liquefaction at front and back face of mono-pile foundation are relatively smaller. Fig. 10 (c)
458 shows the liquefaction depth at the interface between soil and foundation with θ ranging from 0° to
459 180° . The liquefaction depth is about 1 m at the front face of pile foundation; it gradually increases
460 as the observation point moves around the pile perimeter to reach maximum of approximately 1.5 m
461 at $\theta = 90^\circ$, and then slightly reduces as the point moves from $\theta = 90^\circ$ and $\theta = 180^\circ$. The temporal
462 evolution of the liquefaction depth at several θ -locations along the pile perimeter are presented in Fig.
463 10 (d). The liquefaction first appears at front face of mono-pile foundation and then rapidly
464 approaches its lateral side ($\theta = 90^\circ$), where the maximum liquefaction depth occurs. Between the

465 lateral side and the back face there is further slight delay and slight decrease of the maximum
466 liquefaction depth.

467

468 Momentary liquefaction in porous seabed propagates along with the wave trough above seabed. For
469 the purpose of investigating possible threat from momentary liquefaction to scour protection,
470 maximum potential liquefaction depth in the vicinity of mono-pile foundation over a wave period
471 (t/T from 5 to 6) is presented in Fig. 11. It can be observed that maximum liquefaction depth of
472 around 1.5m is located in the lateral zone near mono-pile foundation, with θ approximately ranging
473 from 60° to 110° , while minimum potential liquefaction depth of approximately 1 m occurs at front
474 and back side of mono-pile foundation, where θ equals 0° and 180° , respectively. It can be inferred
475 that for $KC = 8.85$ and $D/L = 0.032$ the scour protection may experience greater liquefaction threat,
476 which may cause it to sink, in the areas close to lateral sides of mono-pile foundation than in the
477 areas close to the front and back side.

478

479 4.4 Influence of embedded depth

480 In reality, the ratios of embedment depth for mono-pile foundation of offshore wind turbine and
481 mono-pile diameter often vary from 4 to 8 at shallow/intermediate water depth (Lesny et al., 2007).
482 Therefore, for the same wave conditions listed in Table 2, the present model is further applied to the
483 examples with two additional embedment depths, namely 30 m and 42 m (Table 2), in order to
484 investigate the effects of embedment depth on the development of pore water pressure and potential
485 liquefaction.

486

487 Figures 12 and 13 show the development of vertical distribution of pore water pressure for the
488 embedment depth of 30 m and 42 m respectively. For both cases the development of pore water
489 pressure along embedment depth, as well as along pile bottom are similar to those already shown in
490 Fig. 7 (section 4.1), for the main case with the embedment depth of 18m. The development of the
491 vertical pressure profiles around the pile perimeter is also similar for the three cases: pore water
492 pressure declines as θ grows from 0° to 90° and then raises with θ ranging from 90° to 180° .
493 However, the magnitude of pore water pressure along the foundation reduces as the embedment

494 depth grows.

495

496 The estimated liquefaction depths in the aforementioned examples with 3 various penetration depths
497 are shown in Fig. 14. At the front face of mono-pile foundation, the embedment depth has minor
498 effect on liquefaction depth. The effect gradually increases as θ grows from approximately 30° to
499 180° : increasing embedment depth results in smaller liquefaction depth. It can be inferred that
500 increasing embedment depth has blocking effect on the pore water pressure propagation from front
501 face to back face of mono-pile foundation. As a result, the pore water pressure along the mono-pile
502 foundation with greater embedment depth presents slower reduction compared to that with smaller
503 embedment depth, which eventually decreases the difference of pore water pressure along the
504 embedment depth and leads to smaller liquefaction depth as shown in Fig. 14.

505

506 **5. Conclusions**

507 The numerical investigation of nonlinear wave-induced dynamic seabed response in the proximity of
508 mono-pile foundation has been performed in detail using one-way coupled solver in OpenFOAM. In
509 order to accurately describe the nonlinear wave interaction with mono-pile waves2Foam (Jacobsen et
510 al., 2012) is applied for the numerical simulation of flow field. In soil model, the quasi-static Biot
511 equations, solved by Finite Volume Method (Liu et al., 2007; Tang et al., 2015), govern the dynamic
512 response of porous seabed around mono-pile foundation. A coupled scheme, based on extended
513 general grid interpolation (GGI) (Tukovic et al., 2014) which allows the integrated model to run in
514 parallel, is used to integrate both sub-models. The comparisons with available laboratory
515 experimental results in the literature show excellent agreement for both wave and soil model. It
516 demonstrates that this integrated WSSI model is capable of estimating nonlinear wave-induced
517 mechanical behaviour of poro-elastic seabed around offshore mono-pile-supported structure.

518

519 The benefits of the present model compared to those so far presented in the literature are: (1)
520 nonlinear interaction of wave and mono-pile, including free surface elevation and inline force, is
521 predicted accurately; (2) the resulting wave-induced dynamic seabed behaviour near mono-pile
522 foundation is simulated simultaneously; (3) the associated momentary liquefaction potential in the

523 vicinity of mono-pile foundation can also be estimated based on available liquefaction criteria. The
524 model at present does not incorporate poro-elasto-plastic soil model, nor the interaction between
525 mono-pile foundation and seabed. These two mechanisms, which may result in different impacts on
526 seabed response, also play vital roles in the assessment of offshore foundation stability and will be
527 integrated into the future model.

528

529 The following conclusions are drawn from the present study:

530 (1) The wave-induced pore water pressure is weakened as soil depth increases. The presence of
531 mono-pile foundation leads to the noticeably different distribution of pore water pressure in the
532 vicinity of foundation. The vertical distribution of pore water pressure around mono-pile
533 foundation varies significantly with θ : within a wave period, the range of pore water pressure
534 reduces substantially between $\theta = 0^\circ$ and $\theta = 90^\circ$, and then gradually increases as θ grows from
535 90° to 180° . The range of pore water pressure at $\theta = 90^\circ$ is the largest due to wave diffraction
536 around mono-pile.

537

538 (2) Since pore water pressure within the seabed are attenuated compared to the pressures at seabed
539 surface, the pressure difference between them generates an upward force resulting in the
540 momentary liquefaction around mono-pile foundation. Application of a momentary liquefaction
541 criterion shows that the horizontal distribution of liquefaction potential around mono-pile
542 foundation (i.e. its variation with θ) is influenced by wave-pile interaction. Under the action of
543 unidirectional regular waves with $KC = 8.85$ and $D/L = 0.032$, the maximum and minimum
544 liquefaction depth take place at approximately $\theta = 90^\circ$ and $\theta = 180^\circ$, respectively. In a wave period,
545 maximum liquefaction depth occurs at the positions with θ varying from 60° to 110° , where the
546 scour protection may experience greater sinking compared to that at front and back sides of
547 mono-pile foundation. However, since only one wave condition is taken into consideration, more
548 investigations regarding various wave conditions are suggested to fully understand potential
549 liquefaction around mono-pile foundation.

550

551 (3) Increasing embedment depth of mono-pile foundation significantly reduces the magnitude of pore
552 water pressure along the embedded foundation, whereas the overall shape of the vertical pressure
553 profiles remains similar. The increased blockage effect of larger embedment depths slightly
554 reduces the difference of pore water pressure between the seabed and its surface, and hence also
555 the corresponding liquefaction depth in the vicinity of the embedded mono-pile foundation.

556

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563

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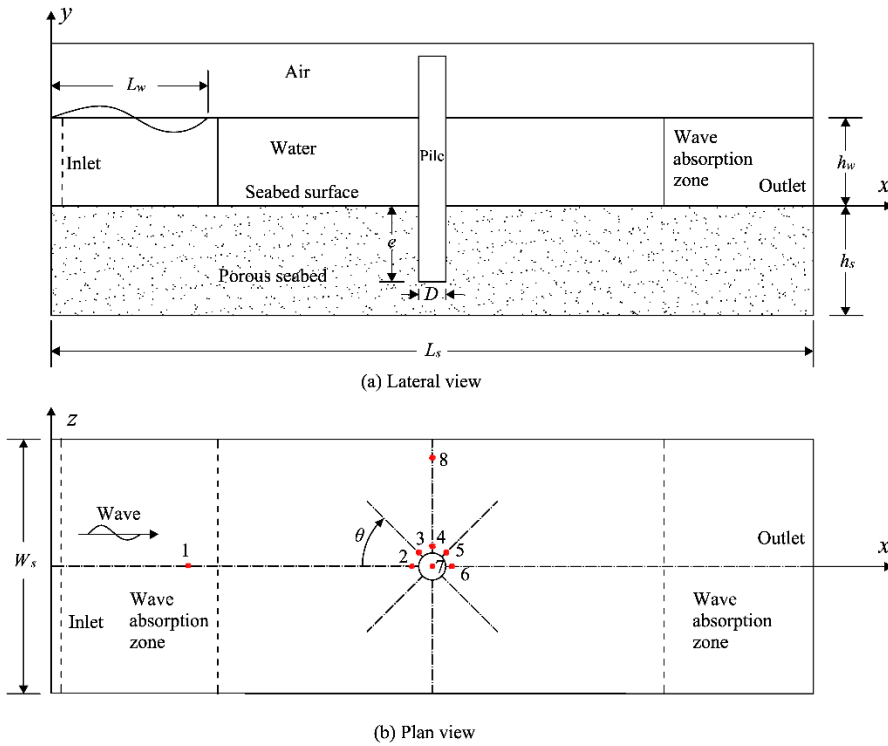
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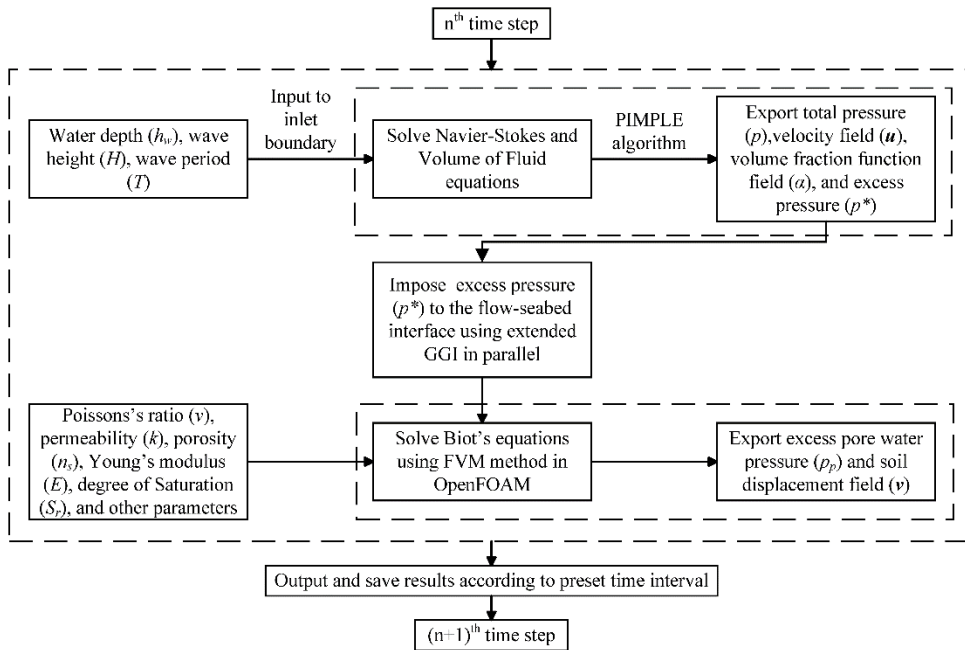


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722 Fig. 1 Sketch of the numerical wave tank. (a) Lateral view, (b) Plane view; the red dots in plan view

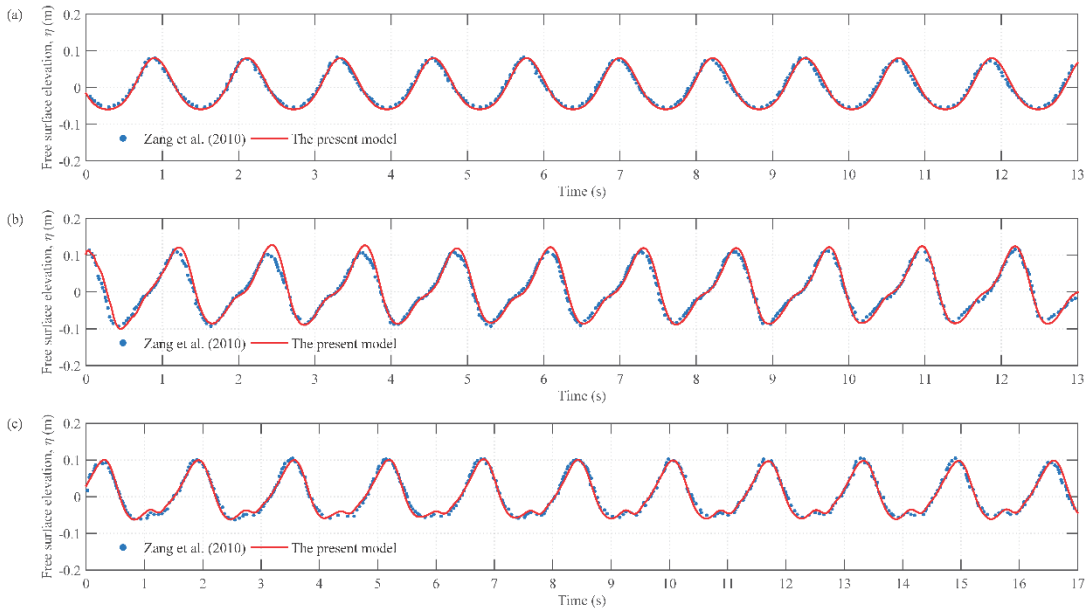
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are the locations of wave gauges or pressure sensors.

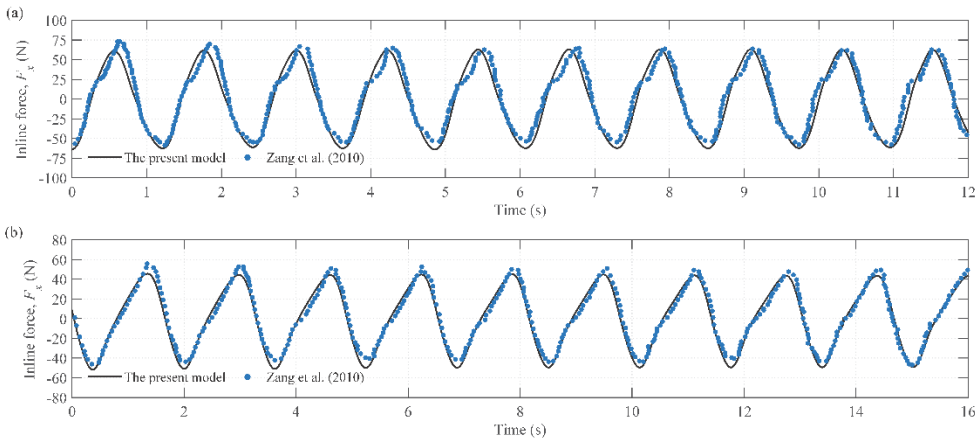


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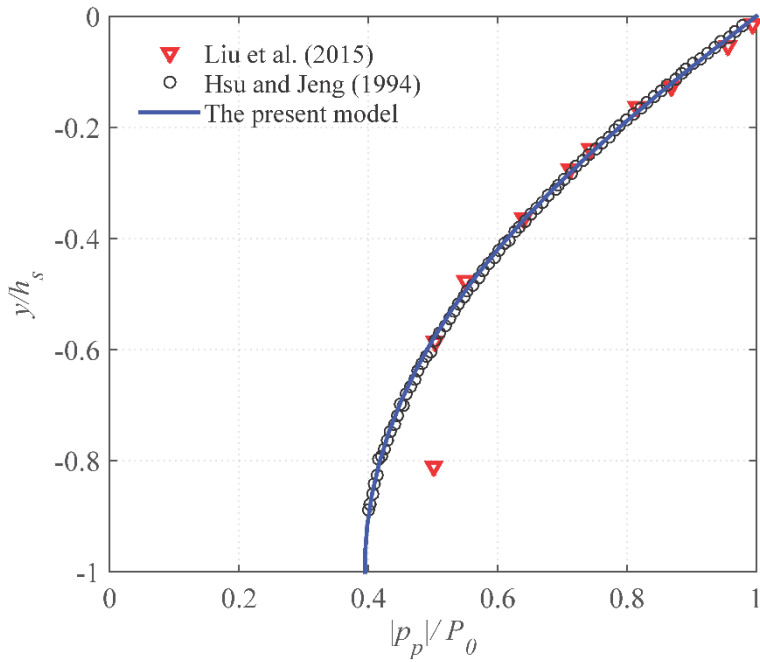
725 Fig. 2 Coupled processes in the integrated WSSI model in OpenFOAM



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 727 Fig. 1 Validation of free surface elevation (η) against experimental data (Zang et al., 2010). (a) Wave
 728 gauge 1 when $H = 0.14$ m and $T = 1.22$ s, (b) Wave gauge 2 when $H = 0.14$ m and $T = 1.22$ s, (c)
 729 Wave gauge 2 when $H = 0.12$ m and $T = 1.63$ s.
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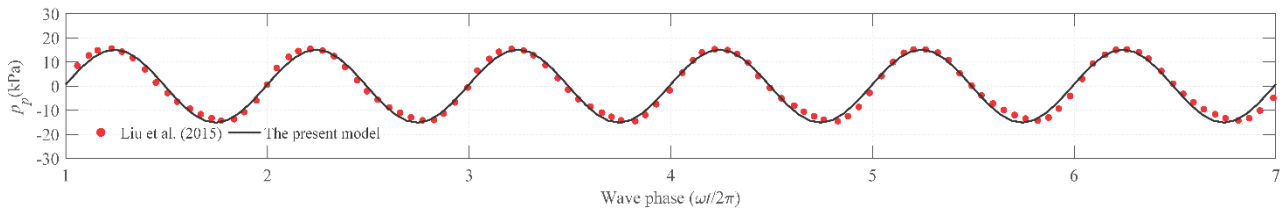
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 732 Fig. 2 Comparison of inline force (F_x) in OpenFOAM and experimental results (Zang et al., 2010). (a)
 733 $H = 0.14$ m and $T = 1.22$ s, (b) $H = 0.12$ m and $T = 1.63$ s.



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735 Fig. 3 Comparison of vertical distribution of maximum pore water pressure between laboratory
 736 experiments from Liu et al. (2015) for $S_r = 0.996$ and numerical reproduction in OpenFOAM.

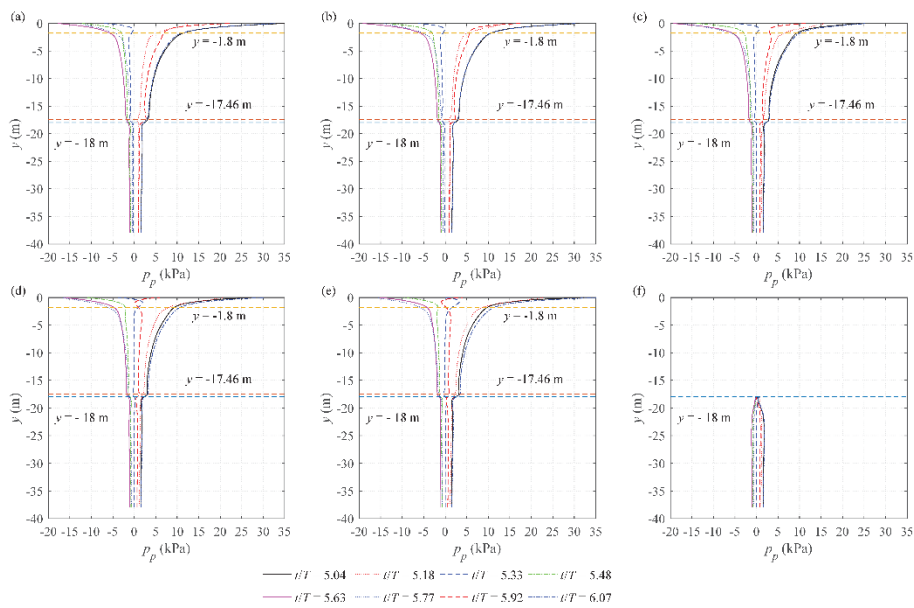
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739 Fig. 4 Comparison of wave-induced pore water pressure p_p between the experimental data for $S_r =$
 740 0.951 and numerical results in OpenFOAM at the depth $y = -0.067$ m ($y/h_s = -0.037$).

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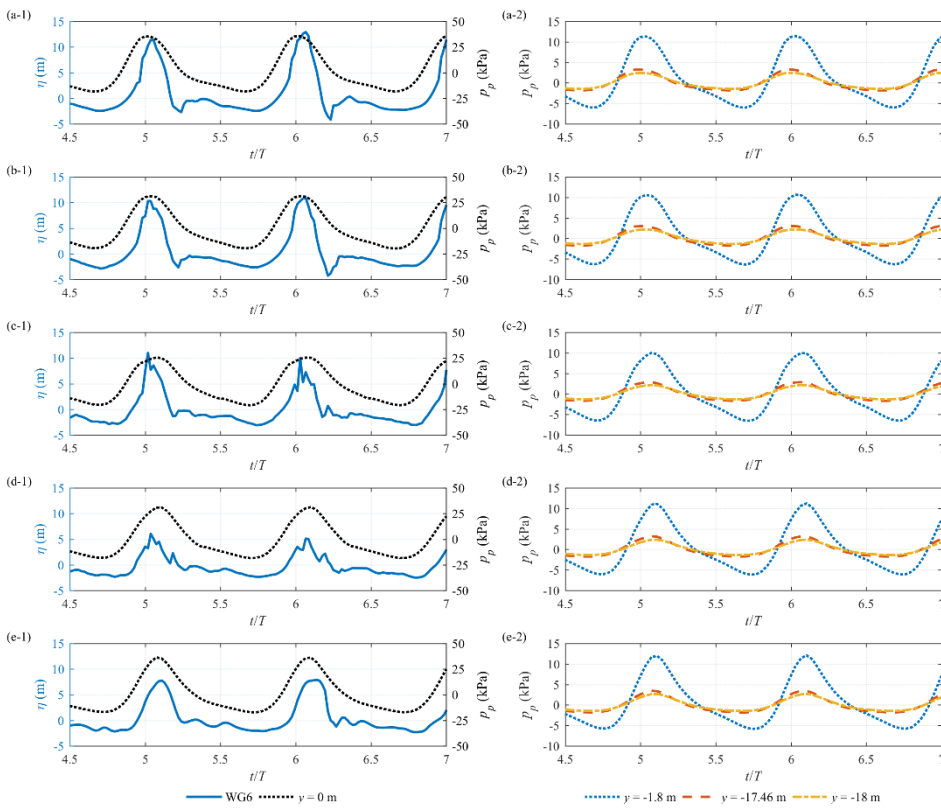


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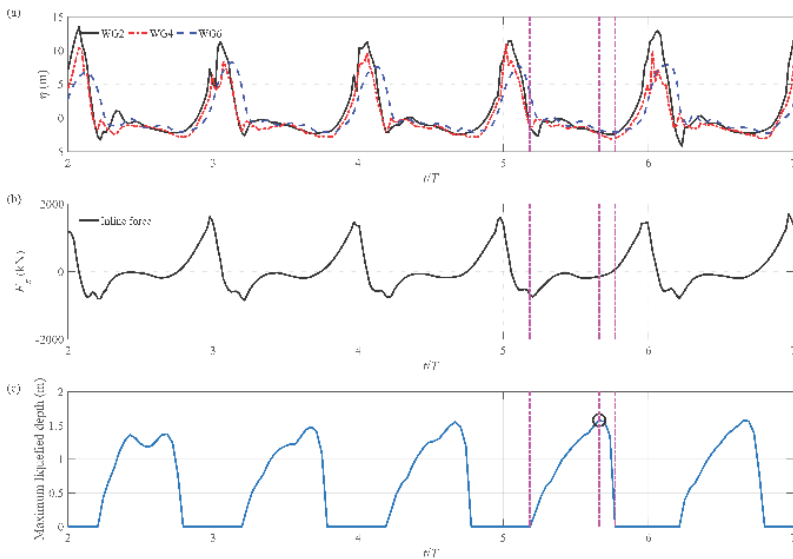
744 Fig. 5 Vertical distribution of pore water pressure at various positions. (a) $\theta = 0^\circ$, (b) $\theta = 45^\circ$, (c) $\theta =$

745 90°, (d) $\theta = 135^\circ$, (e) $\theta = 180^\circ$, (f) Centre of mono-pile bottom.
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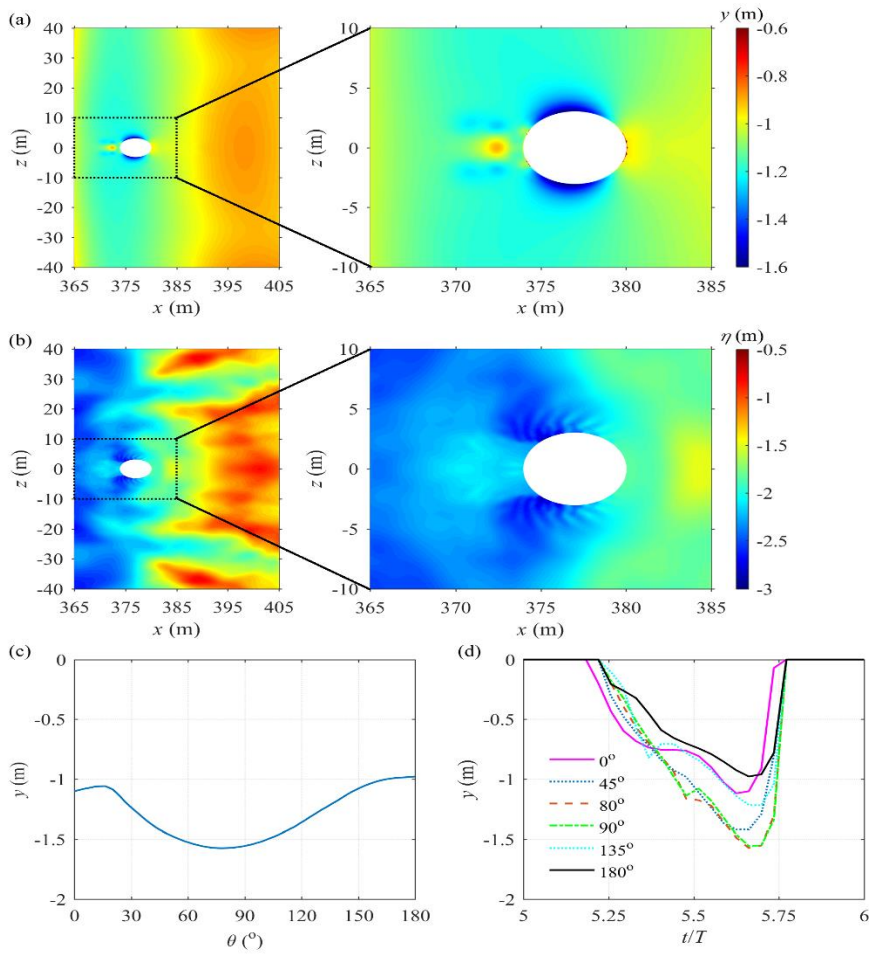


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 748 Fig. 6 Time series of free surface elevation (η) at various wave gauges. (a) $\theta = 0^\circ$, (b) $\theta = 45^\circ$, (c) $\theta =$
 749 90° , (d) $\theta = 135^\circ$, (e) $\theta = 180^\circ$. The first column are the comparisons of wave gauges and pore water
 750 pressure at $y = 0$ m. The second column are the comparisons of pore water pressure at $y = -3$ m, $y =$
 751 -17.46 m, and $y = -18$ m, respectively.

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 756 Fig. 7 Time series of (a) free surface elevation (η), (b) inline force, (c) maximum liquefied depth,
 757 with $KC = 8.85$ and $D/L = 0.032$.



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759 Fig. 8 Liquefaction depth (y) and free surface elevation (η) around mono-pile foundation at $t/T = 5.66$.

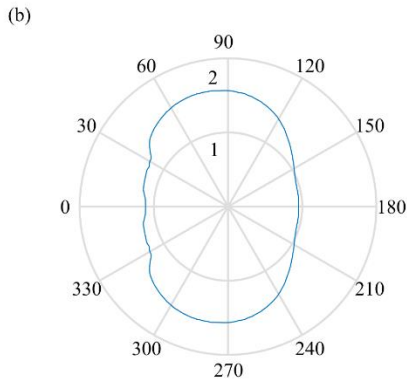
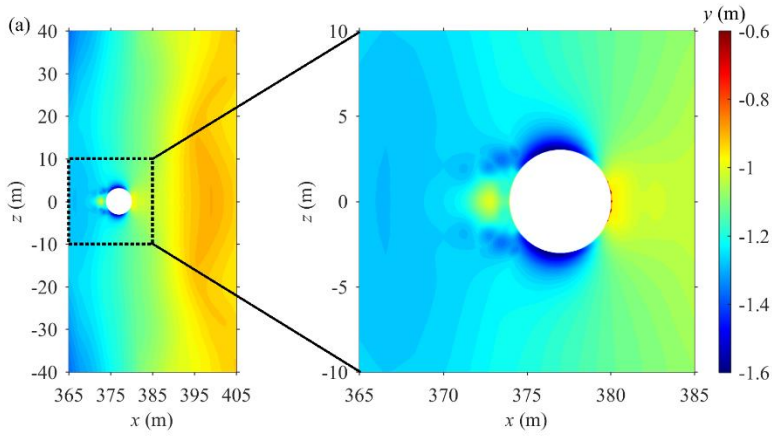
760 (a) Contour plot of liquefied depth, (b) Contour plot of free surface elevation (η), (c) Liquefied depth

761 for various θ -locations at the soil-pile interface, (d) Time series of liquefied depth at various

762 θ -locations on the soil-pile interface.

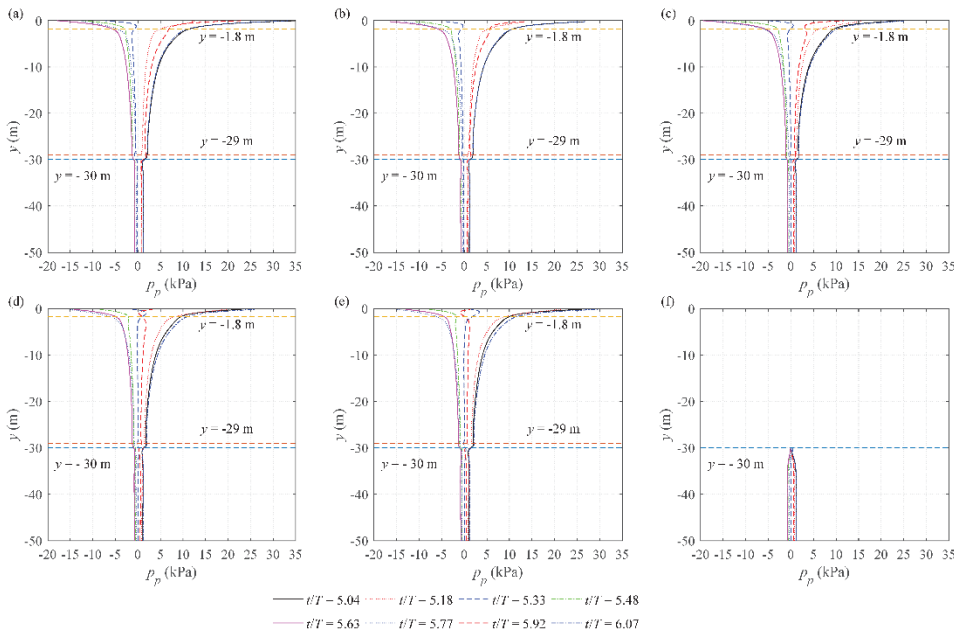
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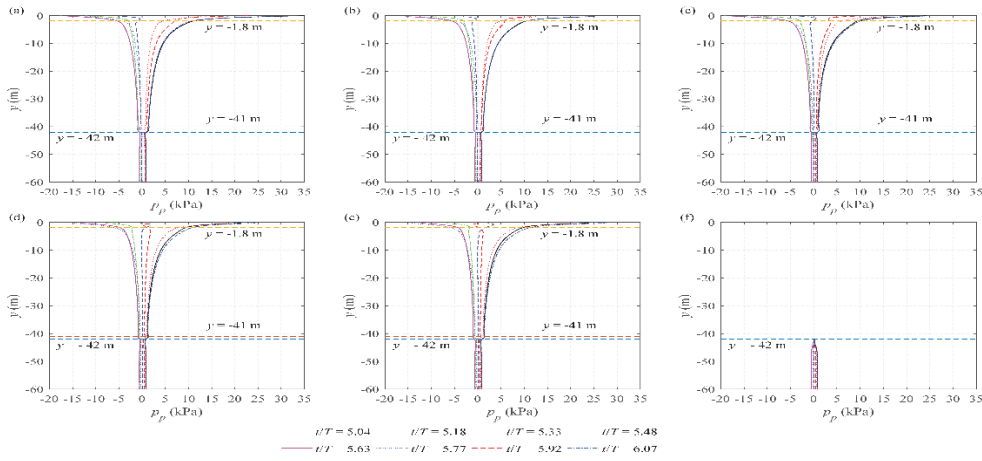
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Fig. 9 Maximum potential liquefaction depth over a wave period (t/T from 5 to 6). (a) Horizontal distribution, (b) Maximum liquefaction depth varying with θ at the distance of 0.05m away from pile surface.

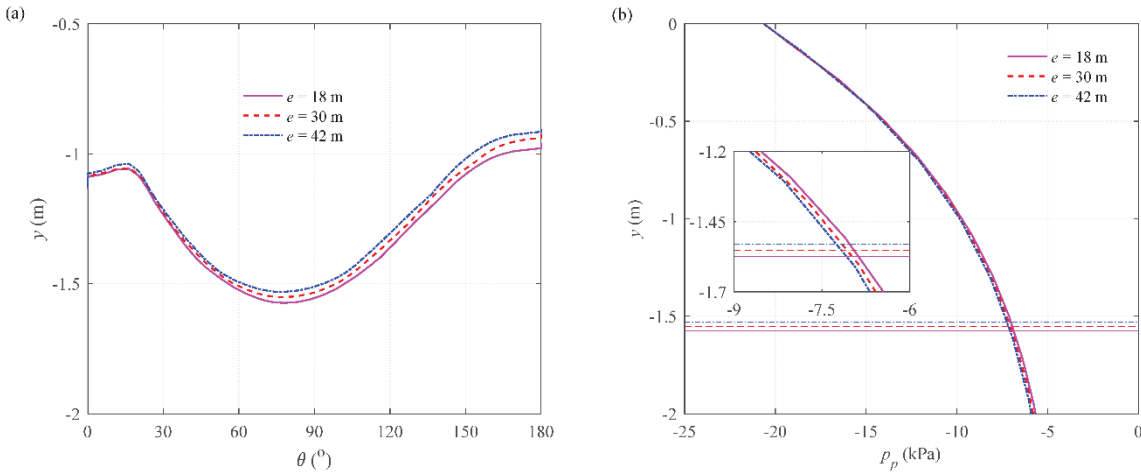


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Fig. 10 Vertical distribution of pore water pressure at various positions when embedment depth $e = 30$ m. (a) $\theta = 0^\circ$, (b) $\theta = 45^\circ$, (c) $\theta = 90^\circ$, (d) $\theta = 135^\circ$, (e) $\theta = 180^\circ$, (f) Centre of mono-pile bottom.



776
 777 Fig. 11 Vertical distribution of pore water pressure at various positions when embedment depth $e =$
 778 42 m. (a) $\theta = 0^\circ$, (b) $\theta = 45^\circ$, (c) $\theta = 90^\circ$, (d) $\theta = 135^\circ$, (e) $\theta = 180^\circ$, (f) Centre of mono-pile bottom.
 779



780
 781 Fig. 12 Comparison of liquefied depth with various embedment depths at $t/T = 5.66$. (a) Spatial
 782 description of liquefied depth varying with θ on the soil-pile interface, (b) Liquefaction depth at
 783 $\theta = 90^\circ$, horizontal lines are maximum liquefaction depth.
 784

785 Table 1 Wave characteristics and soil properties for WSSI model validation

Experiments	H (m)	T (s)	h_w (m)	D (m)	e (m)	G (N/m ²)	ν	k (m/s)	n_s	S_r	h_s (m)
Zang et al. (2010)	0.14	1.22	0.505	0.25	0	0	0	0	0	0	0
	0.12	1.63	0.505	0.25	0	0	0	0	0	0	0
Liu et al. (2015)	3.5	9	5.2	0	0	1.27×10^7	0.3	1.8×10^{-4}	0.425	0.996	1.8
	3.5	9	5.2	0	0	1.27×10^7	0.3	1.8×10^{-4}	0.425	0.951	1.8

Table 2 Parameters for studying wave-seabed-pile interaction

Wave characteristics			
Wave height, H (m)	8.43	Wave period, T (s)	13.6
Water depth, h_w (m)	20	Wave length, e (m)	188.5
KC number	8.85		
Seabed characteristics			
Seabed thickness, h_s (m)	38, 50, 62	Poisson's ratio, ν	0.2
Submerged specific weight of sediment (kN/m ³)	9.5	Permeability, k (m/s)	1×10^{-4}
Degree of saturation, S_r	0.98	Soil porosity, n_s	0.38
Young's modulus	See section 4		
Mono-pile characteristics			
Diameter, D (m)	6	Embedment depth, e (m)	18, 30, 42
D/L	0.0032		