Two-dimensional model of wave-induced response of seabed around permeable submerged breakwater

Richard Asumadu1,2,3, Ji-Sheng Zhang1,2, Osei-Wusuansa Hubert3 and Alex Baffour Akoto2

Abstract
This article focuses on a two-dimensional numerical model established to determine the seabed dynamic response in the region of a permeable submerged breakwater. The wave motion in this article is governed by the volume-averaged Reynolds-averaged Navier–Stokes equation, whereas Biot's poro-elastic equation determines the seabed foundation. The water surface is recorded using the volume of fluid technique. In this study, the results for the two-dimensional seabed dynamic response for both the consolidation status and the dynamic wave-induced response status for the seabed foundation coupled with submerged breakwater are illustrated. The numerical results examined from the dynamic pore pressure, the effective stresses, the shear stress, and the seabed soil displacements revealed that the impact of dynamic response at the offshore zone/seaward on the seabed foundation is more developed than at the onshore zone/harbor side. Parametric results analysis as regards the effect of the wave, the seabed, and the submerged breakwater structure variation significantly affected the seabed foundation response coupled with the breakwater structure. The numerical outcome on the liquefaction potential shows that the seabed foundation is more seemingly to liquefy and happen approximately at the toe of the submerged breakwater under the wave loading.

Keywords
Permeable submerged breakwater, volume-averaged Reynolds-averaged Navier–Stokes, Biot's poro-elastic equation, wave-induced seabed response, porous seabed foundation, two-dimensional numerical model

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Introduction
In the past two decades, there is increasing attention in the usage of permeable submerged breakwaters as a marine defense structure against strong waves. These permeable submerged breakwaters have a minor visual intrusion and esthetic advantage over other marine defense structures. They allow passage of sediment, thereby keeping the environment open and sheltered simultaneously. In the past few years, an immense attempt has been put to model the wave–seabed–structure interaction (WSSI) phenomenon either by the...
analytical, physical, or numerical method to ascertain the disagreement associated with these models. The major reason behind this increasing attention is that several offshore installation facilities, for instance, oil-tube, oil platform, jetty, and pier, have been broken or destroyed by the wave-induced seabed response in the environs of coastal structures as a result of excess pore pressure within the seabed foundation. Many examples of breakwater failure have been published in numerous research works.\textsuperscript{1–7}

In the past two decades, studies on analytical wave-induced interaction based on Biot’s consolidation\textsuperscript{8} theory have been conducted and published in Jeng and Cha.\textsuperscript{9} Numerous analytical solutions concerning wave-induced dynamic response on the infinite seabed under ocean wave loading were proposed.\textsuperscript{10–12} Later, Hsu and Jeng\textsuperscript{13} established analytical wave-induced response on the finite thickness of seabed based on the three-dimensional (3D) short-crested wave. Afterward, numerous studies concerning complex and complicated conditions relating to waves and seabed have been reported, for instance, cross-anisotropic soil behavior\textsuperscript{14,15} and inertial effect.\textsuperscript{16,17} Apart from the analytical solution, many research works on laboratory experiments on the wave-induced application have been published for years.\textsuperscript{18–20} One famous example is Mizutani et al.,\textsuperscript{21} in which they carried out an experimental laboratory study on the problem of wave–porous seabed–structure interaction. Their investigations on the wave-induced dynamic response were to analyze the wave effect characteristics, seabed effect characteristics, and structure dimensions within the seabed foundation. Most recently, Jeng et al.\textsuperscript{22} carried out a laboratory experiment to ascertain the interaction of the ocean waves, a submerged breakwater, a vertical seawall, and sandy seabed foundation. The purpose of the experiment was to read water height and the pore pressure within the seabed foundation. Their investigation results discovered a major change in the wave-induced pore pressure within the seabed due to the strong interaction of the incident wave, submerged structure, and the vertical wall. They also discovered that at the toe of the submerged structure, the wave-induced pore pressure was less significant compared to the submerged structure underneath. Subsequent from the analytical solution and the laboratory experiment development, numerical method application on wave-induced seabed response within the seabed foundation and the breakwater structure has been enormous. Numerical model in preceding years has been engaged as an effective tool for investigating the wave-induced seabed response. Numerical model in recent years has been advanced to consider the interaction between the waves and pore pressure in the porous medium by adopting the volume-averaged Reynolds-averaged Navier–Stokes (VARANS) equation to govern the wave motion and the porous flow in the porous channel according to Ye et al.\textsuperscript{23} Most recently, Jeng et al.\textsuperscript{24} carried out the integrated numerical model on associated problems on the WSSI, in which the VARANS equation was used as the determined equation for the porous flow in the porous channel for seabed foundation and the breakwater structure. Afterward, Zhang et al.\textsuperscript{25} also researched into the wave propagation around a porous seabed foundation in the region of porous breakwater setup using the VARANS and Biot’s poro-elastic equations. Their results established that the seabed response near the foundation in the environs of the permeable submerged breakwater is reliant on the processes of the wave propagation, that is, emanating from the interaction of waves, permeable structure, and porous seabed foundation. Their model proved inadequate in showing the two-dimensional (2D) wave-induced seabed response distribution for the pore pressure, the effective stresses, the shear stress, and the soil displacements within the porous seabed foundation coupled with the permeable submerged breakwater for both the consolidation status and the dynamic wave-induced response status. Ye et al.\textsuperscript{26} developed a 2D semi-coupled numerical model on fluid–structure–seabed interaction (FSSI) for validation, where they applied a section of the 2D wave-induced seabed response distribution to explain their application. Their model was based on coupling the Reynolds-averaged Navier–Stokes (RANS), VARANS equations, and the dynamic Biot’s equation together. They later developed a 3D semi-coupled numerical model of the WSSI,\textsuperscript{27} which also applied a section of the 2D distribution to explain their 3D application.

The purposes of these studies are to investigate the wave-induced seabed response in the submerged permeable breakwater coupled with the seabed foundation by applying the COBRAS\textsuperscript{28,29} and COMSOL Multiphysics\textsuperscript{30} simulation software to determine the 2D distribution for both the consolidation status and the dynamic wave response status. The results from the 2D numerical model are utilized to examine the parametric factors that influence the wave, seabed, and the permeable submerged breakwater characteristics. Finally, the results from the numerical model are engaged to identify the liquefaction potential zones within the porous seabed foundation. The results in the study are validated by comparing it with the experimental laboratory results from Mizutani et al.\textsuperscript{21}

**Methods**

In this article, the submerged breakwater and the wave sub-mode are being governed by the VARANS equation while Biot’s poro-elastic equation governed the seabed sub-mode. The waves were generated using the
second-order Stokes (SOS) wave. Spongy layers are applied on both lateral sides of the fluid channel to absorb the wave reflection.

**Seabed model**

The seabed foundation is a component of solid material (soil particles), liquid (water), and void (air). The soil particles form the skeleton for the seabed foundation, where the pores are filled by the water and air. In this article, the oscillatory wave response for soil, which is governed by Biot’s poro-elastic equations, is only considered. The determining porous equation can be denoted as

\[ \nabla^2 P_{sx} - \frac{\gamma_n n_{sx} \beta_{sx} \partial P_{sx}}{k_{sx}} \frac{\partial}{\partial t} = \gamma_n \frac{\partial}{\partial x} \left( \frac{\partial u_{sx}}{\partial x} + \frac{\partial v_{sx}}{\partial z} \right) \]  

\[ e_{sx} = \left( \frac{\partial u_{sx}}{\partial x} + \frac{\partial v_{sx}}{\partial z} \right) \]  

where \( P_{sx} \) is the pore pressure in Pa; \( k_{sx} \) is the permeability; \( \gamma_n \) is unit weight of pore water in kN/m³; \( u_{sx} \) and \( v_{sx} \) are the horizontal and vertical soil displacements in mm, respectively; \( n_{sx} \) is the soil porosity; \( e_{sx} \) is the volumetric strain; and \( \beta_{sx} \) is the compressibility of pore water, which is expressed as

\[ \beta_{sx} = \frac{1}{K_{sx}} + \frac{1 - S_{sx}}{P_{wo}} \]  

where \( P_{wo} \) is the absolute static water pressure in Pa; \( K_{sx} \) is the elasticity modulus of fluid taken as \( 2 \times 10^9 \) N/m², according to Yamamoto et al.; \( S_{sx} \) is the saturation. The stress–strain relationship that governed the equations for the force equilibrium in the soil is expressed as

\[ G_{sx} \nabla^2 u_{sx} + \frac{G_{sx}}{1 - 2\mu_s} \frac{\partial}{\partial x} e_{sx} = \frac{\partial P_{sx}}{\partial x} \]  

\[ G_{sx} \nabla^2 v_{sx} + \frac{G_{sx}}{1 - 2\mu_s} \frac{\partial}{\partial z} e_{sx} = \frac{\partial P_{sx}}{\partial z} \]  

where \( \nabla^2 \) is the Laplace operator; \( G_{sx} \) is the shear modulus of soil in N/m², which is related to \( E_{sx} \) (Young’s modulus) as well as \( \mu_s \) (Poisson’s ratio). For this study, the model adopted for the seabed is only limited to the case of the elastic seabed. The relationship bounded by them is also expressed as

\[ G_{sx} = \frac{E_{sx}}{2(1 + \mu_s)} \]  

The exchange, within the soil displacement and effective stress with respect to Hook’s law, is denoted as

\[ \sigma'_{x} = 2G_{sx} \left[ \frac{\partial u_{sx}}{\partial x} + \frac{\mu_s}{1 - 2\mu_s} e_{sx} \right] \]  

\[ \sigma'_{z} = 2G_{sx} \left[ \frac{\partial v_{sx}}{\partial z} + \frac{\mu_s}{1 - 2\mu_s} e_{sx} \right] \]  

\[ \tau_{xz} = G_{sx} \left[ \frac{\partial u_{sx}}{\partial x} + \frac{\partial v_{sx}}{\partial z} \right] = \tau_{xz} \]  

where \( \sigma'_{x} \) and \( \sigma'_{z} \) are the horizontal and vertical effective stresses in Pa, respectively. \( \tau_{xz} \) is the shear stress in Pa. The porous seabed foundation model was developed using a finite element method (FEM) software known as COMSOL Multiphysics.

**Wave model**

Many researchers have tremendously played a role in researching into wave motion and its characteristics for all these years. The wave model in this article is governed by the VARANS equation, which is derived from the RANS equation. The equations are denoted as follows:

**Continuity**

\[ \frac{\partial (u_{xi})}{\partial t} = 0 \]  

**Momentum**

\[ \frac{\partial (u_{xi})}{\partial t} + \frac{\langle u_{xi} \rangle}{n(1 + C_{Di})} = \frac{1}{1 + C_{Di}} \left[ \frac{n}{\rho} \frac{\partial <P_{sx}>}{\partial x} - \frac{\partial \langle u_{xi}u_{xi} \rangle}{\partial x} + \frac{1}{\rho} \frac{\partial \langle \tau_{xij} \rangle}{\partial x} - n g_{xi} \right] \]  

where \( u_{xi} \) is the velocity of flow in m/s; \( x_i, x_j \) are the Cartesian coordinates; \( t \) is the period in s; \( \rho \) is the water density in kg/m³; \( P_{sx} \) is the pressure in Pa; \( \tau_{xij} \) is the viscous mean flow; \( g_s \) is the acceleration due to gravity in m/s²; \( n \) is the porosity; and \( d_{s0} \) is the mean diameter of sand particle in mm.
From equation (11), $C_{Di}$ is the added mass coefficient, where the coefficients $\alpha$ and $\beta$ are identified on the linear and non-linear drag force, respectively, according to Liu et al.\textsuperscript{33} The turbulent flow influence is denoted by $\langle u_{xi}, u_{xj} \rangle$, which is derived by determining the $k-\varepsilon$ turbulence model, where $k$ is the kinetic energy in $N$ and $\varepsilon$ is the dissipation rate of kinetic energy in $N$.\textsuperscript{29} The angle bracket symbol \langle '' \rangle is termed as Darcy’s averaging operator (DAO) and is denoted as

$$\langle a \rangle = \frac{1}{V} \int_{V} a \, \text{adv} \quad (12)$$

where $V$ is the total volume, and $v_j$ is the section of $V$ that is subjected by water. The wave generation and wave flow are studied in the COBRAS model,\textsuperscript{33,35} adopting the finite difference method (FDM) for its discretization.

**Boundary condition**

In the determination of the VARANS equation in the fluid channel, acceptable conditions need to be assigned for the boundary. In the mean flow section, the application on the seabed foundation coupled with the permeable breakwater has zero slip boundary condition, whereas the issue of the air-movement is ignored. In the free surface area, tangential velocity stress, effective stress, and normal stress on the gradient kinetic energy of the fluid ($k$) in addition to the dissipation rate ($\varepsilon$) are set to zero. To avoid reflection on both sides of the fluid, spongy layers are mounted to absorb the waves.

The wave-induced response for the seabed also required suitable boundary conditions on the seabed foundation plane. On the seabed foundation plane, the effective vertical stress, likewise the shear stress, is ignored ($\sigma_z = 0$, $\tau_{xz} = 0$). The pore pressure $p_{sx}$ and shear stress $\tau_{xz}$ are equated to the wave pressure and the shear stress derived from the wave model ($p_{sx} = P_w; \tau_{xz} = \tau_w$), respectively. There is continuous pore pressure at the coupling plane of the porous seabed foundation, submerged breakwater, and the ocean wave. The underlying layer of the porous seabed foundation is assumed to be impervious and rigid with zero displacements, as well as the absent vertical flow for the two lateral sides far away.

**Verification**

Numerous laboratory experiments were engineered by Mizutani et al.\textsuperscript{21} on the permeable submerged breakwater as shown in Figure 1. A permeable submerged structure was constructed on a sandy porous seabed foundation. Four number of wave gauges were mounted to read the wave profile, two at the offshore zone/seaward and two at the onshore zone/harbor side, respectively. Four pressure sensors (transducers) were used to collect readings on the pore pressure within the seabed foundation coupled with the submerged structure. A pore pressure sensor (A) was installed into the submerged breakwater, whereas the remaining three transducers B, C, and D were placed in the seabed foundation as shown in Figure 1.

As shown in Figure 1, a numerical wave tank with dimensions same as in Mizutani et al.\textsuperscript{21} is adopted for this article. In the wave model, a 2m length wave absorbing sponge layer is applied at both vertical walls of the numerical computational domain. The internal wave maker is positioned at the far left-hand side of the domain to generate the waves.\textsuperscript{33} To avoid the wave reflection, the submerged breakwater is located at a distance as far as the incident wavelength from the wave maker. Within the computational medium, the domain sizes used for this model is 16 m $\times$ 0.4 m. A uniform grid system of $\Delta x = 0.01$ m is used along the x-direction and $\Delta y = 0.002$ m along the y-direction.

Respective properties for the seabed foundation and the submerged structure and the wave characteristics properties are presented in Tables 1 and 2, respectively.
In the numerical simulation processes, the seabed foundation and the permeable structure are considered as two distinct porous structures within the wave model. The data exchange point is applied by integrating the function at the boundary surface between the solid medium (porous seabed, permeable submerged breakwater) and the fluid mode. Figure 2 displays the plotted agreement for wave profile between the present simulation results and experimental laboratory results of Mizutani et al.,21 where Figure 2(a) and (b) shows good results, while some small differences are observed in Figure 2(c) and (d), which are located at the onshore zone/harbor side. Figure 3 also displays the plotted agreement between the present simulation results and experimental laboratory results of Mizutani et al.,21 for the pore pressure readings from the pore pressure sensors at A, B, C, and D, at which all the results are excellent with a minor difference in boundary element model (BEM)-FEM from Mizutani et al.21 The correlation results show that the integrated numerical model is applicable to the WSSI problems.

**Results and discussion**

**Velocity field**

Figure 4 indicates the velocity field under the wave action as it interacts with permeable breakwater. As the waves approach the breakwater, it is clearly observed that the waves are reflected by the breakwater surface, thereby reducing the energy and flow movement in the permeable breakwater.

Furthermore, it is noted that vortex field at the lee side of the breakwater is far limited. This is due to energy reduction and partial blockage of the cross-sectional pores from the permeable breakwater. Figure 4 also clearly depicts the flow velocity in the fluid domain decreases in a vertical direction with increase in the water depth, while flow velocity above the permeable breakwater shows a higher magnitude than the adjacent areas.

**Consolidation status**

It is known that once a breakwater, jetty, or any marine structure is developed on the seabed foundation, the seabed foundation changed in volume in response to the imposed pressure. In the ocean environment, wave loading and self-gravity are primary factors that influence seabed consolidation. Subsequently, once a breakwater is constructed on the seabed foundation, the seabed soil underneath the breakwater will be compacted and distorted under the gravity of the breakwater.23,24,35 The consolidation status of the seabed foundation is termed as the initial state or equilibrium state. To accurately simulate the WSSI, the initial consolidation state due to static wave loading has to be completely simulated before the dynamic loading from the wave is applied.24

Figure 5 shows the simulated distribution results of the pore pressure (P) and the effective stresses (σ'x, σ'z) within the seabed foundation coupled with the submerged structure. The following phenomena are observed in Figure 5. (1) The pore pressure (P) shows that the profile patterns in the seabed foundation are arranged in coherence with the distribution of the water pressure. (2) According to the effective stresses (σ'x, σ'z) pattern, the magnitude within the seabed foundation underneath the submerged structure increased largely. Further observations also showed a thin firm sheet of horizontal stress exists at the base zone of the submerged structure. This thin sheet appearance can be concluded to mean that the seabed foundation is much brittle. This behavior can lead to the deformity of the breakwater. (3) The shear stress (τxz) is observed to focus at the slanted edges of the submerged breakwater, while within the seabed foundation is much centered at the lower bottom.

Table 1. Porous seabed and the permeable submerged breakwater properties.

| Items                  | Symbols | Permeable breakwater | Porous foundation |
|------------------------|---------|-----------------------|-------------------|
| Porosity               | n and n₀ | 0.3                   | 0.4               |
| Permeability (m/s)     | k and k₀ | 10⁻⁵                  | 10⁻³ or various   |
| Shear modulus (N/m²)   | Gₓ      | 10⁹                   | 5x10⁸             |
| Poisson’s ratio        | μₓ      | 0.28                  | 0.33              |
| Mean size of rubble (m)| d₅₀     | 0.027                 |                   |
| Saturation             | Sₓ      | 0.98                  | 0.98              |

Table 2. Wave characteristics properties.

| Items          | Symbols | Wave period (T) | Wave height (H) | Water depth (d) | Relative density (Dₑ) |
|----------------|---------|-----------------|-----------------|-----------------|-----------------------|
|                |         | 1.4 s           | 0.03 m          | 0.3 m           | 0.38                  |

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![Table 1. Porous seabed and the permeable submerged breakwater properties.](image-url)

![Table 2. Wave characteristics properties.](image-url)
result, the sand particles under the breakwater move toward the lateral side under the compression of the overburden breakwater. In the case of the vertical displacement ($V_d$), the submerged structure is observed to subside downward in the consolidation processes as shown in Figure 6. It is also noted that the magnitude for both the seabed displacements under the consolidation status is minimal and approximately equal to zero.
Dynamic wave-induced responses
To achieve the dynamic wave-induced dynamic response of the seabed foundation and the submerged breakwater in the integration processes, the extracted wave pressure from the wave is exerted on the seabed foundation coupled with the submerged structure in the seabed model at the boundary interface. In this study, under the dynamic wave loading, the following wave characteristics are used $H = 0.03$ m, $d = 0.3$ m, and $T = 1.4$ s.

Figure 7 shows the simulated distributions of the pore pressure ($P$) and the effective stresses ($\sigma_x^*, \sigma_z^*$) with the shear stress ($\tau_{xz}$) within the seabed foundation coupled with the submerged structure at $t = 10.04$ s and $t = 12.2$ s. The following occurrences are observed at Figure 7 for the wave-induced dynamic pore pressure ($P$). (1) The wave profile underneath the wave trough is negative although positive underneath the wave crest. (2) The wave-induced pore pressure effect observed at offshore zone/seaward side ranging 4.0 m < $x$ < 4.7 m shows that the pore pressure is more dominant. This is as a result of the reflection and the high energy dissipation taking place within the seabed foundation coupled with the submerged structure, thus having a magnitude value between $-80$ and $-120$ Pa. (3) At onshore zone/harbor side of the seabed foundation ranging 5.7 m < $x$ < 6.3 m (both area under wave trough) for $t = 12.2$ s, the outcome of the wave-induced dynamic pore pressure magnitude is far limited, mainly between $-40$ and $-80$ Pa; this is due to the partial blockage and dissipation by the submerged structure on the waves, thereby showing the capability of the submerged breakwater as a defense structure.

In Figure 7, the following happenings are observed on the effective stresses ($\sigma_x^*, \sigma_z^*$) and shear stress ($\tau_{xz}$). (1) The effective horizontal stress is compressive and the effective vertical stress ($\sigma_z^*$) is tensile beneath the wave trough. On the contrary, effective horizontal stress ($\sigma_x^*$) is tensile and effective vertical stress ($\sigma_z^*$) is compressive beneath the wave crest. The reason for this is that the wave trough disintegrated the seabed and breakwater underneath it, while the wave crest consolidated or compacted the seabed and breakwater underneath it. (2) The wave-induced shear stress is observed to mostly concentrate at the lower base of the porous seabed foundation and the permeable submerged breakwater.

Figure 3. Correlation between the present simulation and the experimental results: wave induced at (a) section A (within the breakwater), (b) section B (offshore zone), (c) section C (beneath the breakwater), and (d) section D (onshore zone).

Figure 4. Velocity field under the wave action.
Figure 8 illustrates the simulated distributions for the soil displacement ($U_d, V_d$) within the seabed foundation coupled with submerged structure. It is noted that the magnitude of horizontal displacement ($U_d$) and vertical displacement ($V_d$) is small beneath the wave crest and the wave trough.

**Parametric study**

**Wave characteristics.** In the assessment of the wave-induced response within the seabed foundation, the wave characteristic performs a significant function according to Hsu and Jeng. The following wave characteristics are considered for this exercise, water depth $d = 0.3$ m, wave height $H = 0.03$ m, and wave period $T = 1.4$ s. The wave period ($T$) and the wave height ($H$) parameters are examined under this example.

**Wave period.** Figure 9 show the transformation of waves results from the collected wave periods over a submerged breakwater at $t/T = 1.2$. The wave periods applied in this study are $T = 1.0$ s, $T = 1.2$ s, and $T = 1.4$ s with established $H = 0.03$ m and $d = 0.3$ m. The outcome from the diagram shows that longer wave period results in a longer wavelength. It is also observed that wave crest with a longer height developed in front of the submerged structure as a result of the dissipation from the structure. Example of wave period $T = 1.4$ s, from the results, presented exhibited a longer wavelength and higher wave crest.

Figure 10 shows a pore pressure vertical distribution within the seabed foundation for the three wave periods at two specific locations, $x = 4.78$ m and $x = 6.67$ m. According to the illustrated diagram in Figure 9, wave
period $T = 1.0 \text{ s}$ reduced the wavelength and caused relatively higher depth with low magnitude. That is, the basis why higher wave period, for example, $T = 1.4 \text{ s}$, will produce a higher magnitude of wave-induced pore pressure within the seabed foundation as presented in Figure 10. Observations from the presented figures also show that maximum pore pressure in the vertical distribution occurred at location $x = 4.78 \text{ m}$, that is, in front of the submerged breakwater. In summary, the wave-induced pore pressure magnitude increases with respect to the wave period.

Wave height. Wave height is another phenomenon that impacts the transformation of the wave on the seabed foundation. Figure 11 shows wave transformation, resulting from the wave height over a submerged breakwater at $t/T = 1.2$. The three waves’ heights used for this exercise are $H = 0.01 \text{ m}$, $H = 0.02 \text{ m}$, and
$H = 0.03$ m with an established $T = 1.14$ s and $d = 0.3$ m. The presented results from the diagram indicate that when wave height increased, the wave crest increased as shown in Figure 11. The results also showed that the wave height increased when passing over the submerged structure as the structure’s existence partially blocked the narrow pores by which the waves flow passes, but decayed at the onshore zone/harbor side due to dissipation from the submerged structure. Example of wave height $H = 0.03$ m, from the results, depicted an increased wave crest and higher wave crest when flowing over the submerged breakwater.

Figure 12 presents the oscillatory pore pressure vertical distributions for the wave height located at two different locations within the seabed foundation, $x = 4.78$ m and $x = 6.67$ m for three wave heights. It is established that the wave force and the wave energy exerting on the submerged breakwater are precisely affected by the wave height, thus also affecting the pore pressure and the effective stress within the ocean seabed foundation. The maximum pore pressure vertical distributions for the wave height as presented in Figure 12 occurred at location $x = 4.78$ m, thus offshore zone/seaward of the seabed foundation. According to the presented figures, the maximum distribution of the wave-induced pore pressure magnitude within the seabed foundation also increased up to 35% as the wave height increased from $H = 0.01$ m to $H = 0.03$ m.

**Effect of seabed characteristics.** The seabed soil characteristics are primary significant parameters for wave-induced pore pressure within the seabed foundation apart from characteristics of waves. The following soil characteristics are considered for this study, soil porosity $n = 0.4$, soil permeability ($k_s$) is $10^{-3}$ m/s, seabed thickness ($H_s$) is 0.19 m, Poisson’s ratio ($\mu_s$) is 0.33, shear modulus $G$ is $10^8$ N/m$^2$, and soil saturation ($S_r$) is 0.98. The influences of soil porosity and permeability characteristics are analyzed in this study.

Porosity. To determine the soil porosity characteristic, an established $T = 1.4$ s and $H = 0.03$ s flow at a water depth of $d = 0.3$ m are applied on the seabed foundation coupled with submerged breakwater. Soil porosity of the seabed foundation is taken as $n = 0.3$, $n = 0.4$, $n = 0.3$,
and \( n = 0.5 \) at two different locations at \( x = 4.78 \) m and \( x = 6.67 \) m as presented in Figure 13. The diagrams show that increase in the soil porosity of the seabed decreased/weakened the wave propagation. For example, seabed porosity of \( n = 0.5 \) provides more and larger voids in the seabed foundation, thus leading to higher permeability in the seabed foundation. It is also noticed that seabed porosity at location \( x = 4.78 \) m is higher due to the high energy dissipation at that point, although in reality porosity increases with permeability.

Permeability. Permeability is one of the important soil parameters that affects wave-induced soil dynamics. It is established that as seabed sediments undergo consolidation due to the self-weight of the overburden, the permeability of the seabed soil decreases.\(^{36,37}\) In this study, the permeability is assumed to be variable. The seabed parameters for the anisotropic seabed include the permeabilities \( K_x, K_y, \) and \( K_z \). Figure 14 demonstrates the vertical distribution of maximum pore pressure \((p)/p_o\) versus \( z/h \) for three different values of soil permeability (\( K_x = 0.2, K_z = 0.3 \), and \( K_z = 0.05 \)). Based on the figure shown, we can conclude that seabed permeability is the critical parameter that affects liquefaction potential. Consequently, it is challenging for pore pressure to build up in seabed sand with high permeability. In conclusion, seabed sand has high permeability, for instance, coarse grain sand is rare to liquefy under wave loading as a result of the dissipation of pore pressure.

Effect of structure characteristics. The presence of the breakwater structure in the wave-seabed model plays a tremendous role in the wave-seabed transformation effect. The breakwater structure partly or wholly blocks the narrow pores, by which the wave moves and naturally decays due to dissipation from the structure–wave interaction on the incident waves.\(^{25}\) The breakwater structure parameters examined in these studies are structure width and structure height.

Structure height. In this study, the established water depth, the wave period, and the wave height are \( d = 0.03 \) m, \( T = 1.14 \) s, and \( H = 0.03 \) m, respectively. Three structure height variations \( SH = 0.15 \) m, \( SH = 0.20 \) m, and \( SH = 0.25 \) m are applied in this exercise. Figure 15 presents the wave transformation results from breakwater structure height variation at \( t/T = 1.2 \). The simulation results show that lower breakwater structure height apparently leads to higher submerged water depth, indicating lower dissipation and reflection rate of the incident energy wave. In contrary, higher breakwater height results in higher dissipation and higher reflection of the incident energy wave, thereby lowering the submerged water depth. Example of breakwater structure height, \( H = 0.15 \) m,
from the results, shows a higher submerged depth, thereby generating a smaller wave-induced response.

**Structure width.** Figure 16 shows the comparison of three different variations of breakwater structure width $SW = 1.90$ m, $SW = 2.10$ m, and $SW = 2.30$ m on wave transformation over submerged structure at $t/T = 1.2$. Figure 16 indicates that increase in the breakwater structure width (elongated the crest width) apparently decreased the submerged water depth. This indication shows higher dissipation magnitude of the waves. However, higher submerged water depth has lower reflection of the waves in front of the submerged structure. Figure 16 also shows a higher deformation trend for both $SW = 2.10$ m and $SW = 2.30$ m after the waves passed over the submerged structure.

**Liquefaction potential**

Liquefaction potential is a phenomenon where the stable state of the seabed foundation behaves in the form of a solid fluid. It has generally been well established that seabed foundation becomes liquefied under the wave loading due to the excess pore pressure exerted on the seabed foundation when there is the passage of wave over it. The seabed foundation mostly liquefied when the effective stresses at that zone in the seabed foundation decreases or equal to zero ($\sigma_z' \approx 0$).
Liquefaction potential issues are a serious concern to coastal engineers, planners, and scientific researchers in the designing, planning, and execution of marine projects. The liquefaction potential proposed by Okusa\textsuperscript{39} is adopted for this exercise. It is denoted as 
\[ g_s/C_0 - g_w(h)/C_0 d \]  
which was later extended to 3D by Tsai et al.\textsuperscript{40}

\[ \frac{1}{3}(\gamma_s - \gamma_w)d(1 + 2k_o) - \frac{1}{3}(\sigma'_z + \sigma'_x + \sigma'_y) \leq 0 \]  
where \( k_o \) is the coefficient of lateral earth pressure at rest and is defined as \( k_o = \mu/(1 - \mu) \), in which \( \mu \) is Poisson’s ratio, \( d \) is the water depth, \( \gamma_s \) is the unit weight of sea sand, \( \gamma_w \) is the unit weight of fluid, \( h \) is the seabed thickness, \( d \) is the water depth, and \( \sigma'_z \) is the effective vertical stress. Technically speaking, the liquefaction criteria from Okusa\textsuperscript{39} indicated that the seabed foundation will liquefy if the dynamic effective vertical stress \( \sigma'_z \) is equivalent to the static effective stress.

However, equation (13) has limitation for cases where there are the built coastal structures on the seabed foundation. This is due to the inability to determine the effective vertical stress. Subsequently, in order to apply equation (13) on the built coastal structures, the Okusa\textsuperscript{39} equation is modified and is expressed as 
\[ (\sigma'_z)_{ini} \leq (\sigma'_z) \]  
where \( (\sigma'_z)_{ini} \) is the effective vertical stress at the initial consolidation state. Equation (15) presents the modified liquefaction criteria, which are applied to the porous seabed foundation with submerged breakwater constructed on it.

Figure 17 presents the results of liquefaction potential zones at \( t/T = 62 \) under the wave loading, by the application of the modified Okusa\textsuperscript{39} equation. According to the result shown, two liquefaction potential zones are established in the seabed foundation near the surface, which are located in the range of 4.2 m < \( x < 4.7 \) m and 6.8 m < \( x < 7.3 \) m, respectively.

The following occurrences concerning the results are noted:

1. The liquefaction potential zone in the range of 4.2 m < \( x < 4.7 \) m, the offshore zone/seaward of the seabed foundation, is bigger than the zone at the range of 6.8 m < \( x < 7.3 \) m, the onshore zone/harbor side of the seabed foundation. This
is due to higher wave-induced pore pressure occurrence at the offshore zone.

2. It is also noted that the liquefaction potential zones at the surface of the seabed foundation are near to the submerged structure foundation toe(s), and this can have a major effect on the foundation stability of the structure.

3. It is realized that there is no seabed liquefaction underneath the submerged structure, although the wave trough also acts in that zone. The reason behind this phenomenon is that the seabed underneath the breakwater structure is compressed by the gravity of the submerged structure and effectively shelters the seabed foundation from been liquefied. This phenomenon is called shielding effecting.\textsuperscript{41,42}

Conclusion

In this article, a 2D numerical integrated model is developed to investigate the wave-induced seabed response within the seabed foundation coupled with a submerged breakwater. The following conclusions are outlined based on the results of this article:

1. The present simulation results show a complete total coherence with the experimental laboratory results in both the wave and pore pressure measurement.

2. The results from the consolidation status concluded that the self-weight of the submerged structure has an enormous impact on the initial state of consolidation.

3. The result shows that effective stresses most largely affect the submerged structure in the dynamic wave-induced response, where, underneath the wave trough, effective horizontal stress is compressive and effective vertical stress is tensile. On the contrary, underneath the wave crest, effective horizontal stress is tensile and the effective vertical stress is compressive. This is due to the wave trough disintegrates seabed and breakwater structure underneath it, while the wave crest consolidates or compact the seabed and breakwater underneath it.

4. In the case of the breakwater structure, smaller breakwater structure height naturally leads to higher submerged water depth, indicating lower dissipation and reflection rate of the incident energy wave. In contrary, higher breakwater height results in higher dissipation and higher reflection of the incident energy wave, thereby lowering the submerged water depth.

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ORCID iDs

Richard Asumadu https://orcid.org/0000-0001-6107-7966
Ji-Sheng Zhang https://orcid.org/0000-0002-7089-3524

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