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Yuchen Wang
Bureau of Public Works of Shenzhen Municipality, Shenzhen, China, wobushiwangyuchen@live.cn

Erwin Oh
Griffith School of Engineering, Griffith University, Queensland, Australia.

Jiemin Zhan
Department of Civil Engineering, Sun Yat-Sen University, Guangzhou, China

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EXAMINING THE BEHAVIORS OF SANDY AND SILTY SEABED UNDER WAVE ACTIONS

Yuchen Wang[1, *], Erwin Oh[2, *], and Jiemin Zhan[3]

Key words: finite difference method, fine-grained seabed, sandy seabed, linear wave.

ABSTRACT

Coastal structures are always facing the threat of damage caused by different wave actions. A better understanding of different seabed behavior could effectively reduce the damage caused by waves. In this paper, a 2 Dimensional quasi-dynamic u-w-p model is developed to analyze the different behaviors of seabed composed of loose sand, dense sand and silt. In the u-w-p model, acceleration, velocity and displacement has been considered, and three important parameters: pore water pressure, effective stress and shear stress, are obtained from the model with Finite Difference Method (FDM) and applied to describe the general behavior of seabed consisting of various materials. The results indicate that denser and more uniform soil structure and lower permeability could highly increase the stability of seabed, which means a lower probability of having liquefaction or shear failure inside the seabed. In addition, the phase lag plays a more important role in loose sand seabed than in the other types of seabed. This paper presents a comparison study of wave induced stress variation in seabed between fine-grained soil and coarse-grained soil, and provides a view differing from some of the published literature on seabed.

I. INTRODUCTION

Most coastal structures are facing the threat and damage from the repeatedly scouring waves. Repeatedly wave action could potentially cause stress distribution variation in the seabed, and lead to further damage to the coastal structures. Considerable effort has been dedicated to the phenomenon of the wave-seabed-structure interaction (Yamamoto, 1977; Madsen, 1978; Hsu and Jeng, 1994; Wen and Wang, 2013). Previous research has considered seabed stability to be a major problem. However, most case studies adopted in previous research focus on sandy seabed. Therefore the core concern in this paper is about the behavior of seabed composed of different materials (sandy and silty seabed) under wave actions.

As indicated in published literature, two types of seabed failure have been widely analyzed, namely, liquefaction and shear failure. When the pore pressure becomes excessive with accompanying decrease in effective stresses, a sedimentary bed may move in either horizontal (liquefaction) or vertical directions (shear failure), which then leads to an instability of the seabed (Jeng, 1997b).

Herein, a u-w-p model (including vertical displacements, horizontal displacements, and pore water pressure, denoted as "u-w-p") based on quasi-dynamic condition is established to analyze the transient response of different seabed under wave loading. As to the u-w-p model, acceleration, velocity, and displacement terms are considered separately for both solid and fluid phases.

In this paper, the behaviors of sandy and silty seabed have been presented in order to clarify the behaviors of two different types of seabed and identify the leading influence factor of seabed stability during wave actions. Further a comparison study has been conducted between sandy seabed and silty seabed, which aims to report the difference between them. The numerical models have been developed based on conservation law, constitutive law, and linear wave theory. The model aims to examine the dynamic movement of seabed under different wave actions, and outputs the relevant result according to different conditions. The variation of pore water pressure, liquefaction potential, and shear stress will be mainly discussed. A better understanding of the seabed failure process could enhance the understanding of seabed stability and relative damage to the coastal structures. This will benefit the design of coastal structures, and reduce the potential damage caused by waves.

II. LITERATURE REVIEW

As to seabed instability analysis, the major studies could be divided into two different types: numerical analysis and experimental analysis (including both laboratory tests and field tests). According to these existing studies, there are two major types of failure mode in numerical seabed instability analysis.

Paper submitted 01/27/15; revised 06/20/15; accepted 12/31/15. Author for correspondence: Yuchen Wang (e-mail: wobushiwangyuchen@live.cn).
[1] Bureau of Public Works of Shenzhen Municipality, Shenzhen, China
[2] Griffith School of Engineering, Griffith University, Queensland, Australia.
[3] Department of Civil Engineering, Sun Yat-Sen University, Guangzhou, China.
[*] Theses authors contributed equally to this work.

One is shear failure, and another is seabed liquefaction. When the pore pressure becomes excessive with accompanying decrease in effective stresses, a sedimentary bed may move in either horizontal (shear failure) or vertical directions (seabed liquefaction), which then leads to instability of the seabed (Christian and Hirschfeld, 1974; Jeng and Hsu, 1996). Seabed instability can affect the offshore and coastal structure directly; once the seabed becomes unstable, the structure may collapse or be damaged permanently.

Studies of shear failure were commenced by Henkel (1970), and have been substantiated by several researchers (Wright and Dunham, 1972; Mitchell and Hull, 1974; Raham, 1991). Raham (1991) produced a comparative comprehensive summary of shear failure and indicated the influence of seabed material during the seabed shear failure process. Raham (1991) concluded that shear failure is more likely to occur in cohesive sediments, and un-cohesive seabed is unlikely to be unstable due to shear failure. As to seabed liquefaction, the earliest research could be traced back to Yamamoto (1977) or even earlier; however, presently most of the numerical researches in this area are following the basic path of Hsu and Jeng (1994) and Jeng (1997a). However, most of the research in this specific area is based on the sandy seabed (Zen et al., 1990; Jeng, 1997a; Zhang et al., 2011). Recently, some efforts have been focused on developing several new governing equations; for example, the Volume-Averaged Reynolds-Averaged Navier-Stokes (VARANS) equations are adopted as the governing equation in Zhang et al. (2011) and Zhang and Jeng (2013), which aims to provide a more accurate result than previous research. Experimental analysis is another important approach in the study of wave-induced seabed instability. Two typical experimental analyses are conducted by Feng (1992) and Chang (2006). Both have measured the pore water pressure, effective stresses and several other important factors which could be applied to establish and verify the numerical models.

Seabed liquefaction is a major topic in wave-induced soil analysis. Many different approaches have been conducted to find the result, such as using different assumptions (finite or infinite seabed thickness, saturated or unsaturated seabed and others), or applying different basic theory (Biot consolidation theory or conservation law). Table 1 is a brief summary of the milestones of the development in seabed instabilities analysis. The research is generally commenced by Henkel (1970). Yamamoto (1977) and Madsen (1978) developed their own governing equation, and the equation, resulting in what was called the "Yamamoto-Madsen equation", and was widely applied in their time. Then Hsu and Jeng (1994) developed another model based on their new governing equations, which could be applied to nonlinear waves and provide better results. In the next decade, a series of researches were conducted based on this. New governing equations have been applied to obtain accurate solutions in the last several years, like those of Zhang et al. (2011), and Zhang and Jeng (2013). Ever since, several laboratory tests have been presented to provide more experimental data and improve the existing governing equations and results.

Table 1. A short summary for the important literatures on seabed instability analysis.

Research	Key Feature	Methodology
Henkel (1970)	Shear failure is likely to occur at the toe of breakwater.	Complex mathematical method
Mitchell and Hull (1974)	Shear failure analysis	Numerical method
Yamamoto (1977) Madsen (1978)	A series of remarkable governing equations for seabed instability analysis based on linear wave conditions	Analytical analysis Numerical analysis
Raham (1991)	A relative comprehensive illustration for shear fail mechanism and process	Numerical analysis
Hsu and Jeng (1994) Jeng (1996)	A series of governing equations which could be applied to investigate both linear waves and non-linear waves	Analytical analysis Numerical analysis
Feng (1992) Chang(2006)	Provided a lot of important laboratory experimental data.	Experimental Analysis
Zhang et al. (2011) Zhang and Jeng (2013)	Developed more accurate models with new governing equations	Analytical analysis Numerical analysis

III. MODEL INTRODUCTION

The Governing Equations for Poro-elastic Material is established based on the following assumptions:

- Seabed is homogeneous, isotropic and flat surface.
- Linear elastic theory is assumed in all the constitutive models for stress vs. strain relationship.
- Flow behavior is also assumed linear with employing Darcy's Law between hydraulic gradient and pore water flow velocity.
- Sea wave and associated water pressure on seabed surface are given with: Linear wave theory.

The governing equation is expressed as simultaneous partial differential equations. The acceleration terms are neglected in quasi-dynamic analysis, the 2-D quasi-dynamic u-w-p model can be presented as below. The detailed deviation of the governing equation has been presented in Wang and Oh (2013).

[u-w-p] model; quasi-dynamic analysis

$$\begin{aligned} &-F\Delta u_{jj,i} - G(\Delta u_{i,jj} + \Delta u_{j,ij}) + \Delta p_{,i} = \rho_t \psi_i \\ &r_{ij}\Delta \dot{w}_j + \Delta p_{,i} = \rho_f \Delta \psi_i \\ &B_f(\Delta \dot{u}_{i,i} + \Delta \dot{w}_{i,i}) + \Delta \dot{p} = 0 \end{aligned} \tag{1}$$

Table 2. The parameters of soil applied in this paper.

Material Type	Loose Sand	Dense Sand	Silt	Note
ρ_t: bulk density of wet material (kg/m^3)	1.90×10^3	2.00×10^3	1.80×10^3	
ρ_s: density of solid phase (kg/m^3)	2.65×10^3	2.65×10^3	2.65×10^3	
n: porosity	0.454	0.394	0.515	
u_s: shear modulus of solid phase (N/m^2)	0.4×10^8	1.2×10^8	0.3×10^8	
ν_s: Poisson's ratio	0.30	0.30	0.30	
B': Skempton's B-value in 1-D	0.40	0.70	0.80	
k: coefficient of permeability (m/s)	1.0×10^{-4}	1.0×10^{-5}	1.0×10^{-6}	
e: void ratio	0.832	0.650	1.062	$e = n/(1-n)$
λ_s: Lamé's constants (N/m^2)	0.6×10^8	1.8×10^8	0.45×10^8	$E = 2(1+\nu)G$
E_s: Young's modulus of solid phase (N/m^2)	$1.04\times\times 10^8$	3.12×10^8	0.78×10^8	$E = 2(1+\nu)G$
E_{us}: stiffness in 1-D of solid phase (N/m^2)	$1.40\times\times 10^8$	4.20×10^8	1.05×10^8	$E = 2(1+\nu)G$
K_f: bulk modulus of fluid phase (N/m^2)	0.424×10^8	3.86×10^8	2.16×10^8	$E = 2(1+\nu)G$
B_f: averaged bulk modulus (N/m^2)	0.933×10^8	9.80×10^8	4.20×10^8	$E = 2(1+\nu)G$
S_r: degree of saturation of pore (%)	99.30	99.93	99.87	$Sr = (1/K_a - 1/K_f)/(1/K_a - 1/K_w)$
ρ_f: bulk density of fluid phase (kg/m^3)	9.930×10^2	9.993×10^2	9.987×10^2	$\rho_f = \rho_a(1+Sr) + \rho_w Sr$
ρ_d: bulk density of dry material (kg/m^3)	1.45×10^3	1.61×10^3	1.29×10^3	$\rho_d = (1-n)\rho_s$
ω_o: characteristic angular frequency (rad/sec)	4.48×10^4	3.87×10^5	5.06×10^6	$\omega_o = ng\rho_w / k\rho_f$
Maximum B' value	0.974	0.932	0.976	$B' = K_f/(K_f + nE_{us})$

Bulk modulus of saturated water, $K_w = 2.31 \times 10^9$ (N/m^2); bulk modulus of air, $K_a = 3.03 \times 10^5$ (N/m^2) $K_0 = 0.5$ (at rest); Bulk density of water, $\rho_w = 1000.0$ (kg/m^3); bulk density of air, $\rho_a = 0.0$ (kg/m^3)

This model combined in the governing equation is named the [u-w-p] model, where u represents the variation of horizontal displacement under wave actions, w means the variation of vertical displacement and p is the variation of the pore water pressure. In Eq. (1), B_f is the average bulk modulus of fluid phase; t is bulk density of wet soil; f is the bulk density of fluid; is the change of pore water pressure; i is dilatancy angle; and G is the shear modulus which is the same as u_s in Table 2.

Sometimes the model is simplified according to the appropriate assumptions associated with the different problems. For example, u-w-p model could be simplified into u-p model, where the relative acceleration of fluid phase is neglected; that is, the acceleration of fluid is taken to be equal to that of solid phase. If the effect of pore water flow is negligible, as in the problems where the ground is under high frequency wave impacts or in short term analysis, the seepage flow can be eliminated from the governing equation and the undrained condition can be assumed. In this case the model is called [u]-model; the relative velocity of fluid phase as well as the relative acceleration are neglected. If only the pore water pressure and the pore water flow are concerned, and the deformation of the solid phase is out of the scope of the analysis, the model can be simplified and the solid phase is assumed rigid. This model named [w-p] model is effective for the material with high permeability, such as coarse sand and gravel, and for the static condition. All these models will be presented in future papers, only the most complex and general model (u-w-p model) will be discussed in this paper.

Table 3. Uses of parameters in different conditions.

Condition	$\Delta\ddot{u}_i\ \Delta\ddot{w}_i$	$\Delta\dot{u}_i$	$\Delta\dot{w}_i\ \Delta\dot{u}_{fi}$	$\Delta\dot{p}$
Qusi-dynamic	–	+	+	+

"+" = considered, "–" = neglected

Table 4. Wave parameters.

Types of wave	H (m)	T (sec)	d (m)
Wave	10.0	13.0	20.0

IV. WAVE PARAMETERS AND SOIL PARAMETERS

The parameters used in different conditions is shown in Table 3. In Qusi-dynamic model, the second derivative of the displacement in all three directions has been considered, which makes the u-w-p model to be the most comprehensive one in describing the general behavior of seabed. Wave parameter and soil properties is shown in Table 2 and Table 3. The wave parameters in Table 4 are derived based on the linear wave theory, which assumes that the fluid layer has a uniform mean depth and the fluid flow is inviscid, incompressible and irrotational. Linear wave theory could describe the propagation of gravity waves on the surface of a homogeneous fluid layer, which is a well-developed theory in wave-structure-seabed interaction analysis. This linear theory is often used to get a quick and rough estimate of wave characteristics and their effects. It is accurate for small ratios of the wave height to water depth (Suo and Huang, 2004), like the wave condition applied in this study.

The aim of this paper is to illustrate the different behavior between silty seabed and sandy seabed, instead of analyzing the seabed behavior under different wave conditions. Therefore, only one group of typical wave parameters, which could clearly present the difference among varied soils, has been selected.

In the problem, the variable parameters, such as displacements, velocity and pressure, are represented by a function.

$$Displacement = a(z)e^{i(\omega t + \kappa x)} \quad (2)$$

And the function is solved as a kind of one-dimensional problem by finite difference method. The finite difference solution is derived exclusively for two-dimensional dynamic analysis; however, the solution obtained can be easily modified for the other dimensional conditions.

The mechanical property of applied soil is listed in Table 4, where the average Poisson ratio of fully saturated soil has been adopt for easier model establishment and faster model running. The 2-D u-w-p model in quasi-dynamic will be applied to conduct the analysis.

1. Boundary Conditions and Finite Differential Method (FDM) Solution of the Model

This section shows the stress parameters obtained from the governing equation under boundary conditions as shown in Eq. (3). The velocities and displacements in both vertical and horizontal directions have been considered. In addition, the pore water pressure and seepage velocity have been applied. The detailed finite differential solutions of the governing equation have been presented in Wang and Oh (2013). For the response of seabed to sea wave, the following boundary conditions must be satisfied, where is the change of pore water pressure; z is the depth of the seabed; is the vertical stress change; is the shear stress change; and are the displacement increment vectors in horizontal direction and vertical direction, respectively.

$$\begin{aligned}
&\Delta p(x, 0, t) = p_o e^{i(\omega t + \kappa x)}, && z = 0; \\
&\Delta \sigma_{zz}(x, 0, t) = 0, && z = 0; \\
&\Delta \sigma_{xz}(x, 0, t) = 0, && z = 0; \\
&\Delta \dot{w}_z(x, H, t) = -k \frac{\partial \Delta p}{\partial z}(x, H, t) = 0, && z = H; \\
&\Delta u_x(x, H, t) = 0, && z = H; \\
&\Delta u_z(x, H, t) = 0, && z = H;
\end{aligned} \quad (3)$$

Where the is the change of pore water pressure; is the initial pore water pressure at surface layer; is the relative water depth variation comes from the ratio of wave height to wave numbers; t is the wave propagation time; G is the shear modulus of solid phase and F is the shear modulus for fluid phase.

After applying the boundary conditions, the finite differential method solution of u-w-p model is shown from Eq. (5) to Eq. (7), where the p comes from Eq. (4). Eq. (4) is interpreted from the boundary conditions when applied

$$\Delta z = \frac{H}{N}$$

$$\Delta p(x, 0, t) = a_{p(0)} e^{i(\omega t + \kappa x)} = p_o e^{i(\omega t + \kappa x)}; \; a_{p(0)} = p_o \quad (4)$$

$$\Delta \sigma_{xx} = \left\{ -i\kappa (F + 2G) a_{ux(N)} - F \frac{3a_{uz(N)} - 4a_{uz(N-1)} + a_{uz(N-2)}}{2\Delta z} \right\} e^{i\kappa x} e^{i\omega t} \quad (5)$$

$$\Delta \sigma_{zz} = \left\{ -i\kappa F a_{ux(N)} - (F + 2G) \frac{3a_{uz(N)} - 4a_{uz(N-1)} + a_{uz(N-2)}}{2\Delta z} \right\} e^{i\kappa x} e^{i\omega t} \quad (6)$$

$$\Delta \sigma_{zx} = \left(-G \frac{3a_{ux(N)} - 4a_{ux(N-1)} + a_{ux(N-2)}}{2\Delta z} - i\kappa G a_{uz(N)} \right) e^{i\kappa x} e^{i\omega t} \quad (7)$$

V. GENERAL BEHAVIOUR OF SANDY AND SILTY SEABED UNDER WAVE ACTIONS

In this section, three main factors are employed to analysis the seabed behavior. They are pore water pressure, effective vertical stress, and shear stress. The comparison study between sandy seabed and silty seabed will be conducted based on these three factors along with seabed depth, which is from Z = 0 to Z = 3 m. The impact of cyclic waves on seabed only could be significant in shallow layers (Jeng, 1996), which analyzed the shallow seabed layer up to Z = 0.33 m. With the increase of seabed depth, the stability of deeper layers could not be influenced effectively under normal wave actions. Therefore, this study will focus on the shallow layer and middle shallow layers. The u-w-p model under quasi-dynamic 2-Dimensional condition is applied to introduce the seabed behavior in detail.

1. Pore Water Pressure

The pore water pressure variation to different types of seabed materials is shown in Fig. 1. The y-axis in Fig. 1 is normalized depth, which is depth against the total depth of the seabed, and the x-axis is the pore water pressure amplitude. From Fig. 1, it can be seen that as to the three different types of seabed, the pore water pressure decreases with the increase of seabed depth. The pore water pressure of sandy seabed consisting of loose sand has a 53.5% reduction from the surface to 3-m deep. However, as to the seabed composed of dense sand and silt, the reductions are only 4.8% and 6.2% respectively at the same depth of seabed. This phenomenon indicates two leading factors to the variation of pore water pressure in seabed; one is permeability, and another is the frictional effect inside the seabed material. Low permeability material will

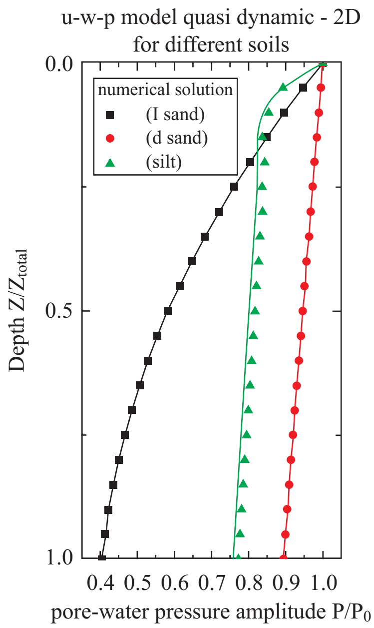

Fig. 1. Pore water pressure variation in different types of seabed.

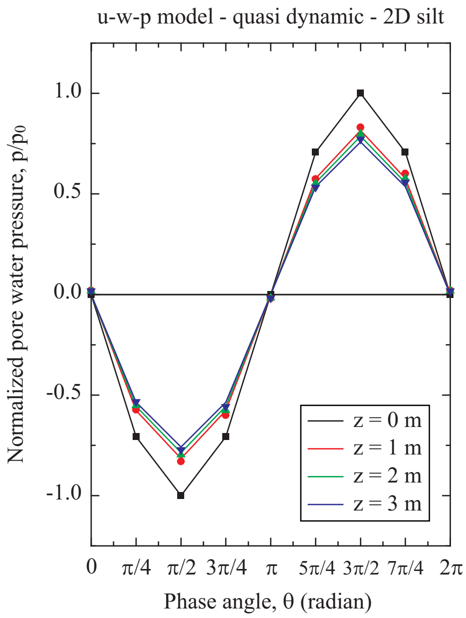

Fig. 2. Normalized Pore water pressure variation for silty seabed.

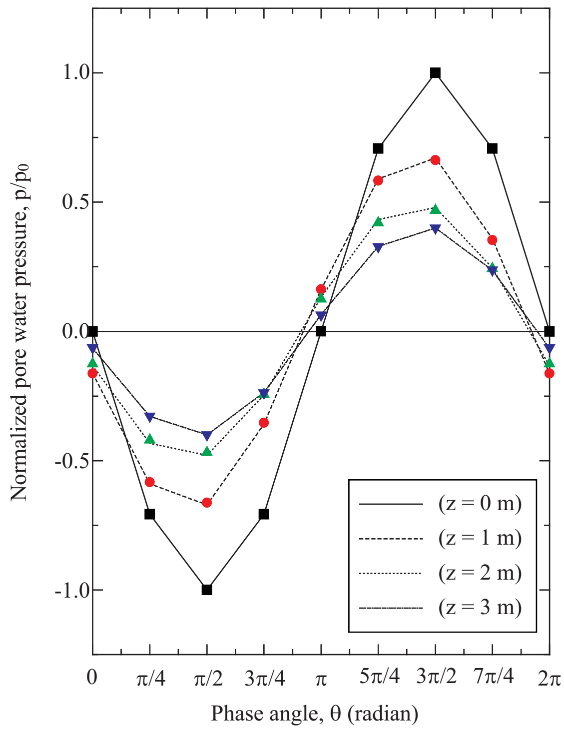

Fig. 3. Normalized Pore water pressure variation for sandy seabed.

impede the dissipation of pore water, and the frictional effect between solid and fluid phases in the seabed materials will also be taken into account in the analysis and is controlled by Darcy's law.

In addition, from Fig. 2 and Fig. 3, where the y-axis is the amplitude of pore water pressure and x-axis is the phase angle of the wave. The pore water pressure is normalized against the initial pore water pressure, which is the pore water pressure of soil before the application of wave action. It can be seen that pore water pressure of the seabed varies with propagation of the wave. As to both silty and sandy seabed, there is a jump of pore water pressure between Z = 0 (seabed surface) and Z = 1 m. This is due to the buoyancy on seabed surface and the weak combination of the soil in top layer. The soil in top layer does not combine with the seabed as a whole system, which makes the surface soil unstable and not fully consolidated. Consequently the soil is easily washed away by the waves. Therefore the surface layer cannot represent the general property of the seabed.

Along the depth (Z = 1 m to Z = 3 m), the normalized pore water pressure reaches the peak value around and, which represent the wave trough and wave crest respectively. Another important phenomenon that should be noticed here is that there is very limited phase lag in the silty seabed (Fig. 2). However, phase lag is very obvious in sandy seabed (Fig. 1). The critical pore water pressure occurs at the trough and crests in silty seabed. In sandy seabed, this happens before or after the application of critical waves. In addition, the pore water pressure could have a huge variation with respect to seabed depth. This will lead to a huge variation of effective vertical stress and highly increase the potential of liquefaction inside the seabed.

2. Effective Vertical Stress

Effective stress can be calculated as total stress subtracts pore water pressure. In reality, once the total stress equals the pore water pressure, the effective stress will be zero and the soil will lose its strength. Therefore, pore water pressure could affect the effective vertical stress directly. However, in this model, the "negative effective stress" is applied to illustrate the

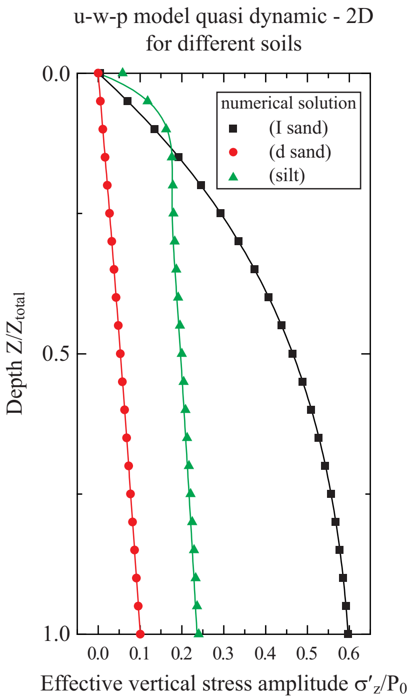

Fig. 4. Effective Vertical stress amplitude for different soils.

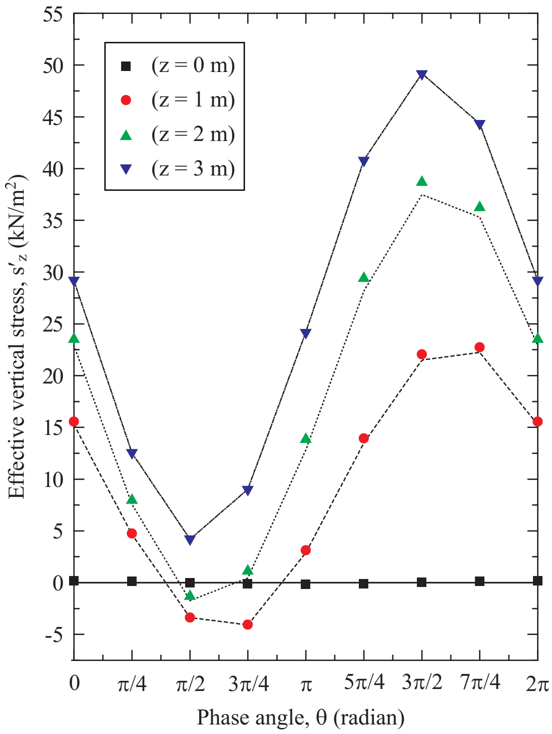

Fig. 5. Effective Vertical stress for loose sand seabed.

area where the excess pore water pressure could overcome the value of total stress. This will cause the seabed to be unstable and have a high potential to trigger the liquefaction of seabed.

The initial stress condition is calculated from the self-weight of the seabed and the coefficient of earth pressure at rest (K_o). Cyclic change in effective stress is caused due to cyclic change of the hydraulic gradient. Upward hydraulic gradient which is associated with upward seepage flow reduces effective vertical stress and downward hydraulic gradient increases the effective vertical stress. If the upward seepage flow is notable and exceeds a certain value, the effective vertical stress may become 'negative' in the model, which indicates that there is no adhesion between seabed particles.

The particles are in a general state of suspension. This condition is recognized as a typical type of liquefaction in seabed instability analysis. In analytical analysis, this condition also indicates that the pore water pressure is approaching to total stress, where the effective vertical stress (the difference between total stress and pore water pressure) could be reduced to zero and result in liquefaction.

As shown in Fig. 4, the amplitude of effective vertical stress for loose sand and dense sand is zero at Z = 0, but effective vertical stress for silt is slightly larger than 0. This is mainly due to the absorbed water layer upon the fine grained soil seabed, such as silty seabed. The fine grained soil could have attached to its surface a layer which absorbs water easily, and the absorbed layer is not free to move under gravity. It causes an obstruction to the flow of water in the pores and hence affect the pore water pressure and effective stress of the seabed.

The amplitude of effective vertical stresses in silty seabed increases with depth from 0 to 0.25. The increase of effective vertical stresses in dense sand seabed is a little bit smaller than that, which only reach 0.1 at Z = 3 m. The increase is almost linear to seabed composed of silt and dense sand, and the growth rate is relatively small. This is caused by the small increase of pore water pressure. It also indicates that stress distributions inside the silty or dense sand seabed are relatively uniform along the depth. The stress distribution and soil skeleton of these two types of seabed will not have a significant variation with the increase of depth. The uniform stress distribution will also highly reduce the probability of liquefaction.

However, the effective stress variation of loose sand seabed is obvious. The amplitude of effective stresses increase with depth from 0 to 0.6. The effective stress is associated with the friction between pore water and solid skeleton, too. With an increase of depth, the effective stress increases in both neutral value and amplitude. And the variation of effective stress tends to shift in the same direction as wave movement that is opposite to the variation of pore water pressure. Since the hydraulic flow is downward around the crest of the sea wave and upward below the trough of the sea wave, z′ is high (increases) below the crest and low (decrease) below the trough of wave.

The effective stress variation is evident in loose sand seabed; therefore the loose sand seabed has high risk in potential liquefaction, as shown in Fig. 5. The effective stress falls into negative in a limited part of shallow seabed when the range of phase angle of 2π/4 and 3π/4; as shown in Fig. 5. The negative value of effective stress occurs when sea wave is at the crest. This negative effective stress suggests the occurrence of cyclic

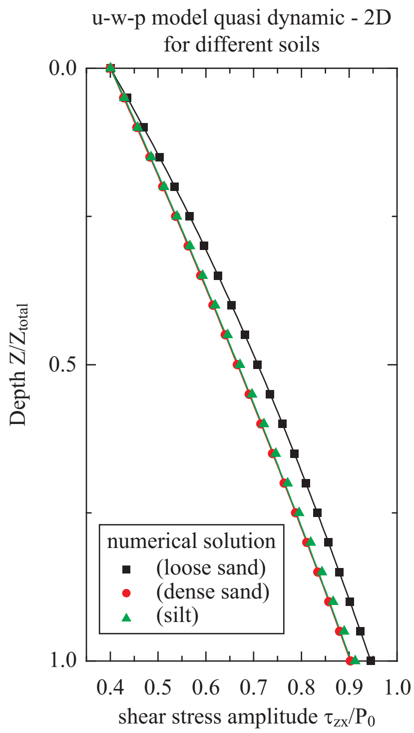

Fig. 6. Shear Stress variation for different seabed.

liquefaction. Liquefaction occurs up to the depth of 2.3 m and near the trough of sea wave and it tends to decrease with an increase of depth. The occurrence and intensity of liquefaction could be influenced by many factors, such as wave types and material types.

3. Shear Stress

The shear stresses variations are almost linear for all three types of seabed, as shown in Fig. 6, but the variation is very small and not going to have impact on seabed structure. If the shear stress is sufficient, it could possibly cause the seabed to liquefy and this kind of liquefaction is named as cumulative liquefaction. Pore water pressure may be generated due to the cyclic shear deformation induced by sea wave loading and accumulated after a series of sea wave.

The cumulative behavior of pore water pressure is a function of the soil type and cyclic stress conditions. For the prediction of the cumulative generation of pore water pressure and the potential cumulative liquefaction, a nonlinear constitutive model, which can take account of negative dilatancy properties of soils under cyclic loading conditions, must be combined in the appropriate analysis method.

VI. CONCLUSION

This paper presents a comparison study of wave induced stress variation in seabed with fine-grained soil and coarse-grained soil. The finite difference method (FDM) is applied in this study to provide a general view of the behavior of seabed composed of different material under wave loadings. In this paper, the u-w-p model under quasi-dynamic condition is mainly presented to simulate the exact seabed behavior under wave actions. According to the result, the following conclusions could be drawn:

- Both the effective stress and the pore water pressure vary linearly according to the depth. Due to the low permeability, this variation is not significant for dense sand seabed and silty seabed. The pore water pressure only has a 4.8% and 6.2% difference, respectively, from the surface (Z = 0 m) to the bottom (Z = 3 m). However, loose sand seabed could have more than 50% changes in pore water pressure, which makes the loose sand seabed easy to be unstable.
- Loose sand seabed is more likely to have liquefaction than silty seabed or dense sand seabed under same wave condition. In silt or dense sand seabed, the excess pore water pressure accumulates in a relatively low rate, and is hard to reach a massive value inside the seabed. This is due to their dense soil structure and low permeability; both the dense sand seabed and silty seabed have very low probability to reach liquefaction. Soil permeability is a measure indicating the capacity of the soil to allow fluids to pass through it. It is represented by the permeability coefficient (K) through the Darcy's Equation. Fine grain (e.g., silt or clay) soil has low permeability. Large void ratio for fine grain soil will potentially have large settlement. Low permeability of silt is due to the pores in fine grain soil are so small that water flows very slowly through them.

However, as to seabed composed of loose sand, under the same wave action, the liquefaction could occur at deeper depth, where the relative depth is 0.77 according to Fig. 1. It's larger than dense sand seabed. This indicates: dense structure and low permeability could reduce the hazard caused by seabed instability efficiently.

- The behaviors of loose sand seabed and dense sand seabed are quite different. The general seabed stability is decided by the structure of the seabed, the arrangement of the particles and the drainage condition.
- The variation of shear stress is almost linear as to the wave action, and it's not likely to have shear failure under the certain wave action. It also implies that the accumulated liquefaction or shear failure of seabed is more difficult to be triggered than liquefactions caused by the excess pore water pressure.

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