Soil response around Donghai offshore wind turbine foundation, China

1 Kuntan Chang BEng

Postgraduate student, Center for Marine Geotechnical Engineering, Department of Civil Engineering, State Key Laboratory of Ocean Engineering, Shanghai Jiao Tong University, Shanghai, China

2 Dongsheng Jeng MEng, PhD

Professor, Center for Marine Geotechnical Engineering, Department of Civil Engineering, State Key Laboratory of Ocean Engineering, Shanghai Jiao Tong University, Shanghai, China; Griffith School of Engineering, Griffith University Gold Coast Campus, Queensland, Australia; State

Key Laboratory of Hydrology – Water Resources and Hydraulic Engineering, Hohai University, Nanjing, Jiangsu, China

3 Jisheng Zhang MSc, PhD

Professor, State Key Laboratory of Hydrology – Water Resources and Hydraulic Engineering, Hohai University, Nanjing, Jiangsu, China

4 Yu Zhang BEng

Postgraduate student, Center for Marine Geotechnical Engineering, Department of Civil Engineering, State Key Laboratory of Ocean Engineering, Shanghai Jiao Tong University, Shanghai, China

Donghai offshore wind farm next to Shanghai is the first and largest commercial operating offshore wind energy system in China that adopts a high-rising structure foundation. This paper consists of two parts. Details are presented of this wind farm project, site conditions and related engineering solutions in the first part. Then, in the second part, a three-dimensional porous model, based on Reynolds-averaged Navier–Stokes equations and Biot’s poro-elastic theory, is developed by integrating three-dimensional wave and seabed models to simulate the wave-induced seabed response around the high-rising structure foundation. A parametric study of the effects of the wave and seabed characteristics on the soil response around the wind turbine foundation is conducted. Results concluded from the numerical analysis are as follows: (a) the existence of the structure has a significant effect on the wave transformation and the distribution of wave-induced pore pressure; (b) the magnitude of wave-induced pore pressure increases as wave height or wave period increases. In addition to the current design of the Donghai offshore wind farm with high-rising structure foundation, a gravity-based foundation is also considered and results are compared with those for the high-rising structure foundation.

Notation

A	area of source region	P_{w0}	absolute water pressure
C	wave phase velocity	p_b	hydrostatic pressure in seabed
E_s	Young’s modulus	p_s	wave-induced pore pressure in seabed
F	function to represent fractional volume of fluid water	$\langle p_f \rangle$	pressure of fluid
G	shear modulus	S	seabed degree of saturation
g_i	gravitational acceleration	T	wave period
H	wave height	t	time
h	seabed thickness	u_s, v_s, w_s	soil displacements
K_w	true modulus of elasticity of water	$\langle u_{fi} \rangle$	ensemble mean velocity
k	turbulence kinetic energy (TKE)	x_i	Cartesian coordinate
k_s	soil permeability	z	soil depth
L	wavelength	β	compressibility of pore fluid
n_s	soil porosity	γ_s	unit weight of soil
		γ_w	unit weight of pore water
		δ_{ij}	Kronecker delta

ε_s	volume strain of soil matrix
μ	dynamic viscosity
μ_{eff}	total effective viscosity
μ_s	Poisson ratio
μ_t	turbulent viscosity
ρ_f	density of fluid
σ_{vi}	initial stress due to dead loading
$\sigma'_x, \sigma'_y, \sigma'_z$	effective normal stress
$\tau_{xz}, \tau_{xy}, \tau_{yz}$	shear stress
Ω	source region
ω	wave frequency
∇	Laplace operator

1. Introduction

Developments of sustainable energy, especially wind energy, have been rapidly advancing in recent decades. The total wind energy capacity in China had reached 44.73 GW by 2011. However, wind farms in China are onshore, except for the Donghai offshore wind farm (DH-OWF), and 89% of the wind power capacity is located in Northern China. In fact, the southeast coastal area has rich offshore wind resources, which could provide a good supplement for some coastal cities like Shanghai. According to the offshore development plan drawn up by Chinese coastal provinces, which identified potential projects for inter-tidal (at water depths less than 5 m) and offshore (at water depths 5–50 m) developments, Shanghai, Jiangsu, Zhejiang, Shandong and Fujian all hope to have a combined offshore cumulative installed capacity of some 10.1 GW by 2015 and 30 GW by 2020 (Rajgor, 2010).

Moving wind energy offshore makes it possible to use a larger capacity turbine, thus leading to a higher hub and larger rotor, and therefore a larger overturning moment imposed on the foundation. Moreover, the offshore environment is more complex than that onshore. Therefore, it is important to select a suitable foundation style for a specific offshore wind turbine. The gravity-based foundation (GBF) was adopted in the first offshore wind farm in the world, which was installed off the Danish coast in 1991. Mono-piles have been the most popular wind turbine foundation, accounting for over 75% of existing turbine foundations, in shallow water, owing to its competitive low costs of fabrication and installation (Doherty and Gavin, 2012). A novel offshore wind turbine foundation that is being tested at Fredrikshavn, Denmark, is the suction caisson. With increasing water depth, multiple footing options are used to provide greater moment resistance. Recently a floating platform with tension legs was investigated for deep water. Breton and Moe (2009) described the current status of the development of offshore wind parks in Europe and North America, as well as the option of foundations.

Apart from the aforementioned types of wind turbine foundation, coastal and structural engineers have to provide new options for particular seabed characteristics and marine environments; for instance, a high-rising structure foundation (HRSF) has been used in the DH-OWF.

Marine structures are exposed to marine environmental loading such as wave loadings that are cyclic in nature. Research on responses of offshore foundations to cyclic loads has been conducted over the past few decades, and a series of two-dimensional (2D) or three-dimensional (3D) models have been developed to obtain a better understanding regarding the interaction between soil and pile foundation (Randolph, 1981; Yang and Jeremić, 2002). However, pore water pressures have been left out in the preceding works and the soil was considered as material without voids. In addition, engineers tend to increase pile stiffness or soil ultimate bearing capacity to avoid foundation failure. As a matter of fact, pore water pressure that accumulates in the wave action process may lead to a decrease of effective stress and the eventual collapse of upper structure. Many coastal structures have been damaged by seabed instability in the vicinity of the structure, rather than by design or construction deficiencies (Lundgren *et al.*, 1989). Thus, research on wave-induced pore pressure response around marine structures is meaningful.

Recently, several studies of seabed response in the vicinity of mono-pile foundations have been reported in the literature (Hansen, 2012; Li *et al.*, 2011). It should be pointed out that there is a difference between mono-pile and multi-pile foundations in terms of dynamic behaviour as well as in the way they transfer the applied loads to the surrounding soil (Cuellar, 2011). Therefore, the behaviour of the HRSF – a type of multi-pile foundation, subjected to cyclic load – deserves investigation. Unlike previous works, the present study focuses on the stability of the foundation installed in the DH-OWF and an advanced model is proposed to investigate seabed response under wave loads.

In this paper, both wave and seabed sub-modes, governed by Reynolds-averaged Navier–Stokes (RANS) equations and Biot's consolidation theory (Biot, 1941) respectively, were integrated into one model to investigate the mechanism of seabed response around the HRSF. The Navier–Stokes equation was solved with internal wave generation, as developed for 2D cases by Lin and Liu (1999). A finite-element method (FEM) model, developed within Comsol Multiphysics, was used to solve the governing equations of the seabed model. Parametric studies were carried out to examine the influence of wave and seabed characteristics on the soil response around the foundation. Furthermore, the seabed liquefaction potential was estimated, then the seabed responses for HRSF and GBF were compared.

2. Donghai offshore wind farm

2.1 General introduction for Donghai offshore wind farm

Shanghai, situated in the western Pacific Ocean, is located at the middle portion of the Chinese coast and at the estuary of the Yangtze River. Affected by the East Asian monsoon, the Shanghai region has rich wind resources, the main wind directions being a southeast wind in summer and a northwest or northeast wind in winter. Wind data collected by an anemometer tower installed on the test pile of Donghai Bridge show that at the height of the wind turbine's hub (90 m), the annual average wind speed reaches 8.6 m/s, with average effective wind energy 661.7 W/m² and accumulated time of annual effective wind power 8248 h (3.5~25 m/s) (Lu, 2010).

The offshore wind farm project of Shanghai's Donghai Bridge is the first commercial offshore wind farm in China. It is located along a line 1 km away from either side of Donghai Bridge, with a 1000 t auxiliary channel going through it, as shown in Figure 1 (Ling and Cai, 2012). Its southernmost end is 13 km from the coastline and the northernmost end is around 6 km from the coastline. The first phase of DH-OWF, which is

composed of 34 Sinovel (3 MW) turbines, began its construction in April 2008 and became operational in July 2010. The project's total electricity output has been up to 4950 GWh by August 2012 (Liu, 2012). Total project investment is £25.19 million with total installed capacity 102 MW and the designed annual power generation utilisation hours reach 2624 h, with an expected annual electricity output of 267 GWh. This will meet the annual demand of over 200 000 families (Chen, 2011).

According to the environmental impact statement (see <http://www.envir.gov.cn>) released by Shanghai Investigation, Design and Research Institute, the second phase of DH-OWF will be located on the west side of the Donghai Bridge with 27 turbines (3.6 MW). In addition, 3.6 MW and 5 MW prototype turbines were installed in 2011 to accumulate experience for further development. Its total installed capacity will be up to 102.2 MW when the project is completed in 2014.

2.2 Characteristics of DH-OWF and its engineering solutions

Donghai offshore wind farm is different from many offshore wind farms elsewhere in the world because of the following characteristics.

- (a) The surface soil of the seabed is silt or a silt quality soil layer that is up to 20 m thick. Compared with that of wind farms in Europe which is mainly 10 m of fine sand soil layer (Carter, 2007), the geological situation is quite unfavourable. It was reported that the 4 m diameter mono-pile foundation can be adopted elsewhere in the world, whereas a 7 m diameter mono-pile foundation is needed in China (Wang *et al.*, 2009).
- (b) It is not uncommon for typhoons to hit southern and eastern provinces of China. At least three typhoons have reached landfall directly in the Shanghai region since 1949. Typhoon Haikui slammed Shanghai in August 2012 and monitoring data acquired from the DH-OWF site show that the maximum instantaneous wind speed reached over 40 m/s (Liu, 2012). Therefore, the typhoon risk must be taken into consideration when developing any offshore wind farm project in China.
- (c) As illustrated in Figure 1, an auxiliary channel lies in the middle of the turbines, and the minimum distance between turbines and channel is only 30 m. This means that collision with ships represents a potential danger for turbines in DH-OWF. Therefore, it is critical to protect these turbines against such ship impacts.
- (d) Although China has built a few pilot demonstration offshore wind farm projects, the key technologies related to turbine manufacture, construction, fabrication and installation are not yet mature. For example, owing to the

Figure 1. Location of DH-OWF

difficulty of the construction technique in China and the seabed characteristics in the region, the DH-OWF used HRSF, rather than one of the more common offshore wind energy structures used in Europe.

The features of the DH-OWF project described above are severely challenging to the Chinese offshore wind farm industry. Designers and engineers have come up with a series of solutions to reduce the impacts of these disadvantages.

- (a) A novel foundation style termed the HRSF (as defined earlier) is proposed, which consists of eight pile-foundations and a mini-platform in the first phase design. The finished structure and sketch model of the HRSF can be seen in Figures 2 and 3 respectively, and more detail will be given in the next section. It is evident that only a mono-pile with a considerably large diameter can be driven in silt soil steadily. The silt soil layer and immature pile driving technique were the main determining reasons why the HRSF was designed to consist of eight piles, each with a diameter of 1.7 m, instead of using a mono-pile with larger diameter.
- (b) To protect wind turbines from collision with ships, devices such as steel pipe should be installed around the wind turbine. However, this method may be expensive if all of the turbines are designed with this protection. In the DH-OWF project, engineers designed a reinforced concrete platform at water level that combined the supporting structure with the anti-collision device, as illustrated in Figure 2. This design not only allows the platform to dissipate collision energy, but also to transfer energy to the pile foundation and seabed.
- (c) Technical issues have also put constraints on turbine installation. The method adopted at DH-OWF was to

assemble the wind turbine on an assembly base, as shown in Figure 1, in advance; then the whole assembled wind turbine is transported and lifted into position, which is a reliable but costly means of installation.

- (d) The potential impact of a typhoon has been taken into consideration when designing wind turbines in DH-OWF. The ‘typhoon-resistant’ wind turbine will feather its blades (reduce their angle to the wind) and stop when average wind speed exceeds 25 m/s or instantaneous wind speed exceeds 36 m/s.

2.3 Foundation of Donghai offshore wind farm

In the first phase of DH-OWF project, the HRSF consists of eight pile-foundations and a mini-platform, as shown in Figure 3. The mini-platform of the HRSF is made up of a 1.5 m high frustum and a 3 m high cylinder at the bottom. The base diameter and cut diameter of the frustum are 14 m and 11 m respectively. Eight 80-m-long piles embedded in the seabed at circa 65 m are arranged equally spaced around a circle of radius 5 m. The steel piles are driven into the seabed with a 5:5:1 slope and connected to the upper platform by pouring concrete. For the second phase of the DH-OWF project, the design of the HRSF was further adapted with different dimensions and number of piles, as presented in Table 1. Since this design is being trialled for the first time in the world, no research information on foundation stability of the whole wind turbine system could be obtained in the literature.

3. Theoretical models

In this study, the phenomenon of wave–seabed interactions in the vicinity of an HRSF, as depicted in Figure 3, is considered. The origin of the Cartesian coordinate is located at the intersection of the centreline of the HRSF and the seabed surface, labelled A. A wave-maker is situated two times the wavelength away from the structure and each end of the computational domain adopts a sponge layer. The foundation structure is modelled from the HRSF of 3 MW turbines with the same scale. Dimensions of the foundation are given in Table 1. In this section, the governing equations for both wave and seabed models will be outlined first, followed by the boundary conditions and numerical scheme.

3.1 Wave model

Based on conservation of mass and momentum, the incompressible fluid motion due to a wave can be described by the RANS equations. The following governing equations are expressed in the Einstein summation convention.

$$1. \quad \frac{\partial \langle u_{fi} \rangle}{\partial x_i} = 0$$

Figure 2. The HRSF in DH-OWF (revised from Lu (2010))

Figure 3. Definition of wave–seabed interactions in the vicinity of HRSE

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

where x_i is the Cartesian coordinate, ρ_f is the density of fluid, $\langle u_{fi} \rangle$ is the ensemble mean velocity, $\langle p_f \rangle$ is the pressure of fluid, μ is the dynamic viscosity and g_i is the gravitational acceleration. The Reynold stress term $\rho_f \langle u'_{fi} u'_{fj} \rangle$ is modelled by the k – ε turbulence model (Rodi, 1993).

By applying the eddy-viscosity assumption, the Reynolds stress term can be estimated by

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

where μ_t is the turbulent viscosity, k is the turbulence kinetic energy (TKE) and δ_{ij} is the Kronecker delta.

Based on Equation 3, Equation 2 can be rewritten as

$$4. \quad \frac{\partial \rho_f \langle u_{fi} \rangle}{\partial t} + \frac{\partial \rho_f \langle u_{fi} \rangle \langle u_{fj} \rangle}{\partial x_j} = - \frac{\partial \langle p_f + \frac{2}{3} \rho_f k \rangle}{\partial x_i} + \frac{\partial}{\partial x_j} \left[\mu_{\text{eff}} \left(\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right) \right] + \rho_f g_i$$

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

The internal wave-maker method developed by Lin and Liu (1999) is used in the present study. In the source region (Ω), a source function $S(x_i, t)$ is added into the mass conservation, Equation 1, that is

$$5. \quad \frac{\partial \langle u_i \rangle}{\partial x_i} = S(x_i, t) \quad \text{in } \Omega$$

where $S(x_i, t) \neq 0$ within the source region. The value of $S(x_i, t)$ depends on wave characteristics, for example, for a linear monochromatic wave, it is

	Turbine: MW	Pile		Platform	
		Number	Diameter: m	Diameter: m	Height: m
First phase	3.0	8	1.7	14	4.5
Second phase	3.6	6	2.0	14	4.5
	5.0	10	1.7	18	4.5

Table 1. Dimension parameters of HRSE

$$6. \quad S(x,t) = \frac{CH}{A} \sin(\omega t)$$

in which C is the wave phase velocity, H is wave height, A is the area of the source region and ω is the wave frequency.

3.2 Seabed model

In this study, we consider an isotropic, homogeneous seabed with the same permeability (k_s) in all directions. The conservation of mass leads to

$$7. \quad \nabla^2 p_s - \frac{\gamma_w n_s \beta_s}{k_s} \frac{\partial p_s}{\partial t} = - \frac{\gamma_w}{k_s} \frac{\partial \varepsilon_s}{\partial t}$$

where ∇ is the Laplace operator; γ_w is the unit weight of pore water; n_s is soil porosity; and the volume strain of soil matrix (ε_s) and the compressibility of pore fluid (β) are defined as

$$8. \quad \varepsilon_s = \frac{\partial u_s}{\partial x} + \frac{\partial v_s}{\partial y} + \frac{\partial w_s}{\partial z}$$

$$9. \quad \beta_s = \frac{1}{K_w} + \frac{1-S}{P_{w0}}$$

in which (u_s , v_s , w_s) are soil displacements, K_w is the true modulus of elasticity of water (taken as 2×10^9 N/m²), P_{w0} is the absolute water pressure and S is the degree of saturation.

$$10a. \quad \frac{\partial \sigma'_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + \frac{\partial \tau_{xz}}{\partial z} = - \frac{\partial p_s}{\partial x}$$

$$10b. \quad \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma'_y}{\partial y} + \frac{\partial \tau_{yz}}{\partial z} = - \frac{\partial p_s}{\partial y}$$

$$10c. \quad \frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma'_z}{\partial z} = - \frac{\partial p_s}{\partial z}$$

in which ($\sigma'_x, \sigma'_y, \sigma'_z$) are the effective normal stress; and ($\tau_{xz}, \tau_{xy}, \tau_{yz}$) are shear stress.

In this study, a poro-elastic constitutional model is used to describe the soil behavior in the seabed. Then, effective normal stresses and shear stresses can be expressed in terms of soil displacements as

$$11a. \quad \sigma'_x = 2G \left[\frac{\partial u_s}{\partial x} + \frac{\mu \varepsilon_s}{1-2\mu} \right], \sigma'_y = 2G \left[\frac{\partial v_s}{\partial y} + \frac{\mu \varepsilon_s}{1-2\mu} \right]$$

$$11b. \quad \sigma'_z = 2G \left[\frac{\partial w_s}{\partial z} + \frac{\mu \varepsilon_s}{1-2\mu} \right], \tau_{xz} = G \left[\frac{\partial u_s}{\partial z} + \frac{\partial w_s}{\partial x} \right]$$

$$11c. \quad \tau_{xy} = G \left[\frac{\partial u_s}{\partial y} + \frac{\partial v_s}{\partial x} \right], \tau_{yz} = G \left[\frac{\partial v_s}{\partial z} + \frac{\partial w_s}{\partial y} \right]$$

where the shear modulus G is related to Young's modulus E_s and the Poisson ratio μ_s in the form of $E_s/2(1+\mu_s)$. It is noted that a positive sign is taken for a compressive normal stress in this study.

Substituting Equation 11 into Equation 10, the conservation of momentum in the x -, y - and z -directions, respectively, becomes

$$12a. \quad G \nabla^2 u_s + \frac{G}{(1-2\mu_s)} \frac{\partial \varepsilon_s}{\partial x} = - \frac{\partial p_s}{\partial x}$$

$$12b. \quad G \nabla^2 v_s + \frac{G}{(1-2\mu_s)} \frac{\partial \varepsilon_s}{\partial y} = - \frac{\partial p_s}{\partial y}$$

$$12c. \quad G \nabla^2 w_s + \frac{G}{(1-2\mu_s)} \frac{\partial \varepsilon_s}{\partial z} = - \frac{\partial p_s}{\partial z}$$

3.3 Boundary conditions

To solve the RANS governing equations, appropriate boundary conditions are required. A sponge layer damping method is applied for vertical velocity component at both domain ends to avoid reflection of waves and interference on current inlet/outlet. At the air–water interface, zero surface tension with $\partial k / \partial \vec{n} = \partial \varepsilon / \partial \vec{n} = 0$ is imposed. At the solid surface, a no-slip boundary is given and the turbulence properties are estimated from the standard wall function. To obtain computational stability, the time interval (Δt) is automatically adjusted at each time step to satisfy the Courant–Friedrichs–Lewy condition and the diffusive limit condition (Lin and Liu, 1999).

The evaluation of the wave-induced seabed response also requires appropriate boundary conditions. At the seabed surface, the pore pressures (p_s) and shear stresses ($\tau_{xz}, \tau_{xy}, \tau_{yz}$) equal the wave pressures and shear stresses obtained from the RANS model (wave mode). The bottom of the porous seabed with finite thickness (h) is treated as impermeable and rigid; therefore, zero displacement and no vertical flow occur in this bottom boundary ($u_s = v_s = w_s = \partial p_s / \partial z = 0$). Zero displacement and gradient of pore pressure are adopted in the four vertical sides far away from the concerned region,

$u_s = v_s = w_s = \partial p_s / \partial x = 0$. The interface between piles and seabed is considered as impermeable and adopts a Neumann boundary condition.

3.4 Numerical scheme

The RANS governing equations are solved by a two-step projection method with finite-volume discretisation (Bussmann *et al.*, 2002). The computational domain is divided into a set of structured/unstructured meshes, in which all the fluid properties, such as pressure and velocities, are defined at the cell centroids. The least-square linear reconstruction (LSLR) method (Barth, 1992) is adopted to convert cell centroid data to face centroids to evaluate the quantity gradient at cell centroid. The time derivative is discretised by the forward time difference method. The volume of fluid (VOF) method developed by Hirt and Nichols (1981) is used to track the free water surface. The idea of VOF is to define a function F to represent the fractional volume of fluid water. $F=1$ indicates that the cell is full of water, whereas $F=0$ corresponds to a cell fully occupied by air cells with value of $0 < F < 1$ containing a water free surface, and the piecewise linear interface calculation (PLIC) method (Rider and Kothe, 1998) is used to reconstruct the air–water interface.

To solve the Biot's consolidation theory outlined previously, a numerical code was developed within Comsol Multiphysics environments. Comsol is a commercial FEM tool that allows the users to design their own governing equations and boundary conditions for specific physical problems. In this study, based on the existing Poro-WSSI II (porous models for wave–seabed–structure interactions, Version II) developed by Jeng's research group (Jeng, 2013), a new Comsol model was developed for 3D porous flow. The outputs of the 3D wave model are integrated into the newly developed Comsol model by the boundary conditions at the interface between water and seabed.

4. Results and discussion

The aim of this study is to investigate the wave-induced pore pressure around the HRSF. The parameters used in this study are as close as possible to the situation in the DH-OWF. In this section, a parametric study is carried out to examine the effects of wave and seabed characteristics on soil response around the foundation. The input data for the parametric study are provided in Table 2. All the numerical results given here are from the fifth wave period at which a fully dynamic interaction between wave and foundation structure on porous seabed is achieved.

4.1 Wave-induced pore pressure around HRSF

In this section, the wave-induced pore pressure around the HRSF at different times will be examined. Wave parameters and soil characteristics are considered as follows: wave period

Parameter	Value	Unit
Wave period, T	10, 7.76, 5	s
Wave height, H	2.0, 1.5, 1.0	m
Water depth, d	13.0	m
Seabed thickness, h	80	m
Permeability, k_s	10^{-4}	m/s
Degree of saturation, S	0.99, 0.98, 0.97	—
Soil porosity, n_s	0.3	—
Poisson ratio, μ_s	0.4	—
Shear modulus, G	10^7	N/m ²

Table 2. Input data of numerical examples

$T=5$ s, wave height $H=2$ m, soil permeability and degree of saturation are 10^{-4} m/s and 0.98, respectively. Figure 4 shows the water surface and wave-induced pore pressure distribution at 1 m below the seabed surface during a full period ($t/T = 5\frac{3}{8}$ to $6\frac{1}{4}$), which is divided into eight roughly equal parts. It can be seen in the figure that pore pressure is positive on the spots of the wave crest and negative on the wave trough.

The wave-induced pore pressures at different time steps are illustrated in Figure 4. The existence of a structure has significant effects on the distribution of pore pressures, especially in the direction perpendicular to wave propagation and an increase of pore pressure around the foundation is observed. In detail, at the sides of the structure, maximum (or minimum) pore pressure is distributed in regions which are away from the centreline of wave propagation; by contrast, maximum (or minimum) pore pressure is concentrated in the HRSF at the inside of the structure. This is because the existence of the structure partly blocks the cross-sectional space through which the wave flow passes, leading to a larger wave height in front of the structure. After passing the foundation, the wave decays clearly as part of the incident wave energy has been dissipated owing to the wave–structure interaction, as shown in the profile of wave transformation in Figure 4.

Figure 5 illustrates the vertical distributions of the wave-induced pore pressure at point A (referring to Figure 3) during the same full period. As presented in the figure, the maximum magnitude of pore pressure in the wave crest is roughly equal to the absolute value of minimum magnitude in the trough. Moreover, the profiles of pore pressure in the seabed are similar between wave crests and wave troughs. On the one hand, positive wave pressures are imposed on the seabed surface from $t/T = 5\frac{3}{8}$ to $5\frac{3}{4}$, and the pore pressure attenuates very rapidly within 1.4 m, where a minimum value is reached. Then, a slight increase in magnitude with depth is observed,

Figure 4. Distributions of wave profile and the wave-induced pore pressure around HRSF 1 m below the seabed surface at different times: $t/T =$ (a) $5 \frac{3}{8}$; (b) $5 \frac{1}{2}$; (c) $5 \frac{5}{8}$; (d) $5 \frac{3}{4}$; (e) $5 \frac{7}{8}$; (f) 6; (g) $6 \frac{1}{8}$; (h) $6 \frac{1}{4}$

and finally decreases slightly with depth again. On the other hand, the pore pressures in the seabed are negative from $t/T = 5 \frac{7}{8}$ to $6 \frac{1}{4}$, and the distribution of pore pressure is approximately symmetrical with the previous half period.

4.2 Effects of wave and soil characteristics

In this section, the effects of several wave and soil characteristics on the wave-induced pore pressure around the foundation are further investigated. The wave height (H), wave period (T) and seabed degree of saturation (S) that are determined to play key roles in the response are used in the parametric study.

The other wave and soil parameters used in numerical examples are presented in Table 2.

Figure 6 describes the distribution of wave-induced pore pressure at point A with three different wave heights. The wave period used here is 7.76 s, which is the average period of the DH-OWT. The incident wave with larger wave height accompanies more energy and therefore results in larger wave pressure on the seabed. Results in Figure 6 confirm that the magnitude of the wave-induced pore pressure increases as wave height increases.

Figure 5. Vertical distributions of the wave-induced pore pressure (p_s) at point A from $t/T=5\frac{3}{8}$ to $6\frac{1}{4}$

Wave periods affect the wavelength together with water depth directly. In this example, three different wave periods, $T = 5$ s, 7.76 s and 10 s are considered, and the corresponding wavelengths are $L = 37.5$ m, 75.3 m and 104.5 m respectively. Figure 7 presents the effects of wave period on the wave-induced pore pressure with wave height $H=2$ m. As represented in Figure 7, a longer wave period will cause a large magnitude of pore pressure in the seabed. A similar phenomenon can be seen in Figures 6 and 7: that is, the

Figure 6. Vertical distributions of the wave-induced pore pressure (p_s) plotted against soil depth (z) for various wave heights at point A

Figure 7. Vertical distributions of the wave-induced pore pressure (p_s) plotted against soil depth (z) for various wave periods at point A

profiles of pore pressure plotted against depth are approximately parallel curves, although different magnitudes of wave pressure are imposed on the seabed.

It is common to observe air/gas in marine sediments (Okusa, 1985). Hsu and Jeng (1994) reported that wave-induced soil response is affected significantly by the degree of saturation. Here, the wave-induced pore pressure for various degrees of saturation is further investigated. In the example, the degree of saturation is considered as $S = 0.99$, 0.98 and 0.97 . Figure 8 illustrates that pore pressure increases in the seabed as the degree of saturation increases; in other words, the dissipation rate of pore pressure increases as the degree of saturation decreases.

4.3 Wave-induced liquefaction around HRSF

Based on the proposed model, the wave-induced oscillatory soil response including pore pressures and effective stresses around the HRSF can be determined. Then, the wave-induced liquefaction around the HRSF can be further examined. An accurate prediction of the liquefaction zone will provide engineers with the evidence to improve the ground and optimise the design of a foundation or structure. Based on the liquefaction criterion proposed by Zen and Yamazaki (1990) and Jeng (1997, 2012), the liquefaction potential can be estimated by

$$13. \quad \sigma_{vi} + (p_b - p_s) \leq 0$$

where σ_{vi} is the initial stress due to dead loading (static water pressure and weight of the HRSF), which can be determined by

Figure 8. Vertical distributions of the wave-induced pore pressure ($|p_s|$) plotted against soil depth (z) for various degrees of saturation at point A

means of the poro-elastic model before the wave loading is applied. In Equation 13, p_b represents the hydrostatic pressure and p_s denotes the wave-induced pore pressure within the seabed.

Figure 9 illustrates the liquefied zone along the centre of the HRSF according to this criterion. As shown in the figure, the seabed near the piled foundation has the potential to be liquefied and the maximum liquefaction depth is up to 1 m with wave parameters measured from normal marine conditions. Then liquefaction depth will increase under an extreme offshore environment, which may further cause instability of the whole system.

4.4 Comparison of seabed response to a GBF

In this section, the wave-induced seabed response in the vicinity of a GBF is investigated, based on the present numerical model. The dimensions of the GBF adopted here are designed according to the optimum solution proposed by Henderson and Zaaijer (2002). The GBF diameter and height are 24 m and 2.4 m respectively. As shown in Figure 10, the vertical distributions of pore pressure for the two foundations are quite different with identical wave and seabed characteristics. Differences exist in the wave field around GBF and HRSF because of their different shape. Furthermore, the different connection format between the structure and the seabed affects pore water accumulation and dissipation in the seabed. In practical work, engineers should pay attention to the different mechanics of wave–seabed–structure interactions and choose the optimised foundation style adapted to the local marine environment.

Figure 9. Wave-induced liquefaction around HRSF

Figure 10. Comparison of vertical distributions of the wave-induced pore pressure (p_s/p_0) plotted against soil depth (z) between HRSF and GBF

5. Conclusions

In this paper, a detailed description of the DH-OWF project is provided. Then, a 3D wave model with internal wave generation is presented, and a new seabed model developed within Comsol environments. These models are linked together to investigate the wave-induced pore pressure around the HRSF. Based on the numerical results presented, the following conclusions can be drawn.

- The existence of structure has a significant effect on the wave transformation and the distribution of wave-induced pore pressure. The magnitude of wave-induced pore pressure varies along the wave propagation periodically.
- Wave and soil parameters affect the wave-induced pore pressure response significantly. The magnitude of wave-induced pore pressure increases as wave height or wave period increases and the dissipation rate of pore pressure increases as the degree of saturation decreased.
- The liquefaction potential around the HRSF is estimated, based on the pore pressure distribution in the seabed, and transient liquefaction occurred due to wave propagation, which may further cause instability of the marine structure.
- The wave-induced seabed responses of the HRSF and GBF are different, even when the related wave and soil parameters are identical.

In this paper, because no laboratory experiments and field measurements for HRSFs at the DH-OWF are available in the

literature, it is not possible to directly validate the present model for the HRSF. However, the numerical model presented in this paper was validated using previous experimental data for other types of marine structures (Jeng *et al.*, 2013; Zhang *et al.*, 2012a, 2012b). Data from physical modelling of such a structure are needed for future study.

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