Combined wave-current induced oscillatory seabed responses around two pipelines in tandem

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Combined wave-current induced oscillatory seabed responses around two pipelines in tandem

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ABSTRACT
Seabed stability around submarine pipelines under wave-plus-current loading is one of the major issues in offshore projects. Unlike previous works mainly focusing on the evaluation of the seabed response around a single pipeline, in this study, two pipelines in tandem will be considered. The previous model (PORO-FSSI-FOAM) will be adopted to investigate the effect of the gap ratios \((G/D)\) of twin pipes on the wave & current-induced oscillatory seabed response. Based on numerical examples, the following conclusions were found: (i) the effect of different \(G/D\) between twin pipes on the pore-water pressure and resulting seabed liquefaction cannot always be ignored; and (ii) as the gap ratio \((G/D)\) is 1.25, the soil response beneath both pipelines is significant than in the condition of a single body.

KEY WORDS: pipelines in tandem; excess pore-water pressure; horizontal distance between twin pipes; OpenFOAM; seabed liquefaction; 2D model

INTRODUCTION
Pipelines have been one of the essential associated installations for the oil and gas industry, which have been used for the transportation of oil and gas from offshore to onshore. Since the first offshore pipeline was built by Brown Root to carry oil in 1954, the development of submarine pipeline networks has across the globe, even been regarded as the “lifelines” of the oil industry (Sumer and Fredsøe, 2002). The existence of a submarine pipeline does not only alter the nearby flow morphology, but also enhances the surrounding seafloor instability (including soil liquefaction, scour and shear failure) and ultimately cause damage or failure of the pipeline (Sumer, 2014).

In general, when the seabed is exposed to the wave, the dynamic pressures along the seafloor can induce the pore pressures, effective stresses and soil displacements within the seabed (Jeng, 2012). Then, the seabed in the vicinity of the pipeline could become unstable or even liquefied due to the increasing excess pore pressures and the reducing vertical effective stresses. Once the liquefaction occurs, the soil will behave like a kind of heavy fluid without any shear resistance, which greatly increase the risk for the pipeline to sag.

Two categories associated with the mechanisms of wave-induced seabed liquefaction have been identified (Zen and Yamazaki, 1990; Nago et al., 1993): oscillatory and residual liquefaction, which are in accordance with the generated procedure of pore water pressure. Among these, the oscillatory liquefaction normally appears instantaneously under wave troughs in very dense sand deposits with linear reversible soil characteristics, which is related to phase lag and amplitude decay in the oscillatory pore pressures (Madsen, 1978). The residual liquefaction is generally caused by the build-up of the excess pore pressure under the volumetric wave loading, which is usually observed within the fully saturated seabed (Seed and Rahman, 1978; Sumer, 2014). In this study, only the oscillatory liquefaction mechanism with transient pore pressure is examined.

Based on Biot’s poro-elastic theory (Biot, 1941), numerous numerical studies for the Wave-Seabed-Structure Interactions (WSSI) have been available in the literature. Among these, Jeng and Cheng (2000) proposed a FDM model to discuss the possibility of wave-driven shear failure around a pipeline. They found that as the burial depth and pipe radius increase, the potential shear failure around a pipeline rises in the seabed with a lower degree of saturation. A few FEM studies have been implemented to explore the wave-induced seabed responses around a buried or trenched pipeline (Gao et al., 2003; Gao and Wu, 2006; Zhang et al., 2011). However, the aforementioned investigations have not considered the combined loading of waves and currents. A more recent attempt related to the wave & current-driven liquefaction around a buried pipeline was numerically performed by Liang et al. (2020) in a 3D pattern. In their study, the design of a trench layer was also involved.

Extensive investigations only focused on a single pipeline, although it is common to have two tandem pipelines in engineering practice for transporting hydrocarbons. With the development of offshore oil and gas engineering, more than one pipe is required to be laid along the same path on the seabed. Regarding the arrangement of multiple pipes, the flow patterns and seabed response around them are difficult to estimate (Rados et al., 2000). Due to technical and economic factors, two identical pipelines are occasionally laid in tandem. For instance, the recent pipeline project, "Nord Stream 2", consists of the construction of two parallel inlet and outlet natural gas pipelines through the Baltic
Sea (Hirschhausen et al., 2018). In the existing design method, the two tandem pipelines were processed individually. However, Li et al. (2020) revealed the twin pipes become more dependent with decreasing horizontal distance, which means that it is not reasonable to treat them separately. The previous research related to the two pipelines that are laid in parallel mainly focused on the local scour under different flow conditions. For example, Zhao et al. (2015) investigated the scour around the twin pipelines under steady current and various pipeline gap conditions. The numerical results demonstrated that the two pipes should be placed as close to each other as possible to decrease scour depths under the conditions of clear water and live bed. Later, Hu et al. (2019) examined the effect of gap ratio and the incipient stream velocity on the seabed scour in the vicinity of two tandem pipelines using the CFD-DEM method. In their work, the scour depths are strongly reliant on and directly proportional to the incoming stream velocity, and the equilibrium bed profiles are similar with varied gap ratios under the same incident stream velocity. The more recent attempt for the scour beneath two pipes in tandem was conducted by Li et al. (2020) by considering the combined loading of wave and current. When the current has equal strength to the waves, they concluded that smaller horizontal gap ratios result in delayed scour beneath the downstream pipeline. Nevertheless, research on the seabed responses and the associated liquefaction beneath two offshore pipelines in tandem subjected to combined ocean wave and current loading is not available in the literature.

In this study, the previous model (PORO-FSSI-FOAM, Liang et al., 2020) will be adopted to investigate the wave & current-induced seabed response around two tandem pipelines in the 2D domain. The present model is implemented with the open-source code OpenFOAM® (version 8.0). Both hydrodynamic and geotechnical models are integrated within the framework of OpenFOAM. The present model is validated against the experimental data first. Afterwards, the impact of horizontal gap ratios \((G/D)\) ranging from 0.25~4 on the liquefaction depth under twin pipes is examined.

**NUMERICAL MODEL**

The configuration of the computational domain in the present model is given in Fig. 1. Specifically, the two pipelines with the same outer diameter \((D)\) located on a porous seabed \((L_x \times d_x)\) are involved in this study. In terms of the two tandem pipes, \(G\) denotes the horizontal distance between them. Besides, the fifth-order Stokes wave theory (Skjelbreia and Hendrickson, 1960) is employed for the wave propagating from left to right.

![Fig. 1 The detailed sketch of the numerical model.](image)

**Flow Model**

Within the framework of OpenFOAM®, Higuera et al. (2013) developed IHFoam (also named as Olaflow later) that is employed for wave-current-structure interactions, based on VARANS equations. In terms of this FVM hydrodynamic model, an advanced algorithm, PIMPLE (a mixture between SIMPLE and PISO algorithms), is used for pressure-velocity coupling. Based on the assumption of incompressible and continuous fluids, the governing equations (i.e., the conservation of mass, conservation of momentum and VOF function advection equation) are adopted for reproducing the movement of two-phase flow, which can be expressed as:

\[
\frac{\partial \mathbf{u}_i}{\partial x_i} = 0
\]  

\[
\frac{\partial \rho_f \mathbf{u}_i}{\partial t} + \frac{\partial}{\partial x_j} \left[ \rho_f (\mathbf{u}_i \mathbf{u}_j) \right] = -\frac{\partial p^*}{\partial x_i} + n g \mathbf{X}_j \frac{\partial \mathbf{u}_i}{\partial x_j} + \frac{\partial}{\partial x_j} \left[ \mu_{eff} \frac{\partial \mathbf{u}_i}{\partial x_j} \right] - \left[ CT \right]
\]  

\[
\frac{\partial \alpha_1}{\partial t} + \frac{1}{n} \frac{\partial (\alpha_1 \rho_f)}{\partial x_i} + \frac{1}{n} \frac{\partial (\alpha_2 \rho_a)}{\partial x_i} = 0
\]  

where \(\mathbf{u}_i\) is the velocity vector; \(n\) presents the porosity while \(p^*\) is the pseudo-dynamic pressure; \(g\) and \(\mathbf{X}\) stand for the acceleration of gravity and the position vector, respectively; \(\rho_f\) is the density of fluid; \(\mu_{eff}\) defines the efficient dynamic viscosity; \(\mathbf{u}_{eff}\) represents the relative velocity field. The last term in Eq (2) represents the resistance of the porous media; \(\alpha_1\) is the VOF indicator function representing the quantity of water unit of volume in each cell. The following expression reveals \(\alpha_1\) varies from 0 to 1 with the corresponding situation:

\[
\alpha_1 = \begin{cases} 
1, & \text{water} \\
0 < \alpha_1 < 1, & \text{free surface} \\
0, & \text{air}
\end{cases}
\]  

Traditionally, as a phase function, \(\alpha_1\) can represent any spatial variation of fluid properties (e.g. density and viscosity) with considering the mixture properties:

\[
\Phi = \alpha_1 \Phi_{water} + (1 - \alpha_1) \Phi_{air},
\]  

in which \(\Phi_{water}\) and \(\Phi_{air}\) present the properties of water and air, separately.

As depicted in Fig. 1, the specified boundary conditions are used in the fluid model. In particular, an active wave absorption theory is adopted to prevent the re-reflection of incoming waves at the outlet by imposing a reasonable velocity profile on absorbent boundaries. Regarding IHFoam, the more detailed description of corresponding boundary conditions can refer to Higuera et al. (2013).

**Seabed Model**

In this study, PORO-FSSI-FOAM (Liang et al., 2020) as a seabed sub-model is employed under the OpenFOAM environment. With neglecting the inertia terms of the solid and fluid, the quasi-static Biot equation (Biot, 1941) is used for the wave-induced seabed response based on the assumption of compressible pore fluid and homogeneous isotropic seafloor. The flow is considered to be governed by Darcy’s law.
For the two-dimensional model, the governing equations for compressible porous soil in a compressible porous medium, combined with the conservation of mass, can be presented by the following expression:

\[\nabla^2 p_s - \frac{\gamma_w n \beta_s \partial p_t}{k_t} \frac{\partial}{\partial t} = \frac{\gamma_w}{k_t} \frac{\partial}{\partial x} \left( \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z} \right),\]

where \(\nabla^2\) stands for the Laplace’s operator; \(p_t\) denotes the wave-induced pore pressure while \(\gamma_w\) is the unit weight of water; \(n_i\) and \(k_t\) present the soil porosity and Darcy’s permeability, respectively; \(u_s\) and \(w_s\) are the soil displacements in the \(x\) - and \(z\) - directions, separately; the compressibility of the pore fluid (\(\beta_s\)) is defined by Verruijt (1969),

\[\beta_s = \frac{1}{K_w} + \frac{1 - S_s}{P_{\text{v0}}},\]

where \(K_w\) stands for the true bulk modulus of elasticity \((K_w = 1.95 \times 10^9 N/m^2)\), Yamamoto et al. (1978); \(S_s\) is the degree of saturation while \(P_{\text{v0}}\) is the absolute water pressure, which defined as \(P_{\text{v0}} = \gamma_w d_w\) \((d_w\) is the water depth). Notably, when \(S_s\) is taken as 1, meaning the soil is fully saturated (i.e., air-free), then \(\beta_s = 1/K_w\).

Based on Hooke’s law, the relationships between stresses and strains are given by:

\[\sigma_{s} = 2G_s \left[ \frac{\partial u_s}{\partial x} + \frac{\mu_s}{1 - 2\mu_s} \left( \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z} \right) \right],\]

\[\sigma_{z} = 2G_s \left[ \frac{\partial w_s}{\partial z} + \frac{\mu_s}{1 - 2\mu_s} \left( \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z} \right) \right],\]

\[\tau_{xz} = G_s \left( \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z} \right) = \tau_{\text{fs}},\]

where shear modulus of soil \(G_s\) is defined with Young’s modulus \(E_s\) and Poisson’s ratio \(\mu_s\) in the form of \(E_s/(1 + \mu_s)\).

The governing equations for force balance in the soil are expressed as:

\[G_s \nabla^2 u_s + \frac{G_s}{(1 - 2\mu_s)} \frac{\partial}{\partial x} \left( \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z} \right) = \frac{\partial p_s}{\partial x},\]

\[G_s \nabla^2 w_s + \frac{G_s}{(1 - 2\mu_s)} \frac{\partial}{\partial z} \left( \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z} \right) = \frac{\partial p_s}{\partial z}.\]

To solve the above governing equations for the wave-current-seabed-pipeline interactions (WCSP), several boundary conditions are required, which are summarised in Table 1. At the seabed surface, the dynamic wave pressure \((p_o)\) from the flow model is directly acting on it, while the vertical effective normal stress and shear stress are assumed to vanish. It is also seen from the table that the vertical flow gradient and the soil displacement are specified to zero at the bottom of the seabed in accordance with the impermeable rigid boundary condition. As for the soil domain’s lateral boundaries, the soil skeleton is allowed to slip, and the normal pore pressure gradient vanishes (i.e., no flow). It is noted that the lateral length of the computational domain is commonly specified as three times the wavelength, contributing to preventing any computational error due to the reflective waves of the lateral boundary (Ye and Jeng, 2012).

In addition, the two parallel pipelines are assumed to be rigid impermeable objects with a no-flow boundary condition on their surface:

\[\frac{\partial p_s}{\partial n} = 0\]

where \(n\) is the surface normal of the pipeline on the boundary.

<table>
<thead>
<tr>
<th>Mesh Convergence</th>
</tr>
</thead>
<tbody>
<tr>
<td>It is indispensable to check the convergence of the present numerical model before studying the dynamic soil response around two tandem pipelines laid on a porous seabed. Notably, the dynamic water pressures extracted from the whole bottom of the flow model are interpolated to the grid points on the seabed interface; thus, the grid points of both sub-models do not need to match correspondingly. Based on that, we use a non-matching mesh method in the one-way coupling process to minimize the computational cost.</td>
</tr>
</tbody>
</table>

Fig. 2 shows the time history of free surface elevation for six different mesh schemes in the fluid model. It can be found that the results have no significant change with the scheme of \(L_w/300\) and \(H_o/30\) in the lateral and longitudinal directions, respectively, which means this sub-model is convergent. As for the solid model, the mesh resolutions of \(L_w/150\) and \(h_o/20\) show the better convergence effect in the horizontal and vertical directions separately, as demonstrated in Fig. 3. Consequently, the present one-way coupling scheme is quite efficient. It allows both sub-models to be solved sequentially with different meshes, significantly reducing computational time.

![Fig. 2 Time variation of free surface elevation for different mesh schemes (when \(H_o=0.5 m\), \(T_o=5.0 s\), \(d_w=2.5 m\) and \(L_w=23.086 m\).)]
Fig. 3 Time variation of dynamic pore-water pressure ($p_s$) for different mesh schemes (when $k_s=1.8 \times 10^{-4}$ m/s, $\mu_s=0.3$, $G_s=3.302 \times 10^7$ N/m$^2$, $n_s=0.425$ and $S_r=0.975$).

Model Validation

In this section, an experimental work related to the wave-induced seabed response around twin pipes is applied to validate the present model. Recently, a series of experiments was conducted by Zhai et al. (2022) for wave-induced excess pore pressure around two tandem pipelines, especially considering the buried depths and the gap conditions. As shown in Fig. 4, the experiment was conducted in a wave flume of 50 m in length, 1.3 in depth and 1 m in width. In their experiment, a piston-type wave generator was equipped at the wave flume, and two porous, slopping wave absorbers at both ends were used to eliminate wave reflection. The sediment basin was located in the middle portion of the flume with 2 m long, 1 m width and 0.58 m deep. Two PMMA pipes with the same diameter of 12 cm were used to model the offshore pipes. Concerning the data obtainment, eight sets of pore pressure transducers were fixed along the periphery of the upstream pipeline circumference with an interval of $\pi/4$ (see Fig. 4).

At P3, the comparison between the measured and simulated excess pore-water pressure ($\Delta u_e = p_s - p_w$) under the fully buried twin pipes ($d_l = D_m$) is exhibited in Fig. 5, where the distance between two pipes is 0.24 m. The overall good agreement between the experimental data and simulation results is clearly found. In a fully buried condition, Fig. 6 shows the comparison of the measured results (Zhai et al., 2022)) and corresponding simulated data for the excess pore pressure amplitude ($\Delta u_e$) along the periphery of a single pipe and the upstream pipe for the pipelines in tandem. It is apparent that $|\Delta u_e|$ around the upstream pipeline for twin pipelines is larger than that in the vicinity of a single pipeline. In summary, the overall agreement between the observed data and simulation results proves that the integrated model is reliable for predicting seabed response around two parallel pipelines.

RESULTS AND DISCUSSIONS

The aim of this paper is to explore the liquefaction potential around the two tandem pipelines under natural dynamic loading. In this study, the combined effects of current and seabed properties on the wave-driven soil
responses for twin pipes laid on the seabed are investigated. Furthermore, the simulated results between twin and single pipelines are compared for otherwise identical conditions. Unless otherwise indicated, all wave parameters, properties of the sandy seabed and the offshore pipes are provided in Table 2. Notably, the external diameter \( D \) of two parallel pipelines is the same in all numerical cases.

Table 2 Input data of this numerical example.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Wave characteristics</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wave height ( (H_w) )</td>
<td>2.0</td>
<td>[m]</td>
</tr>
<tr>
<td>Water depth ( (d_w) )</td>
<td>6.0</td>
<td>[m]</td>
</tr>
<tr>
<td>Wave period ( (T_w) )</td>
<td>6.0 or various</td>
<td>[s]</td>
</tr>
<tr>
<td>Wave length ( (L_w) )</td>
<td>40.852</td>
<td>[m]</td>
</tr>
<tr>
<td>Current velocity ( (U_c) )</td>
<td>1 or various</td>
<td>[m/s]</td>
</tr>
<tr>
<td><strong>Seabed characteristics</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permeability ( (k_s) )</td>
<td>1.0 \times 10^{-3} or various</td>
<td>[m/s]</td>
</tr>
<tr>
<td>Poisson’s ratio ( (\mu_s) )</td>
<td>0.33</td>
<td>–</td>
</tr>
<tr>
<td>Porosity ( (n_s) )</td>
<td>0.425</td>
<td>–</td>
</tr>
<tr>
<td>Degree of saturation ( (S_s) )</td>
<td>97</td>
<td>%</td>
</tr>
<tr>
<td>Shear modules ( (G_s) )</td>
<td>10^7</td>
<td>[N/m²]</td>
</tr>
<tr>
<td>Submerged weight of soil ( (\gamma'_s) )</td>
<td>10.71</td>
<td>[kN/m³]</td>
</tr>
<tr>
<td>Seabed thickness ( (h_s) )</td>
<td>6.0</td>
<td>[m]</td>
</tr>
<tr>
<td>Seabed length ( (L_s) )</td>
<td>3L_w</td>
<td>[m]</td>
</tr>
<tr>
<td><strong>Pipeline characteristics</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young’s modulus ( (E_p) )</td>
<td>2.09 \times 10^{11}</td>
<td>[N/m²]</td>
</tr>
<tr>
<td>Pipeline diameter ( (D) )</td>
<td>2.0</td>
<td>[m]</td>
</tr>
</tbody>
</table>

Comparison of the results between twin pipelines and a single pipe

The distribution of the maximum amplitude of the flow velocity (i.e. \( |u_f| = \sqrt{u_{xf}^2 + u_{zf}^2} \), where \( u_{xf} \) and \( u_{zf} \) are the horizontal and vertical velocities of flow, respectively) in the vicinity of a single pipe and two tandem objects with various \( G/D \) is presented in Fig. 7. Herein, \( U_c = 1 \) m/s denotes the wave travels in the direction of the current with a velocity of 1 m/s. As shown in the figure, \( |u_f| \) above the upstream pipeline has its maximum value. It can be found that with \( G/D < 1.25 \), the maximum \( |u_f| \) above the upstream pipe for two tandem bodies is smaller than a single pipeline. That is mainly attributed to the flow being blocked by the existence of the downstream pipeline. On the other hand, as \( G/D > 1.25 \), the maximum \( |u_f| \) above the front pipe is larger than a single pipe. That implies the two parallel pipelines with the larger gap are more likely to be unstable compared with a single pipeline. Furthermore, when the wave approaches the pipeline, the flow through the gap between two tandem pipes is weak ascribed to the blockage of the upstream pipeline. As two pipes get farther apart, the \( |u_f| \) in the gap between the twin pipes rises slightly as the larger space allows more water to flow. Therefore, the increasing \( G/D \) of twin pipes can result in a non-negligible impact on their vertical stability.

Fig. 8 illustrates the distributions of the pore pressures in the vicinity of a single pipe and pipes in tandem with respect to the varying \( G/D \) when the wave trough travels near the structure. It is observed from the figure that with the increasing \( G/D \), the pore-water pressure beneath the pipelines decreases first and then rises. The reason for this phenomenon may be the effect of the gap on the seepage path and the velocity magnitude in the proximity of the upstream pipeline rises non-monotonously with the declining \( G/D \). When \( G/D \) is larger than 1.25, the seabed response around the twin pipes is more significant than a single pipeline. This can be explained by the reason that the presence of the downstream pipeline disperses the seepage route of pore water resulting in the alteration of the propagation of pore pressure energy transferred from wave-induced seafloor pressure. In addition, the difference between twin pipelines with \( G/D=3.25 \) and a single body becomes smaller, which illustrates the two pipelines gradually become independent.

Effect of currents on liquefaction potential

When a marine structure is included in the study, the initial stress state status is altered due to the body forces of the structure during the consolidation phase. The modified liquefaction criterion based on the mean normal effective stress proposed by Zhao et al. (2014) is expressed as:

\[
\sigma'_0 = \frac{\sigma'_{0o} + \sigma'_{0s}}{2} \leq p_s - p_w, 
\]

(14)
Fig. 8 Variations of the wave & current-induced pore-water pressure ($p_s$) under a single pipe and the different $G/D$ of two tandem pipelines when the wave trough travels near the structure.

where $\sigma'_{x0}$ and $\sigma'_{z0}$ define the effective stress in the $x$- and $z$- directions.

Fig. 9 Distributions of the liquefaction zone under a single pipe and twin pipes with different $G/D$ under the combined wave and steady current ($U_c=1\ m/s$) loading near the wave troughs at the specific time step.

As mentioned in the previous discussions, as the augment of $G/D$, the pore pressure under the pipelines declines first and then increases. Fig. 9 presents the distribution of the potential liquefaction zone under a single pipeline and twin pipes with different $G/D$ subject to the combined ocean wave and current ($U_c=1\ m/s$) loading. It is found that with $G/D < 1.25$, the liquefaction depths under the twin pipelines are slightly less than a single pipe. As for the two tandem pipes, the development of the soil liquefaction zone is marginally increased with the growing $G/D$. Notably, the range of the liquefaction zone is altered due to the variation of the distance between dual pipes.

Fig. 10 Distributions of the maximum liquefaction depth ($Z_L/D$) under a single pipe and the different $G/D$ of twin pipes for various $U_c$ near the wave troughs at the specific time step.

Under the variation of the ocean currents, Fig. 10 shows the distribution of the maximum liquefaction depths ($Z_L/D$) beneath a single pipe and the different $G/D$ of twin pipes. As seen, the maximum liquefaction depths around a single pipe ($Z_{L1}$) and twin objects ($Z_{L2}$) versus $G/D$ are plotted. Compared with the case of wave only, the results imply that the maximum liquefaction depth under the pipelines increases with increasing current velocity. This is because the interactions between the wave and the following current can improve the wave pressure at the seabed surface (Ye and Jeng, 2012), which will further impact the pore-water pressure within the seabed. As for the twin pipelines, a
positive relationship between $Z_l/D$ and $G/D$ is found. As listed results from Table 3, it is clearly found the amplitude of $Z_l/D$ under the two pipelines in tandem is lower than a single pipe. With a small distance between two pipelines, that means, the velocity of flow around the front pipe is very weak ascribed to the shield of the rear pipe. Furthermore, the value of $Z_l/D$ under the two pipelines decreases lightly when $G/D > 3.25$ in the present study, which depicts two bodies are more independent.

**Effect of seabed properties on liquefaction potential**

As reported in the literature (Jeng, 2003), the soil permeability ($k_r$) and degree of saturation ($S_r$) are the key factors that impact the wave and current-driven transient pore pressures. As shown in Fig. 11, the amplitude of $Z_l/D$ is inversely proportional to $k_r$ and $S_r$. More specifically, the effect of the $k_r$ on the seabed response is more sensitive than that induced by a variation of $S_r$ under the two pipelines or twin bodies. Similarly, at a relatively high $G/D$ ($G/D > 1.25$ in the present study), the $Z_l/D$ under the two pipelines is greater than in the condition of a single pipeline in isolation. When the gap between twin pipes is large enough, i.e. $G/D > 3.25$, the amplitude of $Z_l/D$ tends to be stable and gradually begins to reduce.

**Table 3 The results of twin and single pipelines with different $U_c$.**

<table>
<thead>
<tr>
<th>$U_c = 0$ m/s</th>
<th>$U_c = 0.5$ m/s</th>
<th>$U_c = 1$ m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>a single pipeline ($Z_{l2}$)</td>
<td>twin pipelines ($Z_{l2}$)</td>
<td></td>
</tr>
<tr>
<td>$G/D=0.25$</td>
<td>twin pipes with $k_r=5 \times 10^{-5}$ m/s</td>
<td>single pipe with $k_r=5 \times 10^{-5}$ m/s</td>
</tr>
<tr>
<td>$G/D=0.50$</td>
<td>twin pipes with $k_r=5 \times 10^{-6}$ m/s</td>
<td>single pipe with $k_r=5 \times 10^{-6}$ m/s</td>
</tr>
<tr>
<td>$G/D=0.75$</td>
<td>twin pipes with $k_r=1 \times 10^{-5}$ m/s</td>
<td>single pipe with $k_r=1 \times 10^{-5}$ m/s</td>
</tr>
<tr>
<td>$G/D=1.00$</td>
<td>twin pipes with $S_r=0.97$</td>
<td>single pipe with $S_r=0.97$</td>
</tr>
<tr>
<td>$G/D=1.25$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=1.50$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=1.75$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=2.00$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=2.25$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=2.50$</td>
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<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=3.00$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=3.25$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=3.50$</td>
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<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=3.75$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
<tr>
<td>$G/D=4.00$</td>
<td>twin pipes with $S_r=0.99$</td>
<td>single pipe with $S_r=0.99$</td>
</tr>
</tbody>
</table>

# denotes $\Delta Z_l = Z_{l2} - Z_{l1}$. 

CONCLUSIONS

In this study, the seabed response and predicted liquefaction under single and two tandem pipelines are investigated under interactions between wave and current through a 2D integrated OpenFOAM model. Based on the numerical examples, the following conclusions can be drawn:

(I) An overall good agreement between the previous experimental data (Zhai et al., 2022) and the present numerical model in a fully
buried condition demonstrates that the present model is reliable for the prediction of the wave-induced seabed response under the two parallel pipelines.

(2) The $|\nu_1|$ above the upstream pipeline has its maximum value. As $G/D$ increases, the two tandem pipelines become more independent and the velocity of flow around the pipelines rises, resulting in the instability of the structure.

(3) As the augment of $G/D$, the pore-water pressure beneath the two tandem pipelines decreases first and then rises, which may be attributed to the effect of the seepage path. The result becomes smaller when $G/D = 3.25$, which means the twin pipelines may be treated as two separated single objects.

(4) The maximum liquefaction depth in the proximity of the twin pipes increases slightly with the growing $G/D$. When $G/D$ is greater than 1.25 in the present study, the amplitude of $Z_\psi/D$ under the two tandem pipelines is larger than the single pipeline, which indicates the effects of the existence of the downstream pipeline and $G/D$ on the stability of the pipelines are non-negligible. When $G/D$ is larger than 3.25, the results beneath two parallel pipelines tend to be smaller, which illustrates the two bodies gradually become independent.

(5) The existence of ocean current can significantly affect the fluid-induced seabed responses beneath the two tandem pipelines. For the offshore pipelines, co-current waves ($+U_i$) tend to significantly induce liquefaction depth.

(6) The maximum liquefaction depth under either a single pipeline or twin bodies appears to increase as $k_i$ and $S_i$ decrease. The variation of $k_i$ is more sensitive to the evolution of the liquefied zone.

In this study, only oscillatory seabed response is considered. Regarding another mechanism of seabed response, residual liquefaction, the existing model will be further developed in the future.

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