The University of Bradford Institutional Repository

<http://bradscholars.brad.ac.uk>

This work is made available online in accordance with publisher policies. Please refer to the repository record for this item and our Policy Document available from the repository home page for further information.

To see the final version of this work please visit the publisher's website. Available access to the published online version may require a subscription.

Link to Publisher's version: <http://dx.doi.org/10.1016/j.coastaleng.2015.11.003>

Citation: Lin Z, Guo Y, Jeng D, Liao C and Rey N (2016) An integrated numerical model for wave-soil-pipeline interactions. *Coastal Engineering*, 108: 25–35.

Copyright statement: © 2016 Elsevier. Reproduced in accordance with the publisher's self-archiving policy. This manuscript version is made available under the [CC-BY-NC-ND 4.0 license](#).

Manuscript Number: CENG-D-15-00164R1

Title: An integrated numerical model for wave-soil-pipeline interactions

Article Type: Research Paper

Keywords: Finite element method; dynamic seabed response; pipeline;
transient liquefaction

Corresponding Author: Dr Yakun Guo, Ph.D

Corresponding Author's Institution: University of Bradford

First Author: Zaibin Lin

Order of Authors: Zaibin Lin; Yakun Guo, Ph.D; Dongsheng Jeng, PhD;
chencong liao; Nick Rey

Abstract: An integrated Finite Element Method (FEM) model is proposed to investigate the dynamic seabed response for several specific pipeline layouts and to simulate the pipeline stability under waves loading. In the present model, the Reynolds-Averaged Navier-Stokes (RANS) equations are used to describe the wave motion in a fluid domain, while the seabed domain is described using the Biot's poro-elastic theory. The interface between water and air is tracked by conservative Level Set method (LSM). The FEM and backward differentiation formula (BDF) are applied for spatial and temporal discretization respectively in the present model. One-way coupling is used to integrate flow and seabed models. The present model is firstly validated using several available laboratory experiments. It is then further extended to practical engineering applications, including the dynamic seabed response for the pipeline mounted on a flat seabed or inside a trench. The results show that the pipeline buried to a certain depth is better protected than that under partially buried in terms of transient liquefaction.

An integrated numerical model for wave-soil-pipeline interactions

Zaibin Lin¹, Yakun Guo², Dong-sheng Jeng³, Chencong Liao⁴, Nick Rey⁵

1. PhD student, School of Engineering, University of Aberdeen, AB24 3UE, UK
2. Professor, School of Engineering, University of Bradford, BD7 1DP, UK, corresponding author: y.guo16@bradford.ac.uk
3. Professor, Griffith School of Engineering, Griffith University Gold Coast Campus, Queensland 4222, Australia
4. PhD student, Shanghai Jiaotong University, Shanghai, China
5. Principal Engineer, Wood Group Kenny, Aberdeen, UK

Abstract: An integrated Finite Element Method (FEM) model is proposed to investigate the dynamic seabed response for several specific pipeline layouts and to simulate the pipeline stability under waves loading. In the present model, the Reynolds-Averaged Navier-Stokes (RANS) equations are used to describe the wave motion in a fluid domain, while the seabed domain is described using the Biot's poro-elastic theory. The interface between water and air is tracked by conservative Level Set method (LSM). The FEM and backward differentiation formula (BDF) are applied for spatial and temporal discretization respectively in the present model. One-way coupling is used to integrate flow and seabed models. The present model is firstly validated using several available laboratory experiments. It is then further extended to practical engineering applications, including the dynamic seabed response for the pipeline mounted on a flat seabed or inside a trench. The results show that the pipeline buried to a certain depth is better protected than that under partially buried in terms of transient liquefaction.

Key words: Finite element method; dynamic seabed response; pipeline; transient liquefaction

1. Introduction

Knowledge and understanding of wave-induced soil response is of particular importance to the design of offshore pipeline installation and has attracted considerable attentions among coastal and geotechnical engineers. The motivation of this increasing study is partly due to the damage of marine structures caused by wave-induced instability (Sumer and Fredsøe, 2002; Sumer, 2014) instead of

construction or material breakdown. When the wave trough is propagating over the submarine pipeline laid on a porous seabed, the excess pore pressure increases and the effective stress within seabed soil decreases accordingly. As a result of this marine process, the seabed soil under the action of wave troughs will be transiently liquefied, exposing the submarine pipeline to the risky liquefaction zone (Sumer, 2014). Therefore, accurate evaluation of the wave-induced mechanical soil response is particularly important for the protection design of submarine pipeline.

Due to its practical importance and potential engineering applications, extensive studies on the wave-induced seabed response have been carried out since the 1970s. Sleath (1970) proposed an analytical solution, with assumptions of a rigid and permeable sandy seabed and incompressible pore fluid, to compare with his laboratory experiments on wave-induced soil response in two different beds of sand. In his experiments, phase lags between the pore water pressures and dynamic wave pressures were observed, but the analytical solution cannot predict the phase lags (Sleath, 1970). Madsen (1974) analysed the stability of a sand bed induced by the effect of breaking waves. The Biot's model (Biot, 1941) has been used to develop a close-form analytical solution for the wave-induced soil response (Yamamoto et al., 1978; Madsen, 1978). Mei and Foda (1981) then examined wave-induced soil response through a boundary-layer approximation for both the coarse and fine sand, based on the continuum theory. Hsu and Jeng (1994) proposed a closed-form analytical solution for the wave-induced soil response in a finite thickness seabed and concluded that their solution under two-dimensional (2D) progressive wave had a good agreement with the Yamamoto (1977)'s semi-analytical solution. According to the derivation of governing equations by Mei (1989), Yuh and Ishida (1998) directly solved the boundary value problem of an infinite thickness seabed instead of the boundary layer approximation. Since then several analytical solutions for investigating the linear wave-induced soil response to a poro-elastic, isotropic seabed have been developed (Jeng et al., 1999; Jeng and Rahman, 2000; Jeng and Cha, 2003; Jeng, 2013; Liao et al., 2013). Recently, Ulker et al. (2009) further investigated the applicability of different approximations, including fully dynamic (FD), partly dynamic (PD), and quasi-static (QS) soil models. Zhou et al. (2011b) applied the Fourier transformation and transmission and reflection matrices method to investigate wave-induced seabed response within a multi-layered seabed. The aforementioned investigations were based on the assumptions that the surface seabed is flat and wave pressure is extracted from analytical solution. Nevertheless, these assumptions may not be suitable for

engineering application. More studies on the wave-seabed interactions can be found in Jeng (2003). With the rapid development of computational techniques and computing resources, numerical simulation on wave-seabed-structure interaction has been attracted more attentions of researchers. Comparing with scaled-affected laboratory experiments (Guo et al., 2014), numerical simulations will allow researchers to simulate large-scale and realistic model and to couple soil model with fluid model. Cheng and Liu (1986) solved numerically the Biot's model using a Boundary Integral Equation Method considering the impact of uplift seepage force on a buried pipeline. In their study, the pipeline was considered to be buried in a rectangular trench layer with impermeable walls. Jeng and Cheng (2000) applied a Finite Difference Method to investigate wave-induced soil response around a buried pipeline. The wave-induced seabed response in the proximity of a composite breakwater exposed to a linear wave was investigated by Jeng et al. (2001) using a 2D generalized FEM numerical model (GFEM-WSSI (Wave-structure-soil-interactions)), based on the principle of repeatability. More recently, Zhou et al. (2013, 2014) analysed wave-induced seabed dynamic response in the neighbourhood of a buried pipeline in a multi-layered seabed and an anisotropic seabed, where the wave pressure acting on the seabed is calculated by analytical wave solution. Consequently, these models may not be able to predict the seabed response around pipeline when a wave is propagating over a pipeline which is partially buried or mounted on a seabed. Thus, it is required to develop a fully integrated wave-structure-soil-interaction (WSSI) model to predict the stability of a submarine pipeline.

For coupling soil model and wave model, Mizutani and Mostafa (1998) and Mostafa et al. (1999) combined Boundary Element Method (BEM) and FEM to develop an integrated model for the investigation of WSSI. This BEM-FEM model employs the Poisson's equation and the poro-elastic Biot's consolidation equations to the irrotational wave field of an incompressible, inviscid fluid and the porous seabed, respectively. Recently, Jeng and his research group (Jeng et al., 2013; Ye and Jeng, 2011; Ye et al., 2013) proposed an integrated WSSI model based on FVM-FEM, in which the wave field was described by the Reynolds-Averaged Navier-Stokes (RANS) equations and Volume of Fluid (VOF), while the dynamic Biot's equations were adopted to describe the mechanical behaviour of a porous seabed. They analysed the mechanism of wave-induced pore water pressure around the composite breakwater and predicted the potential liquefaction in seabed foundation. Dominant factors, such as permeability, saturation of seabed, and wave height, were important for evaluating

the wave-induced liquefactions around seabed foundation.

In addition to theoretical investigations, some laboratory experiments were conducted to investigate wave-induced seabed response and the stability of offshore pipelines. Sumer and Fredsøe (1991) examined the mechanism of scour in the proximity of the pipeline mounted on a bed. Sumer et al. (2001) further extended the physical experiments to investigate self-buried pipeline. Their experiments demonstrated that the onset of scour below the pipeline was caused by two primary factors, namely the excessive seepage flow and the resulting piping. A series of large-scale laboratory experiments on wave-induced residual pore pressure has been carried out by Pan et al. (2007). Zhou et al. (2011a) performed laboratory experiments to investigate the seabed response surrounding a marine pipeline with regard to the combined effects of wave and current. Recently, Yang et al. (2012a, b; 2014) conducted laboratory experiments to investigate the protection of marine pipelines. Instead of attaching a rigid spoiler on the top of a pipeline to speed up the so-called self-bury, they placed a rubber plate underneath the pipeline. They found that there existed a critical length of rubber plate. Over this critical length the scour around the pipeline was significantly reduced and the pipeline was protected; while if the rubber plate is shorter than this critical length, scour around pipeline was significantly enhanced, thus self-bury was greatly speeded up.

Though these studies have demonstrated some features of wave-induced soil response; majority of these investigations have only examined the wave-induced response with wave obtained by analytical solutions in the neighbourhood of pipeline. Uncoupled or one-way coupling approaches may not fully capture the real mechanism behind this marine process (Jeng, 2013; Jeng et al., 2013). Therefore, it is important to develop an integrated model to properly and accurately predict wave-induced soil response to the installation of a partially or fully buried pipeline in a porous seabed.

Motivated by this demanding, the aim of this paper is to propose a monolithically integrated FEM model to investigate the transient stability of pipeline located on an isotropic seabed or inside a trench in the context of water wave propagating above. To improve the applicability of the model to capture the mechanism of WSSI, a new integrated FEM model will be proposed within the environment of COMOSL Multiphysics. The RANS equations are applied to describe the flow

motion, whereas the interface between water and air is traced by conservative Level Set Method (LSM) (Olsson and Kreiss, 2005; Olsson et al., 2007) . The mechanical response of a porous seabed is described by the quasi-state Biot's equations. In present model, the integration process is one-way coupling, indicating that only excessive wave pressure is exerted on seabed surface. The wave-induced flow field surrounding pipeline is also simulated. The proposed model was firstly validated by comparing with several available experimental results. The validated model is then applied to run parametric studies to investigate wave-induced seabed response and the corresponding potential liquefaction zone around marine pipelines.

2. Numerical model

The present integrated FEM WSSI model consists of two components, namely two-phase wave model and QS model. One-way coupling algorithm will be adopted in this study, which denotes that only dynamic wave pressures act on the seabed surface. The wave-induced soil response, including pressures, soil displacements and effective stresses, is described by QS model.

2.1 Wave model

The governing equations for describing the two-phase incompressible fluid motion are the RANS equations (Ignat et al. (2000):

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

$$\rho \frac{\partial \mathbf{u}}{\partial t} + \rho(\mathbf{u} \cdot \nabla) \mathbf{u} = \nabla \cdot \left[-p\mathbf{I} + (\mu + \mu_T)(\nabla \mathbf{u} + (\nabla \mathbf{u})^T) - \frac{2}{3}\rho k\mathbf{I} \right] + \rho \mathbf{g} + ST \quad (2)$$

where $\mathbf{u}$ is the averaged velocity field; ρ is the fluid density; t is the time; p is the fluid pressure; μ is the dynamic fluid viscosity; $\mu_T = \rho C_\mu k^2 / \epsilon$ is the eddy viscosity or turbulent viscosity (the coefficient $C_\mu = 0.09$); $\mathbf{I}$ is the identity matrix; k is the turbulent kinetic energy; ϵ is the turbulent kinetic energy dissipation rate; $\mathbf{g}$ is the gravity acceleration; ST is the source term which can be demonstrated as either wave generation source term (ST_x^{wg} and ST_y^{wg}) in the wave generation zone or the wave damping term (ST_x^{wd} and ST_y^{wd}) in the wave damping zone shown in Fig. 1 (Choi and Yoon, 2009; Ha et al., 2013)

$$ST_x^{wg} = -g(2\beta x) \exp(-\beta x^2) \frac{D_1}{\omega} \sin(k_{wy}y - \omega t) \quad (3)$$

$$ST_y^{wg} = -g \exp(-\beta x^2) \frac{k_y D_1}{\omega} \cos(k_{wy}y - \omega t) \quad (4)$$

where x and y are the coordinates; ST_x^{wg} and ST_y^{wg} are the source terms in the x and y directions, respectively. The other variables are:

$$\alpha = \frac{Z_\alpha}{h_w} \left(\frac{Z_\alpha}{2h_w} + 1 \right) \quad \text{where } Z_\alpha = -0.530h_w \quad (5)$$

$$D_1 = \frac{2A(\omega^2 - \alpha_1 g k_w^4 h_w^3) \cos \theta_w}{\omega I_1 k_w [1 - \alpha(k_w h_w)^2]} \quad (6)$$

in which angular frequency ω , wave number k_w , wave direction θ_w , water depth h_w , and wave amplitude A are required to generate a prescribed wave train according to the water wave theory.

$\alpha_1 = \alpha + 1/3$ and $I_1 = \sqrt{\pi/\beta} \exp(-k_w^2/4\beta)$ with $\beta = 80/\delta^2/L_w^2$, in which L_w is the wave length and δ is a parameter that denotes the width of internal wave generation domain.

Wei and Kirby (1995) proposed to use wave damping source function to represent ST :

$$ST_x^{wd} = -C_s \mathbf{u}_x \quad (7)$$

$$ST_y^{wd} = -C_s \mathbf{u}_y \quad (8)$$

$$C_s = C_1 \frac{\exp\left[\left(\frac{x_0 - x}{x_s}\right)^{C_2}\right] - 1}{\exp(1) - 1} \quad \text{for } x_0 - x_s \leq x \leq x_0 \quad (9)$$

where ST_x^{wd} and ST_y^{wd} are the source terms in the x and y direction, respectively, and applied in the wave damping zone, $\mathbf{u}_x$ and $\mathbf{u}_y$ are the velocity in x and y direction respectively, x_0 is the coordinate of the start location in sponge layer; x_s is the length of sponge layer; C_1 and C_2 are the empirical coefficients determined by numerical test. The study of Lin and Liu (2004) suggested that C_1 could be taken as 200 and $C_2=10$ and the width of wave damping region was usually adjusted to two or three times of the wave length.

The turbulence model adopted in this study is the standard $k-\epsilon$ turbulence model (Wilcox, 1998).

$$\rho \frac{\partial k}{\partial t} + \rho(\mathbf{u} \cdot \nabla)k = \nabla \cdot \left[\left(\mu + \frac{\mu_T}{\sigma_k} \right) \nabla k \right] + P_k - \rho \epsilon \quad (10)$$

$$\rho \frac{\partial \epsilon}{\partial t} + \rho(\mathbf{u} \cdot \nabla)\epsilon = \nabla \cdot \left[\left(\mu + \frac{\mu_T}{\sigma_\epsilon} \right) \nabla \epsilon \right] + C_{\epsilon 1} \frac{\epsilon}{k} P_k - C_{\epsilon 2} \rho \frac{\epsilon^2}{k} \quad (11)$$

$$P_k = \mu_T [\nabla \mathbf{u} : (\nabla \mathbf{u} + (\nabla \mathbf{u})^T)] \quad (12)$$

where the constant coefficients are taken as (Wilcox 1998) $\sigma_k=1.0$, $\sigma_\epsilon=1.3$, $C_{\epsilon 1}=1.44$, $C_{\epsilon 2}=1.92$.

The standard wall function boundary condition is specified on the surface of both the pipeline and seabed in the simulation.

167 The interface between water and air is determined by conservative LSM (Olsson and Kreiss, 2005;
168 Olsson et al., 2007):

$$\frac{\partial \phi}{\partial t} + \nabla \cdot (\mathbf{u}\phi) = \gamma \nabla \cdot \left(\epsilon_{ls} \nabla \phi - \phi(1 - \phi) \frac{\nabla \phi}{|\nabla \phi|} \right) \quad (13)$$

169 where ϕ is the level set function; γ is the re-initialization parameter and its default value is 1 m/s;
170 ϵ_{ls} is the parameter controlling interface thickness.

171

172

2.2 Seabed model

173

174

175

176

177

178

179

180

181

182

183

184

185

186

187

188

189

190

191

192

193

194

195

196

197

198

199

200

201

202

203

204

205

206

207

208

209

210

211

212

213

214

215

216

217

218

219

220

$$\nabla^2 p_p - \frac{\gamma_w n_s \beta_s}{K} \frac{\partial p_p}{\partial t} = \frac{\gamma_w}{K} \frac{\partial}{\partial t} \left(\frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial y} \right) \quad (14)$$

181 where p_p is wave-induced oscillatory pore pressure; K is the permeability in different directions
182 and is taken as the same value in all directions in this study; n_s is the soil porosity; γ_w is the unit of
183 pore water; u_s and w_s are respectively the soil displacements in x and y direction; the
184 compressibility of pore fluid β_s can be described as :

$$\beta_s = \frac{1}{K_w} + \frac{1 - S_r}{p_{w0}} \quad (15)$$

185 where K_w is the true modulus of elasticity of water (taken as $2 \times 10^9 \text{ N/m}^2$, Yamamoto et al. 1978),
186 p_{w0} is the absolute water pressure; S_r is the degree of soil saturation.

201

202

203

204

205

206

207

208

209

210

211

212

213

214

215

216

217

218

219

220

221

222

223

224

201 The governing equations describing the force equilibrium in a porous poro-elastic soil are:

$$G \nabla^2 u_s + \frac{G}{1 - 2\mu_s} \frac{\partial}{\partial x} \left(\frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial y} \right) = \frac{\partial p_p}{\partial x} \quad (16)$$

$$G \nabla^2 w_s + \frac{G}{1 - 2\mu_s} \frac{\partial}{\partial y} \left(\frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial y} \right) = \frac{\partial p_p}{\partial y} \quad (17)$$

205 where G is the shear modulus of soil and is related to Young's modulus (E) as well as Poisson's
206 ratio (μ_s), which is

$$G = \frac{E}{2(1 + \mu_s)} \quad (18)$$

207 The relationship between soil displacement and effective stress in terms of the generalized Hooke's
208 law can be denoted as

$$\sigma'_{xx} = 2G \left[\frac{\partial u_s}{\partial x} + \frac{\mu_s}{1 - 2\mu_s} \left(\frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial y} \right) \right] \quad (19)$$

$$\sigma'_{yy} = 2G \left[\frac{\partial w_s}{\partial y} + \frac{\mu_s}{1 - 2\mu_s} \left(\frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial y} \right) \right] \quad (20)$$

$$\tau_{xy} = G \left[\frac{\partial w_s}{\partial x} + \frac{\partial u_s}{\partial y} \right] \quad (21)$$

where σ'_{xx} and σ'_{yy} are the effective normal stresses in the horizontal and vertical direction respectively; τ_{xy} is the shear stress.

To obtain the pore pressure p_p and soil displacement (u_s and w_s), several appropriate boundary conditions in the seabed domain are required to specify. At the surface of seabed, where $y = -h_w$, the vertical effective normal stress and shear stress dissipate and correspondingly are set to zero, while the pore pressure is equivalent to the dynamic wave pressure computed from the wave model.

$$p_p = p - p_w, \quad \sigma'_{yy} = 0, \tau_{xy} = 0 \text{ at } y = -h_w \quad (22)$$

where p is the wave pressure on the seabed surface obtained from the wave model, p_w is the hydrostatic water pressure.

The bottom of seabed domain ($y = -(h_w + h_s)$, h_s = soil depth) is an impermeable rigid bottom, indicating that zero displacements and no vertical flow take place there:

$$\frac{\partial p_p}{\partial y} = 0, u_s = w_s = 0 \text{ at } y = -(h_w + h_s) \quad (23)$$

In general, the flow of pore water at two lateral boundaries do not vanish as well as horizontal and vertical displacement. Ye and Jeng (2012) recommended using a large computational domain with fixing two lateral boundaries in the horizontal direction instead of applying the periodic boundary condition. Thus, for the lateral boundary conditions, both sides of seabed domain are considered to be impermeable (Zhou et al., 2013) and thus, it is assumed that there is no horizontal displacement and there is no flow in horizontal direction:

$$\frac{\partial p_p}{\partial x} = 0, u_s = 0 \text{ at } x = x_0 \text{ and } x = x_e \quad (24)$$

where x_0 and x_e are the horizontal coordinates at the beginning and the end in the soil domain, respectively. As reported in Ye and Jeng (2012), the influence of lateral boundary conditions vanishes when the simulated soil domain is longer than double wavelengths. In this study, a soil domain with triple wavelengths is adopted to the soil domain concerning to eliminate the impact of lateral boundary conditions.

The assumption that a pipeline is rigid is adopted by the majority of previous studies on wave-seabed-pipeline interaction (Cheng and Liu, 1986; Magda, 1997). However, in this study, the boundary condition of pipeline is assumed to be a rigid impermeable material and no fluid flows through the pipeline surface, then the oscillatory pore pressure gradient on the surface of pipeline vanishes:

$$\frac{\partial p_p}{\partial n} = 0 \quad (25)$$

where n is normal to the surface of pipeline.

2.3 WSSI integration process

The integration process of the present integrated WSSI model is carried out in COMSOL Multiphysics. The monolithic solver (fully coupled solver) in COMSOL, which solves for all variables simultaneously and is more robust, integrates all the aforementioned governing equations (e.g. wave model and soil model) into a large stiffness matrix. The matching mesh on the jointly used boundaries between wave model and seabed model is adopted, which implies that the surface of seabed model shares the same mesh node as that on the bottom boundary of wave model. No additional interpolation algorithm is required to exchange data for both models.

3. Validation

The integrated WSSI model is firstly validated using the available experimental data in published literatures. The numerical set-up is shown in Fig.1. The characteristic parameters of laboratory experiments are listed in Table 1.

3.1. Wave model

The wave model without coupling soil model in COMSOL is firstly verified against analytical solution as well as the wave generation model using Finite Volume Method (FVM) in OpenFOAM (Jacobsen et al., 2012). The wave is regularly progressive wave with wave height (H) being 0.12 m, wave period being 1.63 s, and water depth being 0.505 m (see Table 1). Fig 2 compares the simulated wave amplitude with the results obtained from the analytical solution and OpenFOAM at $x = 1.0$ m away from the right end of the wave generation zone. As shown in Fig. 2, the wave train computed in COMSOL is the identical as the linear wave theory (Le Méhauté, 1976) and the wave generation in OpenFOAM. As shown in Fig. 3, simulated wave amplitude generally agrees with the laboratory

measurements (Qi and Gao, 2014). Fig. 3 also demonstrates that the numerical model slightly under-predict the wave troughs.

3.2. Validation of hydrodynamics around pipeline in steady current

The second validation case of the model is to compare the simulated and measured hydrodynamics with a boundary flow pasting a pipe placed on a flat porous seabed for embedment depth $e = 0$ m. The validation case is sketched in Fig.4. The computational domain is designed with the horizontal length being 15D and 40D upstream and downstream of the pipe, respectively, and with the height of 10D. The simulated flow Reynolds number ($Re = u_0 D / \gamma_k$, where u_0 = flow velocity, γ_k = kinematic viscosity) is 45,000, while the associated boundary layer thickness (δ_b) is 1D. Both surfaces of the pipe and seabed are specified as the wall function boundary conditions.

The flow-induced pressure around pipe is demonstrated as pressure coefficient, which is defined as

$$C_{p0} = \frac{p - p_0}{\rho u_0^2 / 2} \quad (26)$$

where p_0 is the reference pressure, u_0 is flow velocity. The pressure drop coefficient is defined as $\Delta C_{p0} = C_{pA} - C_{pB}$, in which C_{pA} and C_{pB} are the pressure coefficients at point A and B, respectively.

Fig.5 shows the comparison of simulated pressure coefficients along the seabed surface with the laboratory measurements Tsiolakis (1982) and numerical results of Liang and Cheng (2005). It is seen that the simulated seabed surface pressure coefficient by the present model overall agrees well with the experimental results, demonstrating that the present model can well predict the flow-induced pressure on the seabed surface in the vicinity of pipeline. Fig. 6 is the plot of the snapshots of simulated streamlines (Fig. 6a) and pressure coefficient distribution around pipeline (Fig. 6b). Fig. 6a shows that a large vortex is seen to be generated downstream of pipeline while two small vortices are formed at the front and lee side of pipeline. Fig. 6b demonstrates that the pressure coefficient decreases along the flow direction.

The effect of embedment on pressure coefficient is also simulated and the results are presented in Fig. 7. The experimental results (Chiew, 1990) and numerical results (Liang and Cheng, 2009) are also plotted in Fig. 7 for comparison. It is seen that the pressure drop coefficient decreases with the

increase of embedment depth. The comparison also reveals that the numerical model slightly under predicts the pressure drop coefficient for the middle value of embedment depth. Nevertheless, the validation shows that the model can accurately predict flow and wave-induced pressure along seabed surface. This information will be used as the boundary condition of the porous seabed surface converting wave pressure to pore water pressure in soil model.

3.3. Wave-soil interaction model

Two laboratory experiments are used to validate the wave-induced dynamic soil response without pipeline. Wave characteristics and soil properties of these experiments are listed in Table 1. For details, readers are referred to Liu et al. (2015) and Qi and Gao (2014). The laboratory experiment carried out by Liu et al. (2015) was one-dimensional cylinder model experiment, measuring the vertical distribution of pore water pressure. Fig. 8 compares the simulated vertical distribution of excess pore water pressure with the measurements (Liu et al., 2015). In general, good agreement between simulation and experiment is obtained.

The second validation case for wave-soil interaction is the laboratory experiments conducted by (Qi and Gao, 2014). The experimental parameters can be found in Table 1. Pore water pressure was measured using sensors at the locations far away from monopile. These measurements are used to validate the present model for the case of the wave-induced pore pressures without a structure. The simulated and measured results are plotted in Fig. 9 in which pore pressure is generated by wave-monopile-seabed interaction at the location of 0.4 m upstream of pile where soil depth is 0.1 m. In general, simulated pore pressure agrees well with experimental measurements. Some deviation around wave trough and crest occurs, which may be ascribed to the fact that the regular wave profile in the laboratory flume may change to Stokes wave during its propagation from wave generator. What can be inferred from the results is that the phase of pore pressure in a porous seabed corresponds to the phase of progressive wave above. The validated wave model will be further developed to form an integrated WSSI model for the simulation of the wave train over a porous seabed.

3.4. Wave-seabed-pipeline interaction

The final validation case is the experimental measurements on wave-seabed-pipeline interaction

carried out by Turcotte et al. (1984). The pipeline was buried in a porous bed with embedment depth $e=0.167$ m (see Fig. 1 and Table 1 for experimental details). Fig. 10 shows the comparison of computed and measured normalised excess water pressure (P_0). In general, the numerical results favourably agree with the experimental measurements. Some discrepancy between the simulation and measurements is found for the case $H=0.0524$ m and $T=0.9$ s (see Fig. 10(a)). Similar deviation shown in Fig. 10(a) is also indicated in Cheng and Liu (1986). Some measurement error may take place at $\theta = 45^\circ$ for case $H=0.0302$ m and $T=2.3$ s as it does not fit with other measurements. The overall satisfactory validation indicates that the present model can capture the features of wave-soil-pipeline interaction.

4. Applications

The advantage of present integrated WSSI model is that it can be applied to evaluate the flow field and the relevant wave-induced soil response around a pipeline in a FEM model. This allows us to investigate the mechanical behaviour of wave-seabed-pipeline interaction for various engineering practice and applications. As such, a parametric study of wave-seabed-pipeline interaction has been conducted and the parameters simulated are listed in Table 2.

4.1 Pipeline mounted on a flat seabed

A submarine pipeline on a porous seabed may experience oscillatory and residual liquefaction. When liquefaction occurs around the pipeline, it will accelerate the scour and drive pipeline to sink into seabed or to float to the surface of seabed under the impact of a propagating wave. In addition to the scour underneath pipeline caused by excessive seepage flow and the corresponding piping (Sumer et al., 2001); liquefaction may cause soil to lose its carrying capability, thus failing to support pipeline. The embedment depths of pipeline in the simulation are selected to be 0 m, 1/20 of pipeline diameter (D), half pipeline diameter, and fully buried as shown in Fig. 11. The criterion suggested by Zen and Yamazaki (1990) for identifying the liquefaction zone under a 2D progressive wave is adopted here, which is:

$$(\gamma_s - \gamma_w)(|y| - h_w) + (P_b - p_p) \leq 0 \quad (27)$$

where γ_s is the unit weight of seabed, γ_w is the unit weight of water, P_b is the pore pressure on the mud-line where $y = -h_w$. It can be noted that when the excessive pore pressure exceeds the overburden soil pressure liquefaction may take place in a porous seabed.

335 The liquefaction zones of four different embedded depths below pipeline are displayed when wave
 336 trough is over pipeline in Fig. 11, where the value of vertical and horizontal axis are normalised by
 337 soil depth and wavelength respectively. The green area around pipeline is the wave-induced liquefied
 338 zone. It is clearly seen that the transient liquefied depth (L_d) under pipeline decreases with the growth
 339 of embedded depth and even vanishes below the bottom of pipeline when pipeline is fully buried,
 340 demonstrating that a fully buried pipeline with sufficient embedment depth may avoid sinking or
 341 floating by the potential liquefaction.

342
 343 Fig. 12 presents the wave-induced lee-wake vortex and liquefaction zone in the proximity of pipeline
 344 over one wave period. The flow visualization results of vortex structure of Sumer et al. (2001) is also
 345 plotted for comparison. In Fig. 12, green vector field below the mudline (i.e., $(|y| - h_w)/h_s = 0$)
 346 indicates soil displacement field, blue vector field over zero represents the flow velocity field and the
 347 dashed lines are the liquefaction depth. The comparison of wave-induced flow field around the
 348 pipeline between Sumer et al. (2001) and the present model presented in Fig. 12 demonstrates that
 349 the present integrated WSSI model qualitatively predict the evolution of vortex motion due to the
 350 existence of pipeline interacting with local environmental flow. Through a wave period, lee-wake
 351 vortex is well developed in the neighbourhood of pipeline as wave crest and wave trough are passing
 352 above, respectively. After that, the former vortex vanishes and a new vortex is generated
 353 simultaneously as shown in Fig. 12 (b) and (d). Mao (1987) showed that the vortices formed in the
 354 upstream and downstream of the pipeline were one of the main factors to transport sand particles
 355 away from the foundation of pipeline, leading to the onset of scour around pipeline. The vortex
 356 formed at the lee side may move more soil particles away from the pipeline and has a direct impact
 357 on the stability of pipeline. When a wave trough passes above the overburden soil, pressure vanishes,
 358 making soil lose its carry capacity. In this situation, the pipeline tends to float to seabed surface or to
 359 sink into seabed if liquefaction occurs.

360
 361 The liquefaction zone varying with the different embedment depth together with the detailed vortex
 362 structure and soil displacement in the vicinity of pipeline under wave trough is shown in Fig. 13. As
 363 discussed in Fig. 11, the maximum depth of liquefaction zone beneath pipeline decreases with the
 364 increase of embedment depth and the vortex structure induced by wave propagation diminishes when

365 embedment depth advances. The disappearance of vortex may decrease the possibility of the onset of
366 scour around pipeline, where massive movement of the soil particles can be avoided.

367
2

368 4.2 Trenched pipeline

4

369 How to protect an offshore pipeline is one of the significant issues in offshore pipeline engineering.
370 Generally speaking, to place a submarine pipeline inside a trench is an effective approach to protect
371 pipeline being damaged from fishing activity and oceanic loads (e.g. current and wave action). The
372 stability of pipeline is significantly affected by the appropriate backfilled depth. The present
373 integrated WSSI model is applied to investigate the mechanism of dynamic seabed response and the
374 flow field around a trenched pipeline. The scenarios investigated include fully and partially
375 backfilled pipeline, and pipeline in an open trench. The parameters simulated are listed in Table 2. In
376 general, the backfilled material for covering the pipeline can be the rocks carried from the adjacent
377 shoreline. However, in offshore engineering, the backfilled material is mostly taken from the
378 excavation of the seabed during trenching. Accordingly, the characteristics of backfilled soil are
379 similar to or exactly the same as the soil around trench.

27

280

29

381 The wave-induced oscillatory liquefaction zones with manifold backfilled depths are shown in Fig.
382 14. It can be noted that liquefaction depth around pipeline decreases with the increase of the
383 backfilled area thickness. Palmer and King (2008) suggested that a pipeline buried with backfilled
384 soil is much better protected than a pipeline inside an open trench. However, as indicated in Zhao et
385 al. (2014) and Zhao and Jeng (2014), a pipeline is better protected in the context of the backfilled
386 depth up to about 1.375D for accumulated liquefaction rather than fully backfilled pipeline. From the
387 view of oscillatory liquefaction, partially buried pipeline is not fully protected since its liquefied
388 depth is far below pipeline compared with the other fully backfilled cases. As indicated in Fig. 14
389 and considering to limit the influence of soil liquefaction for the safety operation of pipeline, the
390 moderate backfilled depth is considered to reach seabed surface or even up to one diameter of
391 pipeline over seabed surface in order to restrict the maximum depth of liquefaction zone over the
392 bottom of pipeline. The layout with enough backfilled layer can eliminate the vortex and scour
393 produced around pipeline and protect the pipeline from the transient liquefaction threat.

56

394

58

59

60

61

62

63

64

65

5. Conclusion

A new fully integrated WSSI model for evaluating wave-induced excess pore pressure around pipeline has been presented in this study. The RANS equations are used to describe the ocean wave, while the quasi-state Biot's equations govern the dynamic response of the seabed under the wave actions. All these governing equations are solved in a FEM software of COMSOL Multiphysics with a monolithic solver, which combines and solves two different groups of governing equations into a considerable stiffness matrix. The fully integrated model is verified using four different cases, including the wave model verification and three laboratory experiments. Good agreement between simulation and measurements indicate that the proposed model can be applied to predict the mechanism of wave-seabed-pipeline interaction. The advantages of the present integrated WSSI are: (1) the complex wave-induced vortex around pipeline can be simulated; (2) the wave motion and the dynamic response of porous seabed in the proximity of trenched pipeline can be determined simultaneously. The following conclusions can be drawn from the study:

- (1) The presented WSSI model is capable of predicting wave-induced dynamic soil response and potential liquefaction zone in the vicinity of partially or fully buried pipeline.
- (2) The simulation shows that sufficient embedment can avoid the lee-wake vortex occurring in the upstream and downstream of pipeline. This can significantly decrease the possibility of the onset of scour in the footing area of pipeline. The maximum transient liquefaction depth beneath the bottom of pipeline diminishes and even vanishes with the increasing embedded depth with a propagating wave above.
- (3) Partial burial may be not the best choice to protect pipeline from the point of view of soil liquefaction threat in terms of transient liquefaction. The limited cover layer thickness cannot eliminate the impact of wave-induced vortex around pipeline and effectively control the maximum oscillatory liquefaction depth (L_d) below pipeline. The vortex in the vicinity of a trenched pipeline may transport the soil particles away from the protection area and generate scour hole eventually. Under certain conditions, the sufficient thickness of backfilled depth can fully prevent the porous seabed being liquefied and the soil particles being transported by flow vortex in the vicinity of pipeline. The study suggests that the thickness of cover layer up to the

seabed surface or one diameter of pipeline over seabed surface ensures the soil within a porous seabed to maintain its carrying capacity and effectively support the pipeline to be away from the liquefaction zone.

The limitations of the present model are: (1) the wave-induced accumulated seabed response and onset of scour cannot be simulated; and (2) the present model is limited to a 2D mode due to the lack of 3D wave model in COMSOL.

Acknowledgements

The authors would like to acknowledge the financial support from Energy Technology Partnership (ETP), Wood Group Kenny and University of Aberdeen. The comments and suggestions made by the Reviewers have greatly improved the quality of the final manuscript. ZL thanks Professor Dubravka Pokrajac for useful discussion on hydrodynamics around pipeline.

References

- Biot, M.A., 1956. Theory of propagation of elastic waves in a fluid - saturated porous solid. I. Low - frequency range. *J. Acoust. Soc. Am.* 28, 168-178.
- Biot, M.A., 1941. General theory of three - dimensional consolidation. *J. Appl. Phys.* 12, 155-164.
- Cheng, A.H., Liu, P.L., 1986. Seepage force on a pipeline buried in a poroelastic seabed under wave loadings. *Appl. Ocean Res.* 8, 22-32.
- Choi, J., Yoon, S.B., 2009. Numerical simulations using momentum source wave-maker applied to RANS equation model. *Coast. Eng.* 56, 1043-1060.
- Guo, YK, Zhang, ZY and Shi, B. 2014. "Numerical simulation of gravity current descending a slope into a linearly stratified environment." *J. Hydraul. Eng., ASCE*, 140(12) doi:10.1061/(ASCE)HY.1943-7900.0000936.
- Ha, T., Lin, P., Cho, Y.-S, 2013. Generation of 3D regular and irregular waves using Navier-Stokes equations model with an internal wave maker. *Coast. Eng.* 76, 55-67.
- Hsu, J.R.C., Jeng, D.-S., 1994. Wave-induced soil response in an unsaturated anisotropic seabed of finite thickness. *International Journal for Numerical & Analytical Methods in Geomechanics* 18, 785-807.
- Ignat, L., Pelletier, D., Ilinca, F., 2000. A universal formulation of two-equation models for adaptive computation of turbulent flows. *Comput. Methods Appl. Mech. Eng.* 189, 1119-1139.

Jacobsen, N.G., Fuhrman, D.R., Fredsøe, J., 2012. A wave generation toolbox for the open - source CFD library: OpenFoam®. *Int. J. Numer. Methods Fluids* 70, 1073-1088.

Jeng, D.-S., 2003. Wave-induced sea floor dynamics. *Appl. Mech. Rev.* 56, 407-429.

Jeng, D.-S., 2013. *Porous Models for Wave-Seabed Interactions*. Springer Berlin Heidelberg, Germany.

Jeng, D.-S., Cha, D.H., 2003. Effects of dynamic soil behavior and wave non-linearity on the wave-induced pore pressure and effective stresses in porous seabed. *Ocean Eng.* 30, 2065-2089.

Jeng, D.-S., Cha, D.H., Lin, Y., Hu, P., 2001. Wave-induced pore pressure around a composite breakwater. *Ocean Eng.* 28, 1413-1435.

Jeng, D.-S., Cheng, L., 2000. Wave-induced seabed instability around a buried pipeline in a poro-elastic seabed. *Ocean Eng.* 27, 127-146.

Jeng, D.-S., Rahman, M., 2000. Effective stresses in a porous seabed of finite thickness: Inertia effects. *Canadian geotechnical journal* 37, 1383-1392.

Jeng, D.-S., Rahman, M., Lee, T., 1999. Effects of inertia forces on wave-induced seabed response. *Int. J. Offshore Polar Eng.* 9.

Jeng, D.-S., Ye, J.-H., Zhang, J.-S., Liu, P.L.-., 2013. An integrated model for the wave-induced seabed response around marine structures: Model verifications and applications. *Coast. Eng.* 72, 1-19.

Le Méhauté B., 1976. *An Introduction to Hydrodynamics and Water Waves*, spring study ed. Springer Berlin Heidelberg, Germany.

Liang, D., Cheng, L., 2005. A numerical model of onset of scour below offshore pipelines subject to steady currents. *Frontiers in Offshore Geotechnics* 637–644. Liao C.C., Jeng D.-S., and Zhang L.L., 2013. An Analytical Approximation for Dynamic Soil Response of a Porous Seabed due to Combined Wave and Current Loading. *J. of Coast. Res.*, In-Press.

Lin, P., Liu, P.L., 2004. Discussion of “Vertical variation of the flow across the surf zone” [*Coast. Eng.* 45 (2002) 169–198]. *Coast. Eng.* 50, 161-164.

Liu, B., Jeng, D.-S., Ye, G. L. & Yang, B. 2015. Laboratory study for pore pressures in sandy deposit under wave loading. *Ocean Engineering*, 106, 207-219..

Madsen, O.S., 1974. The stability of a sand bed under breaking waves. *Proc. 14th Coastal Engineering Conf, ASCE.* 2, 776-794.

488 Madsen, O.S., 1978. Wave-induced pore pressures and effective stresses in a porous bed.
489 Geotechnique 28, 377-393.

490 Magda, W., 1997. Wave-induced uplift force acting on a submarine buried pipeline in a compressible
2 seabed. Ocean Eng. 24(6), 551-576.

491
4
492 Mao, Y., 1987. The interaction between a pipeline and an erodible bed. SERIES PAPER
6
493 TECHNICAL UNIVERSITY OF DENMARK .

494 Mei, C.C., 1989. Applied dynamics of ocean surface waves, New York, Wiley-Interscience.

10
11
495 Mei, C.C., Foda, M.A., 1981. Wave-induced responses in a fluid-filled poro-elastic solid with a free
12
496 surface—a boundary layer theory. Geophysical Journal International 66, 597-631.

13
14
497 Mizutani, N., Mostafa, A.M., 1998. Nonlinear wave-induced seabed instability around coastal
15
16
498 structures. Coast. Eng. J. 40, 131-160.

17
18
499 Mostafa, A.M., Mizutani, N., Iwata, K., 1999. Nonlinear wave, composite breakwater, and seabed
20
21
500 dynamic interaction. Journal of Waterway, Port, Coastal, and Ocean Engineering, ASCE, 125, 88-97.

22
23
501 Olsson, E., Kreiss, G., 2005. A conservative level set method for two phase flow. Journal of
24
25
502 Computational Physics 210, 225-246.

26
27
503 Olsson, E., Kreiss, G., Zahedi, S., 2007. A conservative level set method for two phase flow II.
28
29
504 Journal of Computational Physics 225, 785-807.

30
31
505 Palmer, A.C., King, R.A., 2008. Subsea Pipeline Engineering, second ed. PennWell Books, Tulsa,
32
33
506 Oklahoma.

34
35
507 Pan, D., Wang, L., Pan, C., Hu, J., 2007. Experimental investigation on the wave-induced pore
36
37
508 pressure around shallowly embedded pipelines. Acta Oceanologica Sinica. 26, 125-135.

38
39
40
509 Qi, W., Gao, F., 2014. Physical modeling of local scour development around a large-diameter
41
42
510 monopile in combined waves and current. Coast. Eng. 83, 72-81.

43
44
511 Sleath, J.F.A. (1970). Wave-induced pressures in beds of sand. J. Hydraul. Div., ASCE. 96, 367-379.

45
46
512 Sumer, B.M., Fredsøe, J., 2002. The Mechanics of Scour in the Marine Environment. World
47
48
513 Scientific Publishing Co Pte Ltd.

49
50
514 Sumer, B. M., 2014. Liquefaction around marine structures. World Scientific, New Jersey.

51
52
515 Sumer, B.M., Truelsen, C., Sichmann, T., Fredsøe, J., 2001. Onset of scour below pipelines and
53
54
516 self-burial. Coast. Eng. 42, 313-335.

55
56
517 Sumer, B., Fredsøe, J., 1991. Onset of scour below a pipeline exposed to waves. International
57
58
518 Society of Offshore and Polar Engineers.

519 Tsiolakis, E.P., 1982. Reynoldische spannungen in einer mit einem kreiszylinder gestörten
520 turbulenten plattengrenzschicht. Dissertation, Dortmund University, Dortmund, Germany (in
521 German).

522 Turcotte, B.R., Kulhawy, F.H., Liu, P.L., 1984. Laboratory Evaluation of Wave Tank Parameters for
523 Wave-Sediment Interaction. School of Civil and Environmental Engineering, Cornell University.

524 Ulker, M., Rahman, M., Jeng, D.-S., 2009. Wave-induced response of seabed: various formulations
525 and their applicability. Appl. Ocean Res. 31, 12-24.

526 Wei, G., Kirby, J.T., 1995. Time-dependent numerical code for extended Boussinesq equations.
527 Journal of Waterway, Port, Coastal, and Ocean Engineering 121, 251-261.

528 Wilcox, D.C., 1998. Turbulence Modeling for CFD. DCW industries La Canada, CA.

529 Yamamoto, T., Koning, H., Sellmeijer, H., Hijum, E.V., 1978. On the response of a poro-elastic bed
530 to water waves. J. Fluid Mech. 87, 193-206.

531 Yamamoto, T., 1997, Wave-induced instability in seabed. Proc. Coastal Sediment '77, ASCE, pp.
532 898-913.

533 Yang, L.P., Guo, Y.K., Shi, B, Kuang, C.P., Xu, W.L. and Cao, S.Y., 2012. Study of scour around
534 submarine pipeline with a rubber plate or rigid spoiler in wave conditions. ASCE Journal of
535 Waterway, Port, Ocean and Coast Engineering, 138:484-490.

536 Yang, L.P., Shi, B., Guo, Y.K., and Wen, X.Y, 2012. Calculation and experiment on scour depth for
537 submarine pipeline with a spoiler, Ocean Eng., 55:191-198.

538 Yang, L.P., Shi, B, Guo, Y.K., Zhang, L.X., Zhang, J.S. and Han Y., 2014. Scour protection of
539 submarine pipelines using rubber plates underneath the pipes. Ocean Eng., 84, 176-182.

540 Ye, J.-H., Jeng, D.-S., Wang, R., Zhu, C., 2013. Validation of a 2-D semi-coupled numerical model
541 for fluid–structure–seabed interaction. J. Fluids Struct. 42, 333-357.

542 Ye, J.-H., Jeng, D.-S., 2011. Effects of bottom shear stresses on the wave-induced dynamic response
543 in a porous seabed: PORO-WSSI (shear) model. Acta Mech Sin 27, 898-910.

544 Ye, J.-H., Jeng, D.-S., 2012. Response of Porous Seabed to Nature Loadings: Waves and Currents. J.
545 Eng. Mech. , ASCE. 138, 601-613.

546 Yuhi, M., Ishida, H., 1998. Analytical solution for wave-induced seabed response in a soil-water
547 two-phase mixture. Coast. Engineering in Japan 40, 367-381.

548 Zen, K., Yamazaki, H., 1990. Mechanism of wave-induced liquefaction and densification in seabed.
549 Soil Found 30, 90-104.

550 Zhao H.Y., Jeng D.-S., 2014. Numerical study for wave-induced pore pressure accumulations around
551 buried pipeline: Effects of back-filled trench layer. The 14th International Conference of the Inter-
552 national Association for Computer Methods and Advances in Geomechanics (14IACMAG) , Kyoto,
553 Japan, September 22-25, 2014, 1113-1118.

554 Zhao, H.-Y., Jeng, D.-S., Guo, Z., Zhang, J., 2014. Two-dimensional model for pore pressure
555 accumulations in the vicinity of a buried pipeline. Journal of Offshore Mechanics and Arctic
556 Engineering, ASME, 136, 042001.

557 Zhou, C., Li, G., Dong, P., Shi, J., Xu, J., 2011a. An experimental study of seabed responses around
558 a marine pipeline under wave and current conditions. Ocean Eng. 38, 226-234.

559 Zhou, X., Jeng, D.-S., Yan, Y., Wang, J., 2013. Wave-induced multi-layered seabed response around
560 a buried pipeline. Ocean Eng. 72, 195-208.

561 Zhou, X., Wang, J., Zhang, J., Jeng, D.-S., 2014. Wave and current induced seabed response around
562 a submarine pipeline in an anisotropic seabed. Ocean Eng. 75, 112-127.

563 Zhou, X., Xu, B., Wang, J., Li, Y., 2011b. An analytical solution for wave-induced seabed response
564 in a multi-layered poro-elastic seabed. Ocean Eng. 38, 119-129.

Table 1

Experiments	H (m)	T (s)	h_w (m)	G (N/m ²)	μ_s	K (m/s)	n_s	S_r	h_s (m)
Jacobsen et al. (2012)	0.12	1.63	0.505	0	0	0	0	0	0
Qi and Gao (2014)	0.12	1.4	0.5	1×10^7	0.3	1.88×10^{-4}	0.4353	1.0	0.5
Liu and Jeng (2013)	3.5	9	5.2	1.27×10^7	0.3	1.8×10^{-4}	0.425	0.996	1.8
Turcotte et al. (1984)	0.0524	0.9	0.533	6.4×10^5	0.33	1.1×10^{-3}	0.42	0.95	0.826
	0.143	1.75	0.533	6.4×10^5	0.33	1.1×10^{-3}	0.42	0.95	0.826
	0.0302	2.3	0.533	6.4×10^5	0.33	1.1×10^{-3}	0.42	0.95	0.826
	h_t (m)	e (m)	D (m)						
	0.826	0.167	0.168						

Table 2

Wave characteristics			
Wave height(m)	5	Wave period (s)	8
Water depth (m)	20	Wave length (m)	88.73
Seabed characteristics			
Seabed thickness (m)	40	Poisson ratio	0.4
Shear modulus (Pa)	1.5×10^7	Permeability (m/s)	1×10^{-5}
Degree of saturation	0.98		
Backfill soil characteristics			
Shear modulus (Pa)	1.5×10^7	Permeability (m/s)	1×10^{-3}
Trench bottom width (m)	3	Trench slope angle (°)	30
Trench depth (m)	3	Degree of saturation	0.995
Pipeline characteristics			
Diameter (m)	1	Buried depth (m)	2

Figure 1
[Click here to download high resolution image](#)

Figure 2
[Click here to download high resolution image](#)

Figure 3
[Click here to download high resolution image](#)

Figure 4
[Click here to download high resolution image](#)

Figure 5
[Click here to download high resolution image](#)

(a) Streamlines

(b) Contour of pressure coefficient

Figure 7
[Click here to download high resolution image](#)

Figure 8
[Click here to download high resolution image](#)

Figure 9
[Click here to download high resolution image](#)

(a) $H = 0.0524$ m and $T = 0.9$ s

(b) $H = 0.143$ m and $T = 1.75$ s

(c) $H = 0.0302$ m and $T = 2.3$ s

(a) Embedded depth: 0 m

(b) Embedded depth: 0.05D

(c) Embedded depth: 0.5D

(d) Embedded depth: one diameter

(a) Wave crest over pipeline

(b) After wave crest

(c) Wave trough over pipeline

(d) After wave trough

(a) Pipeline positioned on a flat seabed

(b) Embedment depth up to $0.05D$

(c) Embedment depth up to $0.5D$

(d) Embedment depth up to one diameter

(a) Pipeline inside an open trench without backfilled layer

(b) Backfilled depth up to half diameter of pipeline

(c) Backfilled depth up to one diameter of pipeline

(d) Fully Backfilled to seabed surface

(e) Fully backfilled to one diameter of pipeline over seabed surface

Fig. 1 Sketch of wave-seabed-pipeline interaction

Fig. 2 Comparisons of wave model in COMSOL and wave model in OpenFOAM and analytical solution

Fig. 3 Comparisons of wave model in COMSOL and wave model in OpenFOAM and experimental results

Fig 4 Sketch of hydrodynamics of pipeline in a steady current

Fig 5 Pressure coefficient distribution along the seabed surface at both sides of pipeline with $e/D = 0$, $Re = 45,000$, and $\delta_b = 1D$.

Fig 6 Numerical flow predication around pipeline with $e/D = 0$, $Re = 45,000$, and $\delta_b = 1D$. (a) Streamlines; (b) Distribution of pressure coefficient.

Fig 7 Variation of ΔC_{p0} with e/D , $Re = 10,000$, and $\delta_b = 10D$

Fig. 8 Comparison of the simulated and measured veritcal distribution of excess pore water pressure

Fig. 9 Comparison of simulated and measured time history of pore pressure at $h_s = 0.1$ m for progressive wave over a porous seabed

Fig. 10 Comparison of pore pressure around pipeline with experimental results (Turcotte et al., 1984)

Fig. 11 Liquefaction zone (above the dashed line) of various embedded depth beneath the pipeline

Fig. 12 Simulated (left) wave-induced vortex around pipeline and liquefaction zone (above the dashed line) during one wave period compared with the measured vortex (right) (Sumer et al.,2001)

Fig. 13 Liquefaction zone (above the dashed line) and wave-induced vortex for different embedment depth under wave trough

Fig. 4 Liquefaction zone (above the dashed line) of various backfilled depth