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Effects of Wave Non-Linearity on Residual Pore Pressures in Marine Sediments

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Abstract: A better understanding of the wave-induced pore pressure accumulations (i.e., residual pore pressure) is a key factor in the analysis of the wave-induced liquefaction in marine sediments. In this paper, the residual mechanism of non-linear wave-induced pore water pressure accumulation in marine sediments is examined. Unlike previous investigations, the second-order Stokes wave theory is considered in this study. The new model is verified with experimental data and provides a better prediction of pore pressure accumulation than the previous solution with linear wave theory. The parametric study concludes that the influences of wave non-linearity increase as the following parameters increase (i) wave steepness (H/L) and (ii) residual parameter (β). However, an opposite trend is found for (i) wave period (T), (ii) relative water depth (d/L), (iii) seabed thickness (h/L) and (iv) another residual parameter (α). Furthermore, the effect of wave non-linearity becomes more significant in soft seabed (Soil C).

INTRODUCTION

The evaluation of the wave-induced soil response in marine sediments is particularly important for coastal engineers involved in the design of foundation of many marine installations, e.g., offshore mono-piles, breakwaters, pipelines and platforms etc. The prediction of the wave-induced excess pore pressure is a key procedure in the analysis of seabed instability such as liquefaction and scour. Therefore, it is necessary to have a better understanding of the mechanism of the wave-induce pore pressure in marine sediments.

Two mechanisms for wave-induced pore pressure have been observed in the previous field measurements and laboratory experiments [1], as shown in Fig. (**1**), The first mechanism is resulted from the transient or oscillatory excess pore pressure and is accompanied by attenuation of the amplitude and phase lag in the pore pressure changes [2, 3]. The second mechanism is termed the residual pore pressure, which is the build-up of excess pore pressure caused by contraction of the soil under the action of cyclic loading [4].

Numerous studies for wave-induced momentary liquefaction, caused by oscillatory pore pressure, have been carried out since the 1970s. Among these, Yamamoto *et al.* [4] proposed a closed-form analytical solution for an infinite seabed. Mei and Foda [5] proposed a boundary-layer approximation to derive a rather simplified formulation for the wave-induced transient pore pressure, which is only valid for coarse sand [6, 7]. Jeng [3, 6-9] derived a series of analytical solutions for the oscillatory pore pressure within marine sediments. His models considered various soil behaviors, such as cross-anisotropic soil behaviors and variable permeability. Later, Kianoto and Mase [10] and Yuhi and Ishida [11] further suggested a new and simplified formulation for the wave-induced pore pressure in a cross-anisotropic seabed. An intensive review of the previous research for the wave-induced oscillatory pore pressure can be found in [12].

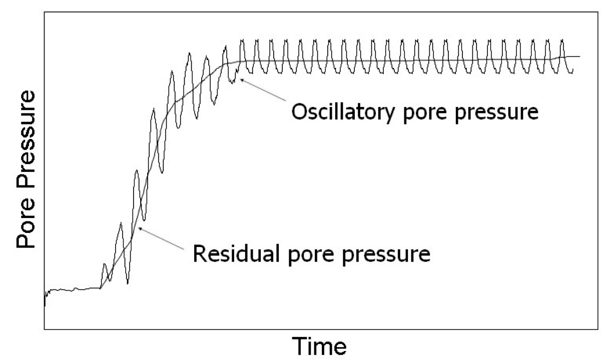

Fig. (1). Mechanisms of oscillatory and residual pore pressures in marine sediments.

The mechanism of pore pressure build-up due to ocean waves has been studied by many researchers since the 1970s. Seed and Rahman [4] established a simple one-dimensional finite element model by taking into account the distribution of cyclic shear stresses in the soil profile, and pore-pressure dissipation. Sekiguchi *et al.* [13] proposed an elasto-plastic model for the standing wave-induced liquefaction using a Laplace transformation. Later, some numerical models for post-liquefaction and progressive liquefaction and densification in marine sediments were developed [14].

In addition to numerical modeling, McDougal *et al.* [15] proposed a set of analytical solutions for wave-induced pore pressure build-up in a uniform layer of soil, based on the assumption of an incompressible soil. In their approach, the source term in the modified Biot's consolidation equation is derived using a linear relationship between pore pressure

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ratio (u_g / σ_0') and cyclic ratio (τ / σ_0') [16, 17]. To provide a convenient practical result for engineers, McDougal *et al*. [15] proposed three solutions for the cases of shallow, finite and deep soil depths, respectively. These analytical solutions are useful for both engineers and researchers, as they can be used for either the investigation of qualitative behaviors of complicated engineering problems or the validation of numerical methods. Recently, using a similar approach, Cheng *et al*. [18] re-examined the analytical solution of McDougal *et al*. [15] and proposed a numerical model to investigate the same problem. As pointed out by Cheng *et al*. [18], the analytical solution proposed by McDougal *et al*. [15] revealed some errors in the formulations. However, after a close examination of both the McDougal *et al*. [15] and Cheng *et al*. [18] solutions, the author found numerous errors in both publications, as reported in [19-21].

Recently, a series of analytical approximations for the wave-induced accumulated pore pressure in marine sediments have been proposed by the author [19-21]. Both cases of infinite and finite soil layers were considered in the models. A simplified universal formula was proposed for the case of infinite seabed [19]. However, all these approximations were limited to linear regular wave loadings, although it should be non-linear waves in natural environments.

In this paper, the models developed by the author [19-21] are further extended to non-linear wave loadings. The new non-linear wave model will be verified by comparing the previous experimental data, together with the previous linear wave model. Based on the present model, we will examine the effects of wave steepness and residual parameters on the wave-induced accumulated pore pressure in a porous seabed.

THEORETICAL FORMULATIONS

Non-Linear Wave Theory

Herein, a soil matrix subject to a two-dimensional progressive waves system is considered, as depicted in Fig. (**2**). The wave crest propagates in the positive *x*-direction, while the *z*-axis is positive upward from the *seabed surface*, as shown in Fig. (**2**).

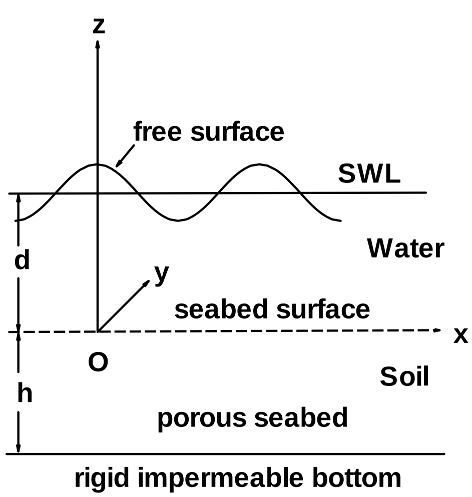

Fig. (2). Sketch of wave-seabed interaction.

Referring to a wave theory to the second-order [22], the velocity potential (ϕ) is given as

$$\phi = -\frac{gH}{2\omega}\frac{\cosh kz}{\cosh kd}\sin(kx-\omega t) - \frac{3\omega H^2}{32}\frac{\cosh 2kd}{\cosh 2kd}\sin 2(kx-\omega t) \tag{1}$$

where H is the wave height, d is water depth, g is the gravitational acceleration and t is the time. The wave number (k) and wave frequency (ω) can be determined with the following wave dispersion relation:

$$\omega^2 = gk\tanh kd \tag{2}$$

The water surface displacement to the second-order (η) can be expressed as

$$\eta = \frac{H}{2}\cos(kx-\omega t) + \frac{kH^2}{16}\frac{\cosh kd}{\sinh^3 kd}(2+\cosh 2kd)\cos 2(kx-\omega t) \tag{3}$$

and the dynamic wave pressure (P_b) at the seabed surface (*z=0*) can be written as,

$$P_b(x,t) = P_{w0} + P_{w1}\cos(kx-\omega t) + P_{w2}\cos 2(kx-\omega t) \tag{4}$$

$$P_{w0} = -\frac{\gamma_w kH^2}{8\sinh 2kd}, \quad P_{w1} = \frac{\gamma_w H}{2\cosh kd}$$

$$P_{w2} = \frac{3\gamma_w kH^2}{8\sinh 2kd}\left(\frac{1}{\sinh^2 kd} - \frac{1}{3}\right) \tag{5}$$

Since this study focuses on the dynamic wave pressures, the first term on the left hand side of (4) is excluded here.

Boundary Value Problem for Seabed

The Biot consolidation equations [23] have been generally adopted to model the dynamic response of marine sediments for various applications. In general, the wave-induced pore pressure within marine sediments consists of two components: oscillatory ($\tilde{p}$) and residual (u) mechanism, which can be expressed as

$$p(x,z;t) = \tilde{p}(x,z;t) + \bar{p}(z,t) = \tilde{p}(x,z;t) + u(z,t) \tag{6}$$

where p is the pore water pressure, $\tilde{p}$ represents the oscillatory pore pressure, which leads to momentary liquefaction, while $u = \bar{p}$ represents the period-averaged pore pressure, which leads to residual liquefaction, and defined by

$$u = \frac{1}{T}\int_t^{t+T} p\,dt \tag{7}$$

where T is the wave period and t is the time.

A series of analytical solutions for the linear wave-induced oscillatory pore pressure within marine sediments have been developed since the 1970's [2].

In this study, referring to (4), the second-order dynamic wave pressure can be expressed in three terms with different wave frequencies. Thus, the wave-induced seabed response variegates can be expressed in three terms:

$$SP(x,z,t) = \sum_{m=1}^{2} SP^{(m)}(z)\cos(k_m x - \omega_m t) \tag{8}$$

where $k_m = mk$ and $\omega_m = m\omega$ and SP denotes soil response variables such as pore pressure and stresses etc.

The amplitude of the oscillatory pore pressure ($\tilde{p}_0^{(m)}$) and shear stress ($\tilde{\tau}_0^{(m)}$) for each component in a saturated seabed are given by [3]:

$$\begin{aligned}\tilde{p}_0^{(m)} = \frac{P_{wm}}{(1-2\mu)} \Big\{ &(1-2\mu)\left(C_2 e^{k_m z} - C_4 e^{-k_m z}\right) \\ &+(1-\mu)(\delta_m^2 - k_m^2)\left(C_5 e^{\delta_m z} - C_6 e^{-\delta_m z}\right)\Big\}\end{aligned} \tag{9}$$

$$\begin{aligned}\tilde{\tau}_0^{(m)} = P_{wm} \Big\{ &\left(C_1 - C_2 k_m z\right) e^{k_m z} - \left(C_3 - C_4 k_m z\right) e^{-k_m z} \\ &+ k_m \delta_m \left(C_5 e^{\delta_m z} - C_6 e^{-\delta_m z}\right)\Big\}\end{aligned} \tag{10}$$

where $\tilde{p}_0^{(m)}$ is the amplitude of dynamic wave pressure in each frequency mode; μ is the Poisson's ratio, and the $C_i (i = 1, \cdots, 6)$ coefficients can be found in [7].

The residual pore pressure (u) in a homogenous, isotropic soil can be derived from the one-dimensional Biot's consolidation equation [23],

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} + f \tag{11}$$

in which f is the mean accumulation pore pressure source term associate with the surface water waves [15] and c_v is the coefficient of consolidation, which is given by

$$c_v = \frac{2Gk_z(1-\mu)}{\gamma_w(1-2\mu)} \tag{12}$$

To solve, the following boundary and initial conditions are required:

$$u(0,t) = u(z,0) = 0\,, \quad \left.\frac{\partial u(z,t)}{\partial z}\right|_{z=-h} = 0 \tag{13}$$

where h is the thickness of a seabed.

Source Term

In this section, we now discuss the "source term" of the pore pressure generation (f). The laboratory results of De Alba *et al.* [17] relate the development of pore water pressure to the number of load cycles in simple shear tests. Their non-linear relationship is given by

$$\frac{u_g}{\sigma_0'} = \frac{1}{2} + \frac{1}{\pi}\sin^{-1}\left[2\left(\frac{N}{N_\ell}\right)^{1/\theta} - 1\right] \tag{14}$$

where u_g is the pore pressure generation due to cyclic loading, σ_0' is the effective over burden, N is the number of cyclic loading, N_ℓ is the number of cycles to liquefaction, and θ is the shape factor varying from 0.5 to 2.0 [4].

Herein, the effects of the shape factor on the relationship of pore pressure generation will be discussed. Fig. (**3**) illustrates the relationships between u_g / σ_0' and the number of load cycles (N / N_ℓ) with various values of θ. As shown in the figure, the shape factor, $\theta = 0.7$, is likely to present a linear relation, as reported in [4].

The pore pressure source term in (11) is given by Seed *et al.* [16]

$$f = \frac{\partial u_g}{\partial t} \tag{15}$$

To simplify the problem, a linear mechanism of pore pressure generation was proposed [16, 17]

$$\frac{u_g}{\sigma_0'} = \frac{N}{N_\ell} \tag{16}$$

from which the source term of pore pressure generation can be expressed as

$$f = \frac{\partial}{\partial t}\left(\sigma_0' \frac{N}{N_\ell}\right) = \frac{\sigma_0'}{TN_\ell} \tag{17}$$

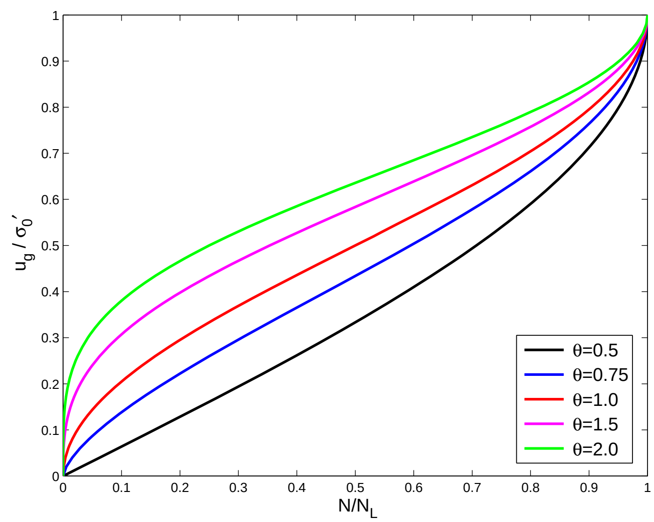

Fig. (3). Distributions of the development of pore pressure (u_g / σ_0') versus the number of load cycles (N / N_ℓ) for various values of the shape factor (θ).

In (17), N_ℓ is the number of cycles to liquefaction, which is a function of the cyclic shear stress ratio [4],

$$N_\ell = \left(\frac{\tilde{\tau}_0}{\alpha\sigma_0'}\right)^{-1/\beta} \tag{18}$$

where $\tilde{\tau}_0$ is the amplitude of wave-induced shear stress, and α and β are the functions of the soil type and relative density.

Substituting (18) into (17), we have

$$f = \frac{\sigma'}{T}\left[\frac{\tilde{\tau}_0}{\alpha\sigma_0'}\right]^{1/\beta} \tag{19}$$

which is a generalised definition of the source term. It is noted that the linear mechanism of pore pressure generation was first applied to the wave-induced pore pressure build-up in marine sediment by Seed and Rahman [4].

Solutions

The shear stress in the source term depends on both wave and soil characteristics. The complete solution of (19) is required for the source term. An analytical approximation for the linear mechanism of pore pressure generation was derived in [19-21] by using a Fourier series expansion. The residual pore pressure can be expressed as

$$u = \sum_{m=1}^{2}\sum_{n=1}^{\infty} a_n^{(m)} \left[1 - e^{-c_v \kappa_n^2 t/h^2}\right] \sin(\kappa_n z) \tag{20}$$

$$a_n^{(m)} = \frac{2h}{c_v \kappa_n^2} \int_0^h f^{(m)}(r) \sin(\kappa_n r) dr \tag{21}$$

where $\kappa_n^2 = (2n-1)\pi/2$ and $f^{(m)}$ the *m*-th component of the source term, given in (19). It is noted that the previous models [19-21] are limited to linear wave loadings, while the present study focuses on non-linear wave loadings.

Comparisons

In this section, the experimental data of Clukey *et al*. [24] is compared with the present solutions. Several tests were conducted in a small wave tank [24]. A comparison of the calculated and measure pore pressure accumulation is shown in Fig. (**4**). The input data of the experiments [24] are tabulated in Table (**1**). The results of the previous analytical solutions for linear wave loading [21], the present non-linear wave loading are included in the comparison. The relative soil depth for the data is in the range of $0.2<h/L<0.3$, hence the soil depth is intermediate. As shown in the figure, the present model for non-linear wave loading is approaching to the experimental data and provides a better prediction than that of Jeng *et al*. [21].

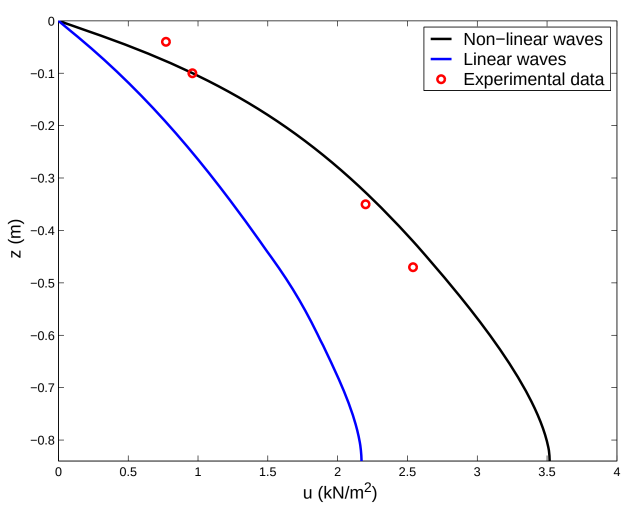

Fig. (4a). Comparison of the previous model for linear waves [21], the present model for non-linear waves and experimental data [24]-Case A.

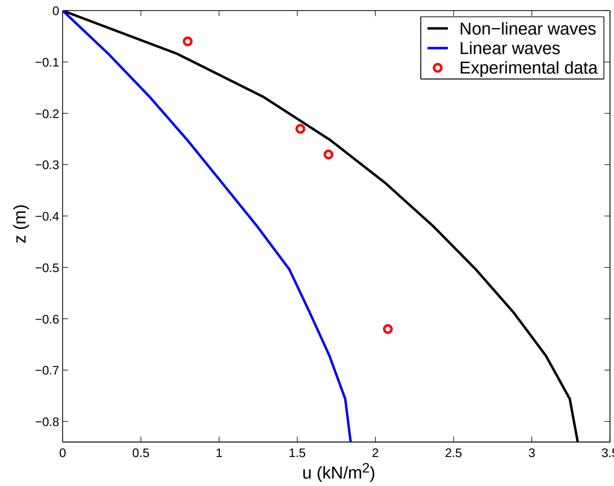

Fig. (4b). Comparison of the previous model for linear waves [21], the present model for non-linear waves and experimental data [24]-Case B.

RESULTS AND DISCUSSIONS

The objective of this paper is to investigate the effects of wave non-linearity on the wave-induced pore water pressure accumulation in marine sediments. In this section, A parametric study for examining the influences of wave and soil characteristics and residual parameters on the equilibrium pore pressure. The input data for numerical examples are listed in Table (**2**). In the numerical examples, three different sandy beds are considered.

Table 1. Input Data for Experimental Data [24]

Wave Characteristics		
	Case A	**Case B**
wave period (*T*) [sec]	1.76	2.02
water depth (d) [m]	0.5	0.5
wavelength (*L*) [m]	3.473	4.103
wave height (H) [m]	0.22	0.21
Soil characteristics		
seabed thickness (*h*) [m]	0.84	
Poisson's ratio (μ)	0.49	
soil porosity (n_e)	0.46	
shear modulus (*G*) [n/m²]	5.6×10^6	
Soil permeability (k_z) [m/sec]	4×10^{-8}	
unit weight of soil grain (γ_s) [N/m³]	18,306	
unit weight of pore fluid (γ_w) [N/m³]	9806	
coefficient of earth pressure (K_o)	0.4	
coefficient of consolidation (c_v)	0.0001165	
residual parameter (α)	0.246	
residual parameter (β)	0.165	

Table 2. Input Data for Parametric Study

Wave Characteristics	
wave period (T) [sec]	10 or various
water depth (d) [m]	20 or various
wave height (H) [m]	3 or various
Soil characteristics	
seabed thickness (h) [m]	20 or various
Poisson's ratio (μ)	0.35 (Soil A) 0.45 (Soil B) 0.49 (Soil C)
soil porosity (n_e)	0.3 (Soil A) 0.4 (Soil B) 0.46 (Soil C)
shear modulus (G) [n/m^2]	10^7 (Soil A) 5.6×10^6 (Soil B) 5.6×10^6 (Soil C)
Soil permeability (k_z) [m/sec]	10^{-2} (Soil A) 10^{-4} (Soil B) 10^{-6} (Soil C)
unit weight of soil grain (γ_s) [N/m^3]	2.65 γ_w
coefficient of earth pressure (K_o)	0.5
Residual parameter	
residual parameter (α)	0.25 or various
residual parameter (β)	0.16 or various

Effects of Wave Characteristics

In this section, three wave characteristics will be examined to see their influences on the non-linear wave-induced equilibrium pore pressure (u_e). Numerical results are plotted in Figs (**5**)-(**7**).

Wav period is one of important wave characteristics, which is related to the wave length and appearing in all wave characteristics such as wave energy, pressure and forces, etc in term of wave angular frequency ($\omega = 2\pi / T$). In the parametric study, we vary the wave period from 2.5 seconds to 15 seconds, which covers all possible ranges of ocean waves. Fig. (**5**) illustrates the vertical distribution of the equilibrium (u_e) versus soil depth for both non-linear wave [the present model] and linear wave [21] models. As shown in the figure, the value of u_e increases as wave period decreases. This is because the pore pressure will be accumulated more in a shorted wave period, as the pore pressure is more unlikely to be discharge from soil matrix during a shorter duration. It is also observed from the figure that the relative differences between linear and non-linear wave models increases as wave period decreases with a fixed wave height. This is because that the wave steepness will increase when wave period decreases with a same wave height. That is, the wave pressure and forces are larger.

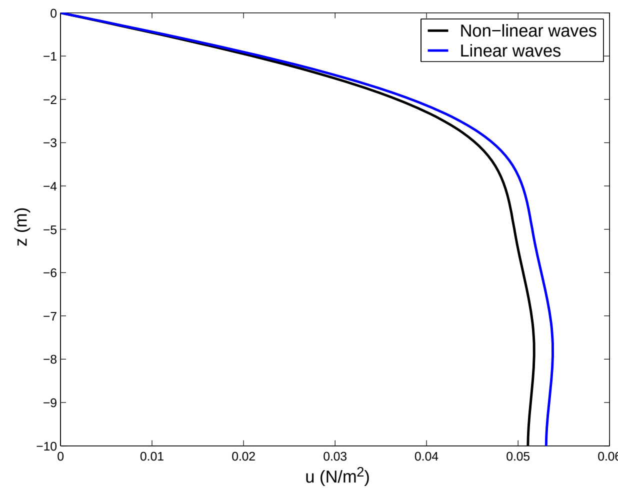

Fig. (5a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for T=15 sec.

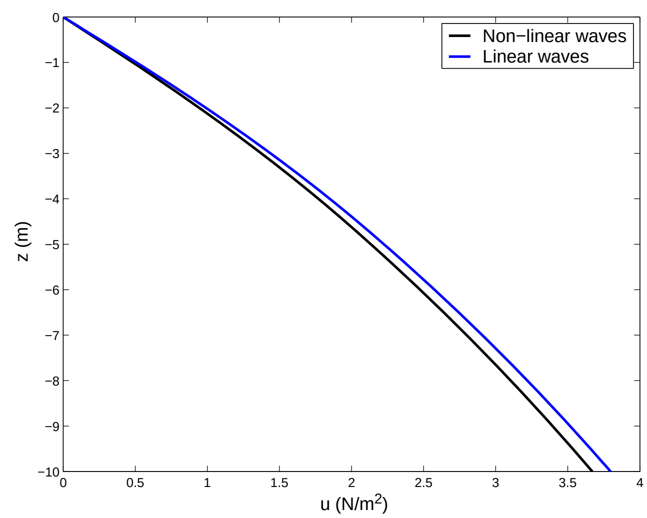

Fig. (5b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for T=10 sec.

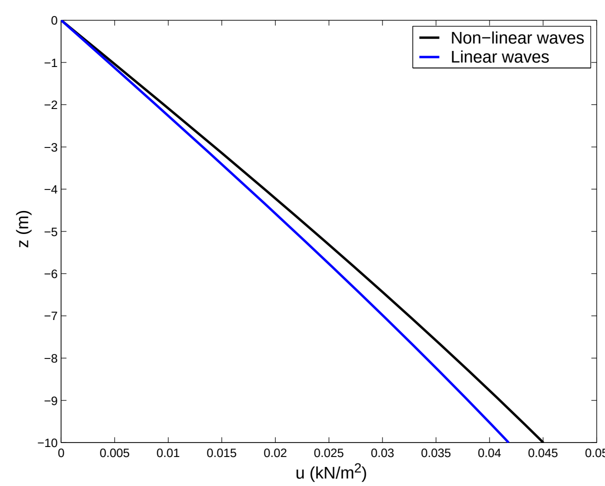

Fig. (5c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for T=7.5 sec.

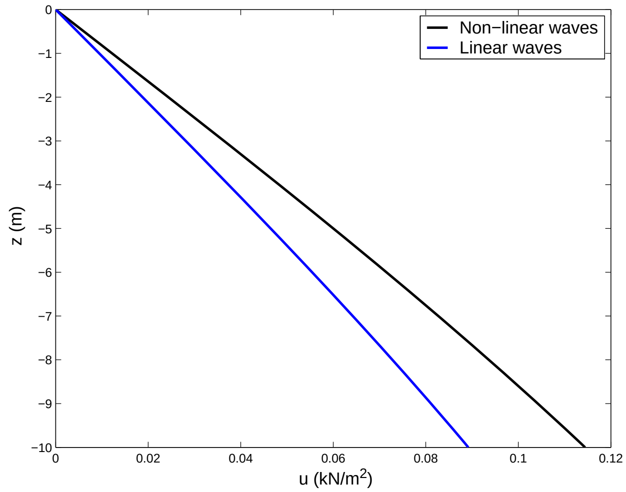

Fig. (5d). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for T=5 sec.

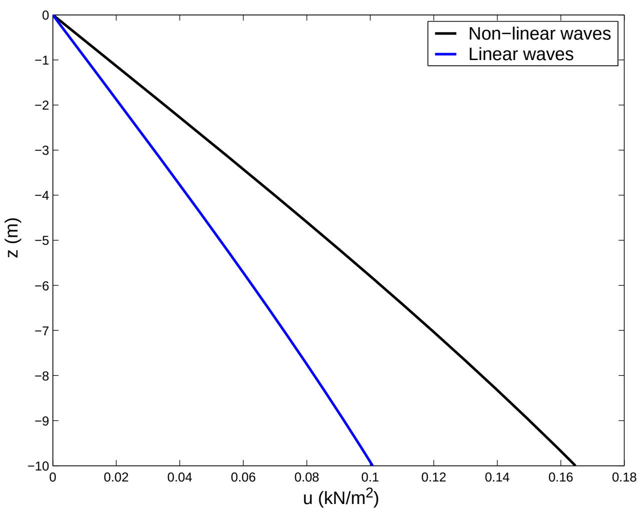

Fig. (5e). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for T=2.5 sec.

Water depth is another important wave parameter, which is also involved in the determination of wavelength and the maximum wave height before wave breaking. In this study, we vary the relative water depth from 0.05 (shallow water) to 0.3 9nearly deep water). Fig. (**6**) presents the distribution of pore pressure (u_e) versus soil depth for different relative water depth (d/L). As illustrated in the figure, pore pressure (u_e) increases as relative water depth (d/L) decreases. That implies that the residual pore pressure will accumulate more in shallow water, compared with deep water. It is also observed from the figure that the relative differences between linear and non-linear wave models increases as relative water depth (d/L) decreases with a fixed wave height.

Wave height is an important factor in the determination of wave forces and wave energy acting on marine structures. In this study, we consider non-breaking waves, which has the maximum wave steepness with the following criterion:

$$\left(\frac{H}{L}\right)_{\max} = 0.142 \tanh\left(\frac{2\pi d}{L}\right) \tag{22}$$

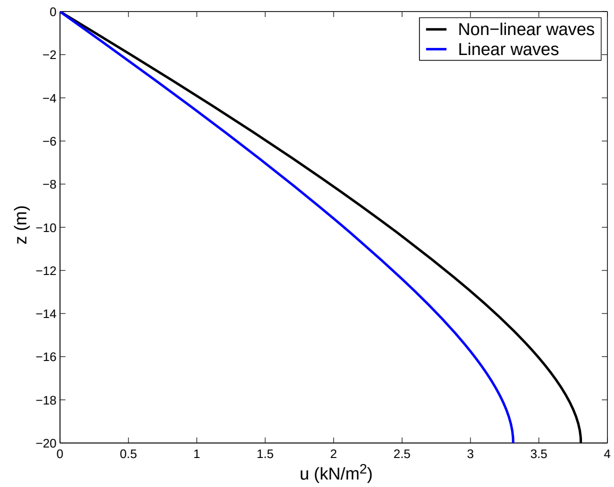

Fig. (6a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for d/L=0.3.

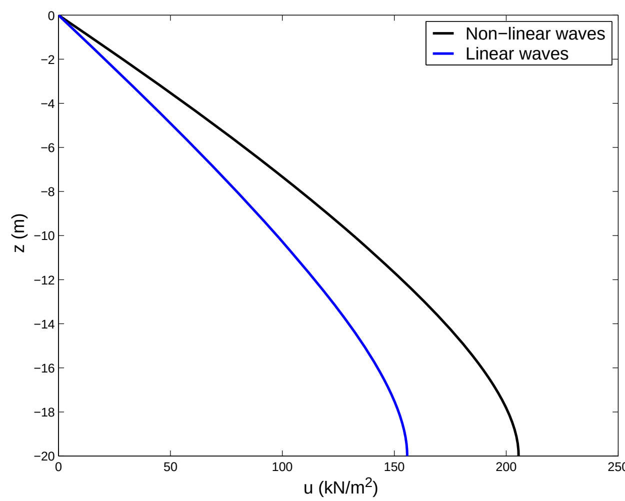

Fig. (6b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for d/L=0.2.

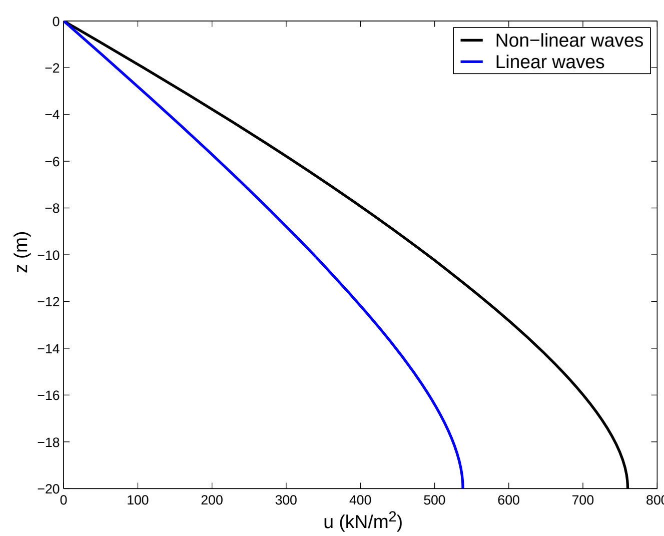

Fig. (6c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for d/L=0.15.

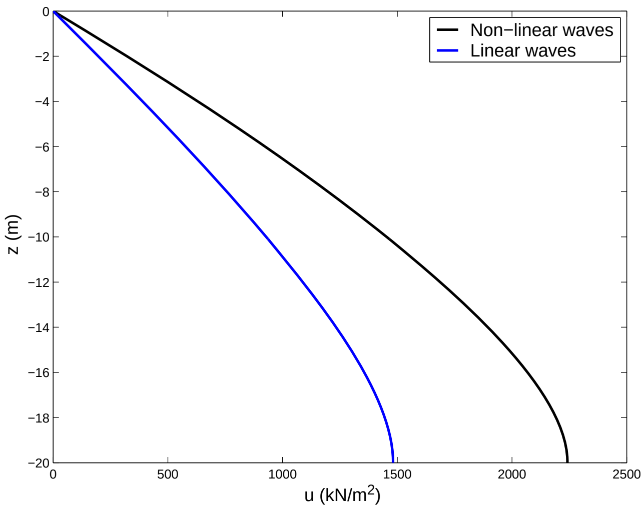

Fig. (6d). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for d/L=0.1.

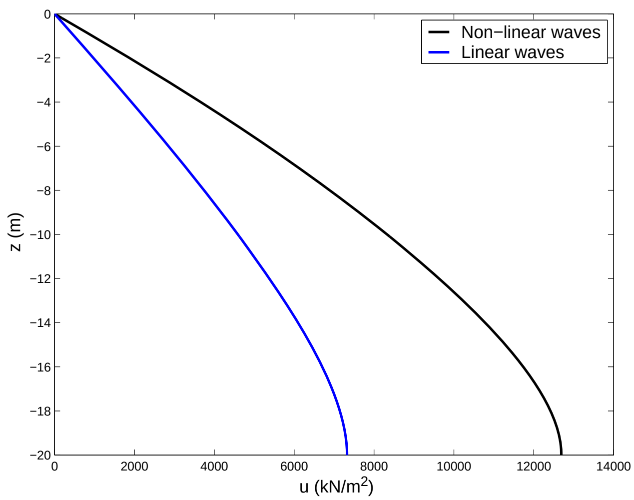

Fig. (6e). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for d/L=0.05.

One of the aims of this study is to investigate the effects of wave non-linearity on the wave-induced accumulated pore pressure in marine sediments. Fig. (7) illustrates the effects of wave steepness on the equilibrium accumulated pore pressure u_e). Generally speaking, the equilibrium accumulated pore pressures (u_e) increase as wave steepness (H/L) increases, comparing Fig. (**7a**)-(**7e**). Furthermore, the differences of pore pressure (u_e) between linear and non-linear wave components increase as the wave steepness (H/L) increases. All these differences majorly come from the second-order wave components.

Soil Characteristics

As reported in [25], several important soil parameters will significant affects the non-linear wave-induced oscillatory seabed response. They are: seabed thickness and soil type (in a combination of shear modulus, soil permeability, Poisson's ratio and porosity). In this section, we will focus on the effects of these two soil parameters.

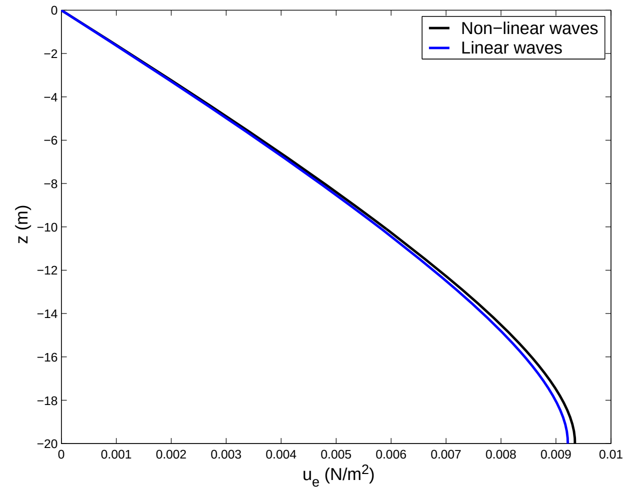

Fig. (7a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for H/L=0.005.

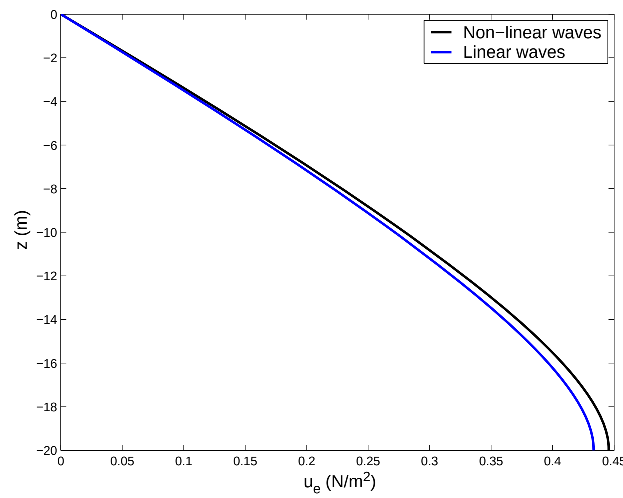

Fig. (7b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for H/L=0.01.

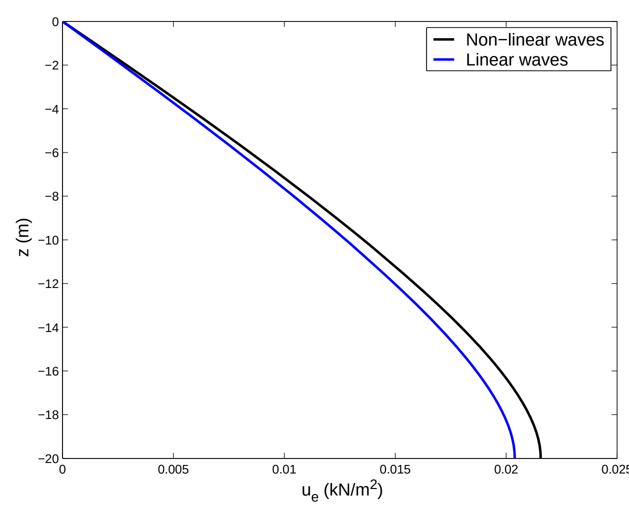

Fig. (7c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for H/L=0.02.

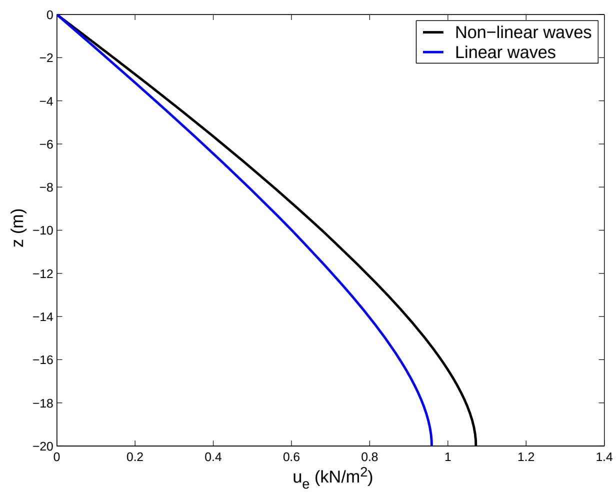

Fig. (7d). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for *H/L*=0.04.

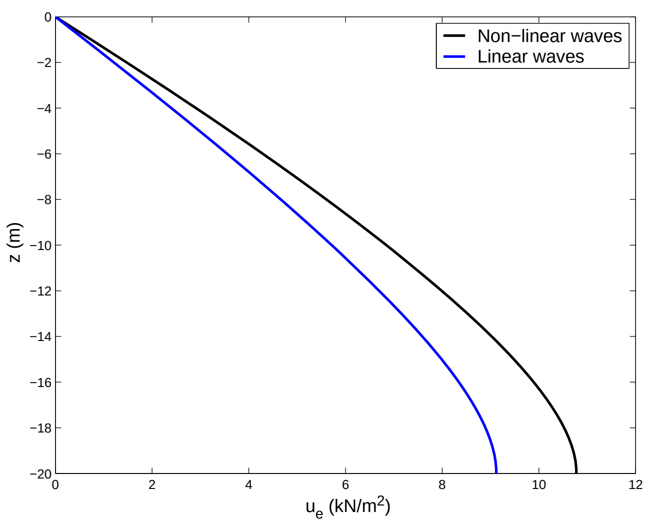

Fig. (7e). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for *H/L*=0.06.

Soil type is one of important soil characteristics in most geotechnical problems. The properties of three different soil types are tabulated in Table (**2**). Fig. (**8**) illustrates the influence of soil types on the pore pressure (u_e). As shown in the figure, The effects of wave non-linearity on the equilibrium pore pressure (u_e) become significant in softer seabed (Soil C).

Seabed thickness somehow will cause 40% relative errors of oscillatory pore pressure, if we use the solution of an infinite seabed is used [7]. Fig. (**9**) illustrates the difference of linear and non-linear wave models on the wave-induced residual pore pressure (u_e). As shown in the figure, the influences of seabed thickness increase as *h/L* increase when $h/L \leq 0.2$. However, an opposite trend is observed for the case of $h/L > 0.2$.

Residual Parameters

In addition to the wave and soil characteristics, the influences of two residual parameters, α and β, will be examined in this section. Basically, these two residual parameters, α and β, were proposed by Seed and Rahman [4]. These two parameters are normally determined by laboratory experiments with certain ranges of values. For example, α varies from 0.2 to 0.5, while β varies between 0.16 and 0.24. In this parametric study, we vary α and β within these ranges.

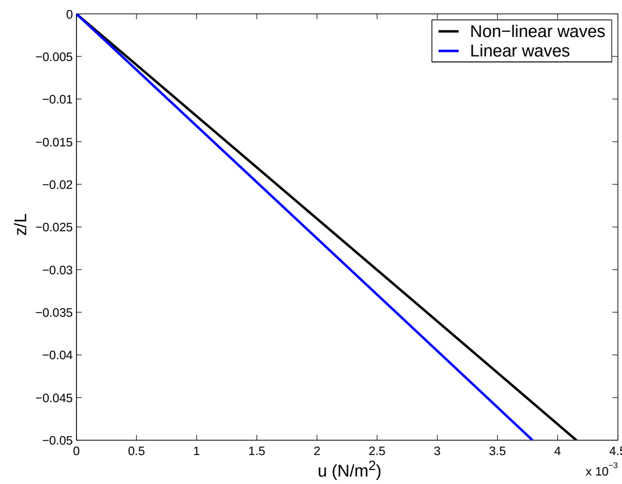

Fig. (8a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for Soil A.

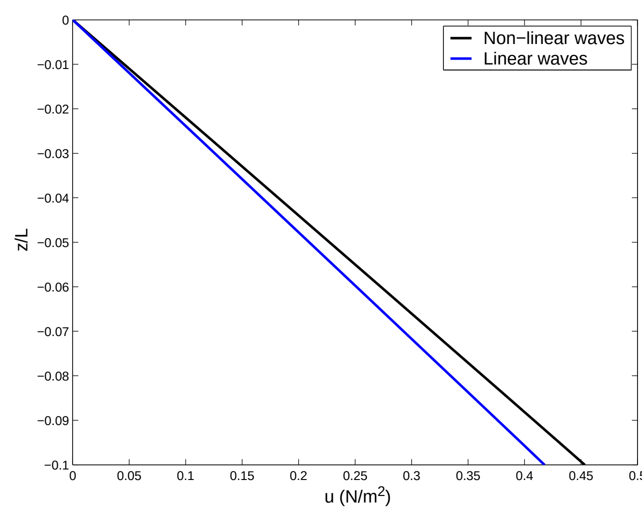

Fig. (8b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for Soil B.

Figs. (**10**) and (**11**) illustrate the influence of residual parameters on the accumulated pore pressure. As shown in the figures, the residual pore pressure increases as α decreases, and the difference between the previous linear wave model and the present non-linear wave model increases as α increase. This is because a small α will increase the source term, which will enhance the pore pressure build up. However, the influence of β is opposite to that of α, comparing Figs. (**10**) and (**11**). It is also noted that a minor change in α and β will cause a remarkable change in the equilibrium pore pressure (u_e). Therefore, determination of these residual parameters requires extremely attentions, because they are very sensitive to u_e.

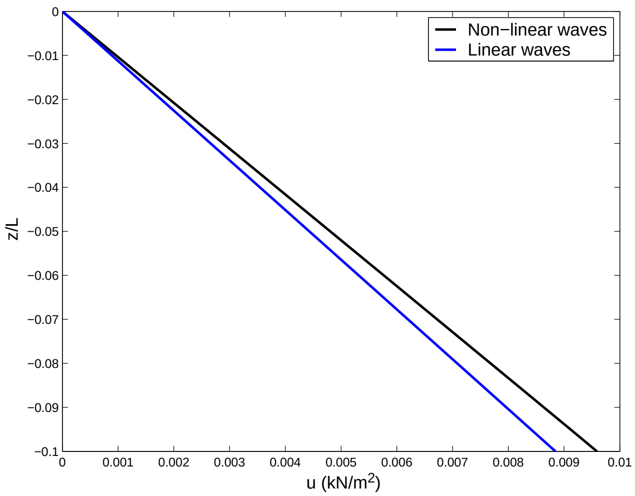

Fig. (8c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for Soil C.

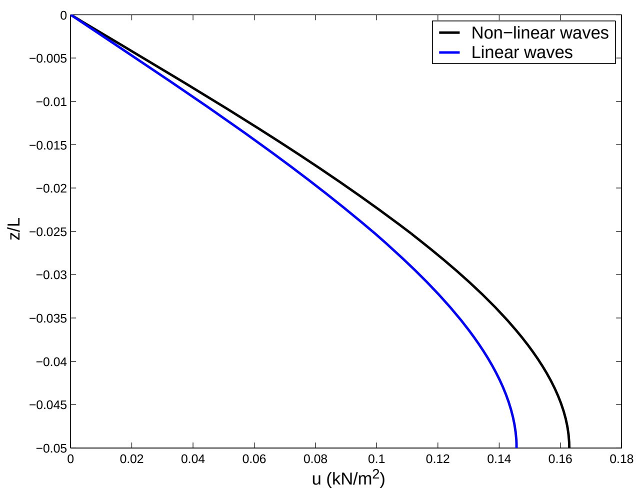

Fig. (9a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for h/L=0.05.

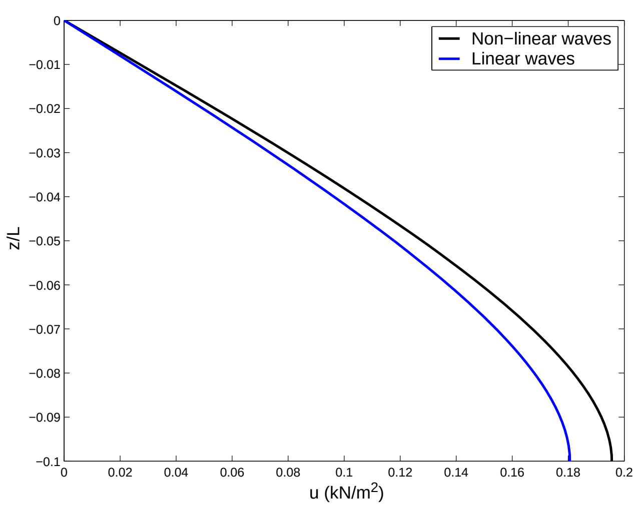

Fig. (9b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for h/L=0.1.

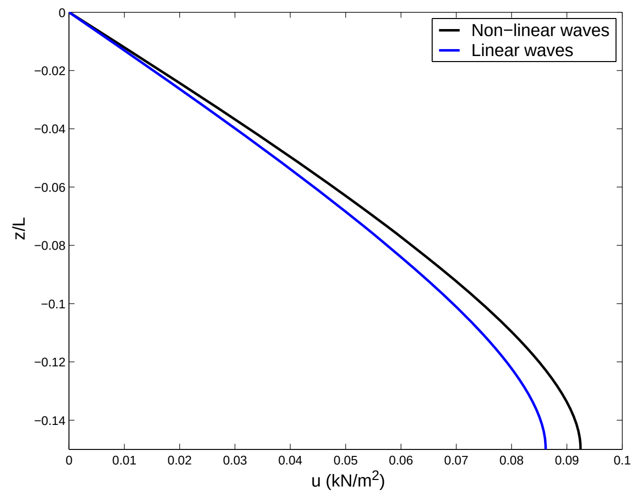

Fig. (9c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for h/L=0.15.

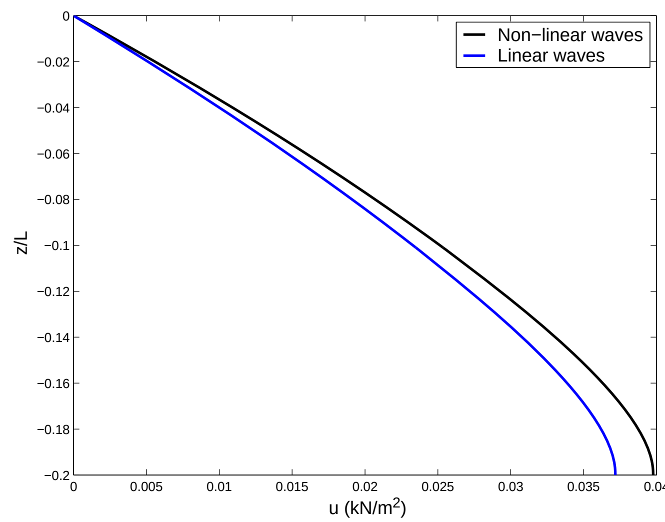

Fig. (9d). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for h/L=0.2.

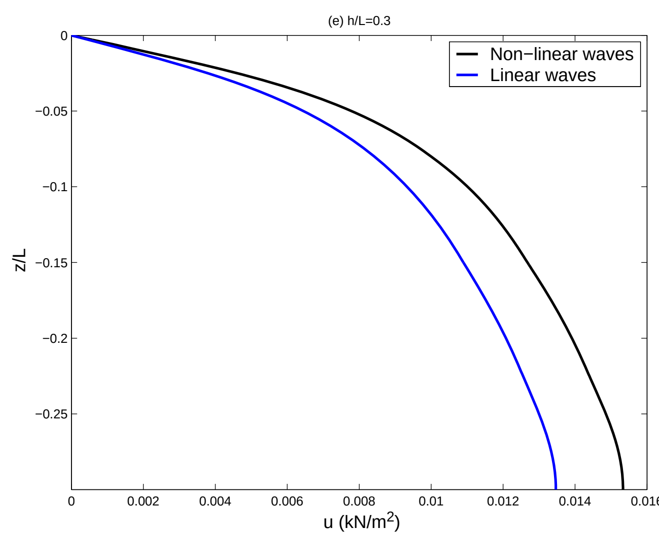

Fig. (9e). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for h/L=0.3.

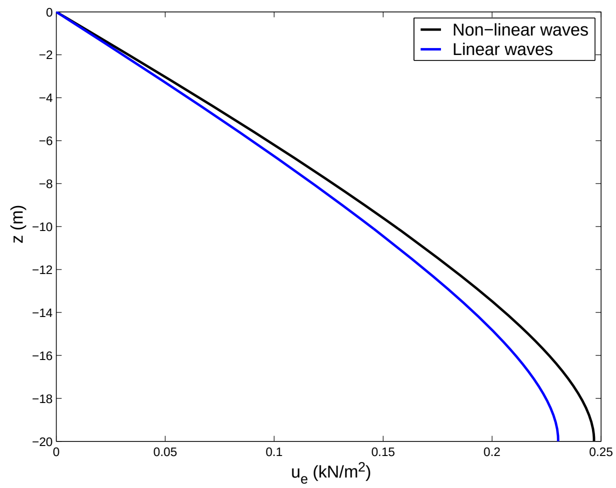

Fig. (10a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (*z*) for $\alpha = 0.2$.

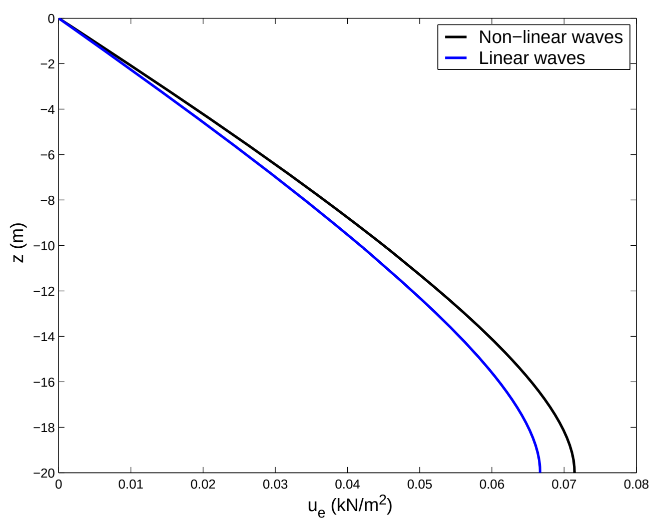

Fig. (10b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (*z*) for $\alpha = 0.25$.

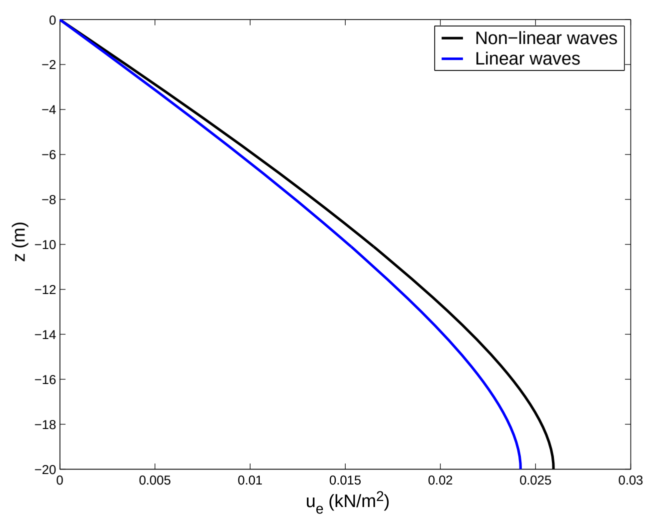

Fig. (10c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (*z*) for $\alpha = 0.3$.

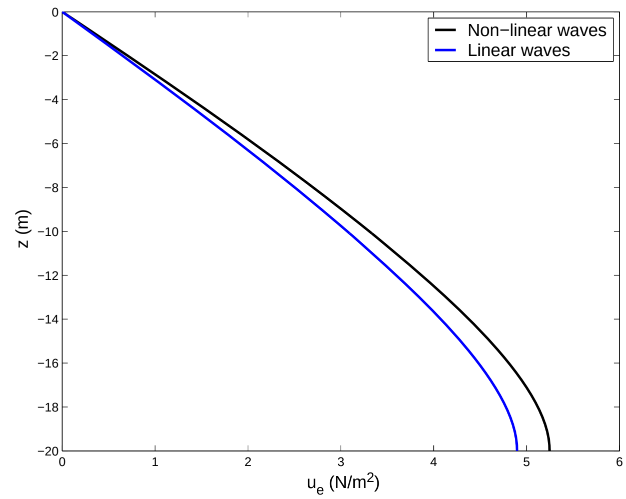

Fig. (10d). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (*z*) for $\alpha = 0.4$.

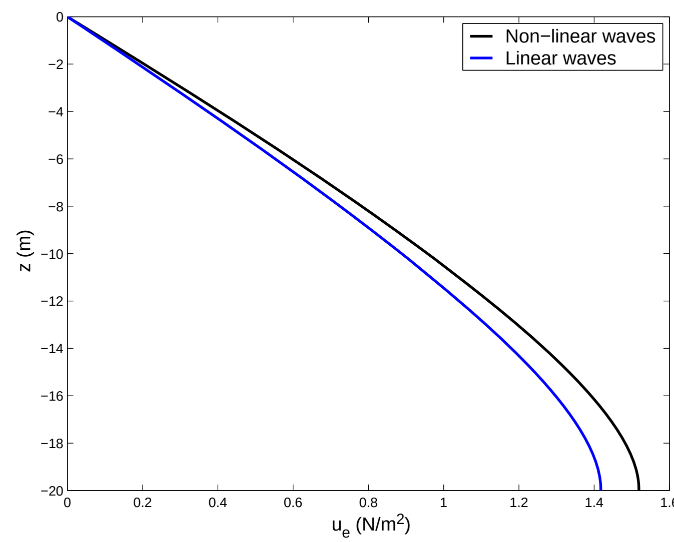

Fig. (10e). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (*z*) for $\alpha = 0.5$.

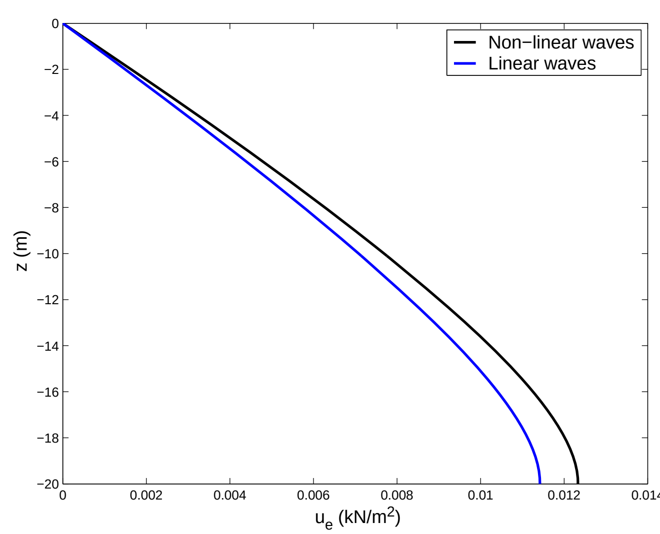

Fig. (11a). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (*z*) for $\beta = 0.16$.

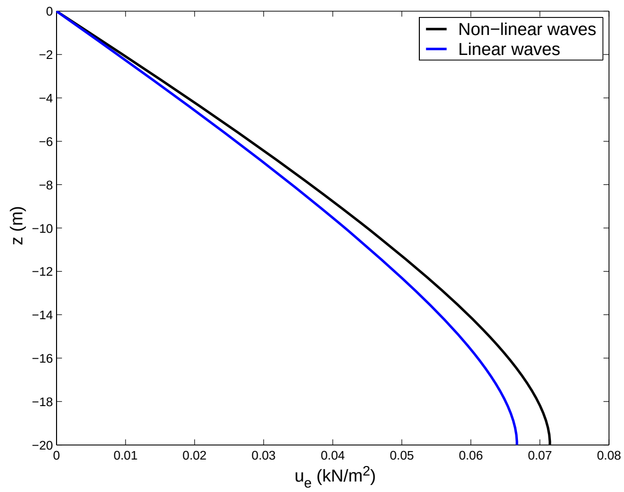

Fig. (11b). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for $\beta = 0.18$.

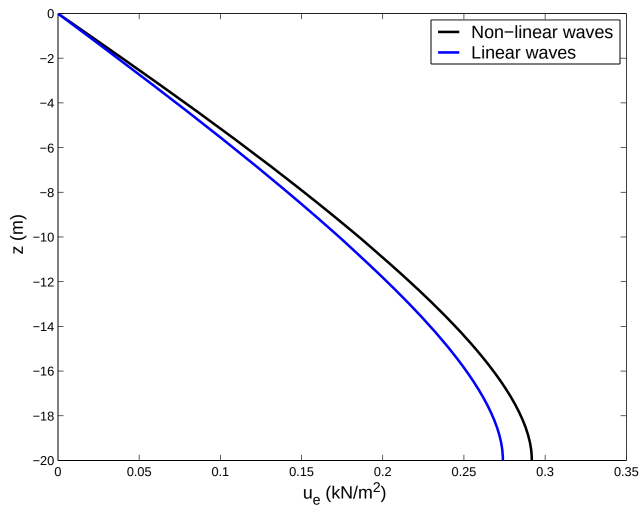

Fig. (11c). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for $\beta = 0.2$.

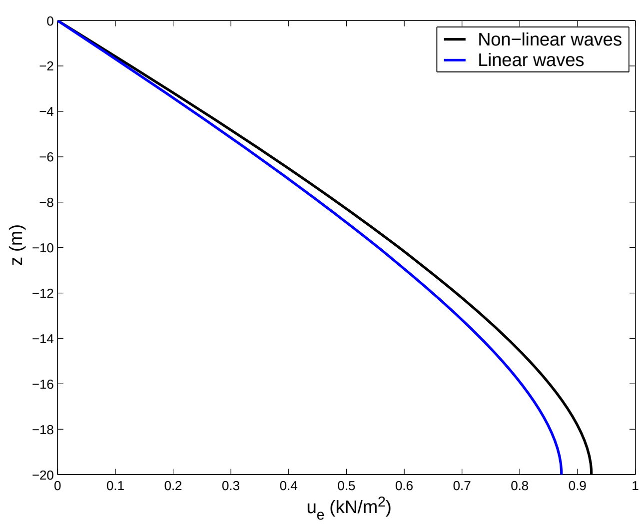

Fig. (11d). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for $\beta = 0.22$.

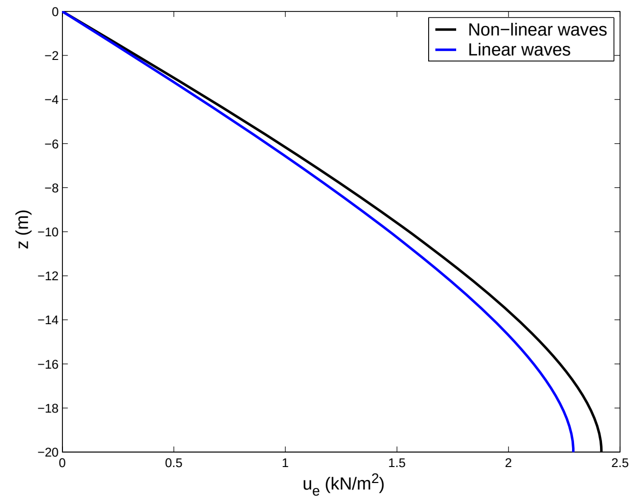

Fig. (11e). Vertical distribution of equilibrium pore pressure (u_e) versus soil depth (z) for $\beta = 0.24$.

CONCLUSIONS

In this paper, effects of wave non-linearity on the wave-induced pore pressure accumulations in marine sediments are examined, and the following conclusions can be drawn.

1. A comparison against the previous wave tank experiments show that the present model with non-linear wave theory provided a better prediction of the wave-induced equilibrium pore pressure (u_e).
2. Numerical examples indicated that the effects of wave non-linearity on pore pressure (u_e) increases as wave steepness (*H/L*) and the residual parameter (β). However, an opposite trend is found for wave period (*T*), relative water depth (*d/L*), seabed thickness (*h/L*) and another residual parameter (α).
3. The effect of wave non-linearity becomes more significant in soft seabed (Soil C).

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