

Differences of the wave-induced soil response between dynamic and quasi-static solutions.

D. H. Cha¹, D. S. Jeng¹, M. S. Rahman², H. Sekiguchi³, Y. N. Oh¹, L. Teo¹

¹School of Engineering, Griffith University, Gold Coast Campus, QLD 9726, Australia.

²Department of Civil Engineering, North Carolina State University, Raleigh, NC2765-7908, U.S.A.

³Disaster Prevention Research Institute, Kyoto University, Uji, Kyoto 611, Japan

ABSTRACT

Marine geotechnical engineers have extensively studied the phenomenon of wave-seabed interaction problems in the last decades. Most previous investigations have focused on the quasi-static soil behavior, because of the complicated mathematic procedure associated with dynamic behaviors. However, the influence of dynamic soil behavior on the wave-induced seabed response cannot always be ignored under certain combination of wave and soil conditions. In this paper, an analytical solution for the wave-seabed interaction with dynamic soil response is derived. Numerical results demonstrate the significant effects of dynamic soil behavior on the wave-induced seabed response. The applicable range of the present solution will also be clarified.

KEY WORDS: Dynamic soil behavior, wave-seabed interaction, pore pressure.

INTRODUCTION

Recently, the phenomenon of the wave-seabed interaction problem has attracted attentions from marine geotechnical and coastal engineers. The reason for this growing interest is that many marine structures have been reported to be damaged by wave-induced seabed instability, rather than from construction causes. (Silvester and Hsu, 1989).

Since the 1970's, numerous investigations related to the wave-induced seabed response have been undertaken. Since the completely dynamic analysis for the wave-seabed interaction is too complicated, most previous investigations have been limited to quasi-static analysis (Madsen, 1978; Jeng, 1997). Recently, Jeng *et al* (1999) derived an analytical solution for such a problem with a *u-p* dynamic form based on Biot's poro-elastic theory (Biot, 1960). They concluded that the inertial effects on pore pressure are significant in shallow water. However, the *u-p* dynamic approximation undertaken by Jeng *et al.*, (1999) did not include the second time derivative of the relative fluid displacements. It is expected that the acceleration term due to fluid displacements is important for large wave loading. Zienkiewicz *et al* (1980) investigated the influence of dynamic soil behavior based on

one-dimensional analysis. They concluded that the dynamic soil behavior is not important for the wave-induced soil response. However, the speed of compression a wave velocity used in their paper was fixed as 1000 m/s, which may not be valid for most cases under wave loading, especially in an unsaturated seabed. Moreover, their approach is only one-dimensional analysis, while the wave loading is a two-dimensional problem.

The main purpose of this paper is to examine the effects of dynamic soil behavior on the wave-induced pore pressure by a two-dimensional analysis. A set of governing equations will be formulated and represented in a set of non-dimensional parameters. Then, a fully dynamic analysis will be derived. Finally, the influence of several soil and wave parameters will be examined, as well as the differences between quasi-static and dynamic solutions.

THEORETICAL FORMULATIONS

Governing Equations

In this study, we consider an ocean wave propagating over a porous seabed of infinite thickness. The definition of the problem is illustrated in Fig. 1.

A two-dimensional wave-seabed interaction is considered, and the porous seabed is tested as hydraulically isotropic with uniform permeability. Biot (1941, 1960) presented a general set of equations governing the behavior of a linear elastic porous solid under dynamic conditions. They are summarised in tensor form as bellow

$$\sigma_{ij,j} = \rho_f \ddot{u}_i + \rho_f \ddot{w}_i \quad (1)$$

$$-p_{,i} = \rho_f \ddot{u}_i + \frac{\rho_f}{n} \dot{w}_i + \frac{\rho_f g}{k_z} \dot{w}_i \quad (2)$$

$$\dot{u}_{i,i} + \dot{w}_{ii} = -\frac{n}{k_f} \dot{p} \quad (3)$$

where p is pore pressure, u and w are the displacements of the solid and the relative displacement between solid and fluid and $1/k_f$ is the compressibility of pore fluid, which is defined by

$$\frac{1}{k_f} = \frac{1}{2 \times 10^9} + \frac{1 - S_r}{P_{w0}} \quad (4)$$

where S_r is the degree of saturation, P_{w0} is the absolute water pressure in unit of $[N/m^2]$.

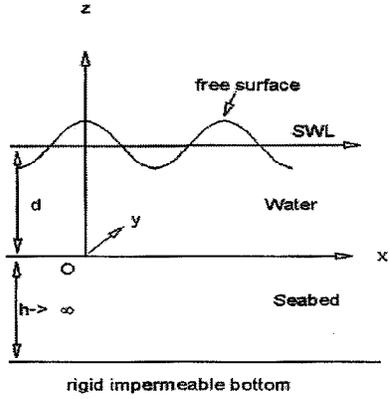


Fig. 1: Definition of wave-seabed interaction

The definition of effective stresses, σ'_{ij} , which are assumed to control the deformation of the soil skeleton, given for the total stress σ_{ij}

$$\sigma_{ij} = \sigma'_{ij} - \delta_{ij} p \quad (5)$$

where δ_{ij} is the Kronecker Delta.

Boundary conditions

To obtain the wave-induced pore pressure and then soil and fluid displacements involved in (1)–(4), appropriate boundary conditions are required. The boundary conditions are summarized here.

$$u = w = p = 0 \text{ as } z \rightarrow -\infty \quad (6)$$

$$\sigma'_{22} = \sigma'_{12} = 0, \quad p = p_o \cos(kx - \omega t) \text{ as } z = 0 \quad (7)$$

where $p_o = \gamma_w H / 2 \cosh kd$, which is the amplitude of wave pressure at the seabed surface, γ_w is the unit weight of pore fluid, H is the wave height and d is the water depth.

Fully Dynamic Solution

From Eq (3) we have

$$-\left(\frac{n}{k_f} \dot{p} \right)_i = (\dot{\epsilon}_{ii} + \dot{w}_{ii})_i \quad (8)$$

Substituting (8) into (2) and (5), the governing equation can be rewritten as:

$$\frac{k_f}{n} (u_{i,i} + w_{i,i})_i = \rho_f \ddot{u}_i + \frac{\rho_f}{n} \ddot{w}_i + \frac{\rho_f g}{k_z} \dot{w}_i \quad (9)$$

$$\sigma'_{ij,j} = -\frac{k_f}{n} \delta_{ij} (u_{i,i} + w_{i,i})_i + \rho \ddot{u}_i + \rho_f \ddot{w}_i \quad (10)$$

If the acceleration terms are neglected in the above equation, it becomes the consolidation equation, which has been used in previous works (Jeng, 1997).

Since the wave-induced oscillatory soil response fluctuates periodically in time domain, all quantities can be replaced immediately by their complex forms

$$f = \bar{f} e^{i(kx - \omega t)} \quad (11)$$

Herein, to simplify the expression of equations, we introduce three new parameters defined by

$$K_1 = G / [G / (1 - 2\mu) + k_f / n] \quad (12)$$

$$K_2 = [k_f / n] / [G / (1 - 2\mu) + k_f / n] \quad (13)$$

$$V_c^2 = [G / (1 - 2\mu) + k_f / n] / \rho \quad (14)$$

Equations (9)–(10) can be expressed in scalar form as

$$K_1 D^2 \bar{U}_1 - \frac{2(1 - 2\mu)}{1 - 2\mu} K_1 \bar{U}_1 + \Pi_2 \bar{U}_1 - K_2 \bar{U}_1 + iD \bar{U}_2 + (\Pi_2 \beta - K_2) \bar{W}_1 + iKD \bar{W}_2 = 0 \quad (15)$$

$$\left[\frac{2(1 - \mu)}{1 - 2\mu} K_1 D^2 + K_2 D^2 - K_1 + \Pi_2 \right] \bar{U}_2 + iD \bar{U}_1 + iK_2 D \bar{W}_1 + (K_2 D^2 + \Pi_2 \beta) \bar{W}_2 = 0 \quad (16)$$

$$(\beta \Pi_2 - K_2) \bar{U}_1 + iK_2 D \bar{U}_2 + \left(\frac{\beta}{n} \Pi_2 - K_2 + \frac{i}{\Pi_1} \right) \bar{W}_1 + iK_2 D \bar{W}_2 = 0 \quad (17)$$

$$iK_2 D \bar{U}_1 + [K_2 D^2 + \beta \Pi_2] \bar{U}_2 + iK_2 D \bar{W}_1 + \left[K_2 D^2 + \frac{\beta}{n} \Pi_2 + \frac{i}{\Pi_1} \right] \bar{W}_2 = 0 \quad (18)$$

where $\beta = \rho_f / \rho$, $\bar{u}_i = \bar{U}_i e^{i(\bar{x} - \bar{t})}$ and $\bar{w}_i = \bar{W}_i e^{i(\bar{x} - \bar{t})}$, and Π_1 and Π_2 are defined as

$$\Pi_1 = \frac{k_z V_c^2 k^2}{\rho_f g \omega}, \text{ and } \Pi_2 = \frac{\rho \omega^2}{\left(\frac{G}{1-2\mu} + \frac{K_f}{n} \right) k^2} \quad (19)$$

The characteristic equation of (15)-(18) can be written as

$$(\alpha_4 D^6 + \alpha_3 D^4 + \alpha_2 D^2 + \alpha_1) \bar{U}_i = 0 \quad (20)$$

where α_i are derived from (15)-(18), which are given by

$$\alpha_1 = -[(A_{12}A_{31} - A_{11}A_{32})(A_{22}A_{41} - \Pi_2^2)] \quad (21)$$

$$\begin{aligned} \alpha_2 = & [K_2(-A_{31}A_{41} + A_{11}A_{22}K_2 - A_{22}A_{31}K_2 \\ & + A_{11}A_{41}K_2 + A_{31}\Pi_2 - 2A_{11}K_2\Pi_2 \\ & + A_{31}K_2\Pi_2) + A_{32}(A_{41} + A_{11}A_{21}A_{41} \\ & + A_{22}A_{41}K_1 + A_{11}A_{22}K_2 + A_{22}K_2^2 \\ & - 2K_2\Pi_2 - 2A_{11}K_2\Pi_2 - K_1\Pi_2^2) \\ & - A_{12}(A_{21}A_{31}A_{41} + K_2(A_{22}A_{31} + A_{41} \\ & + A_{22}K_2 - \Pi_2 - 2A_{31}\Pi_2 - K_2\Pi_2))] \end{aligned} \quad (22)$$

$$\begin{aligned} \alpha_3 = & [A_{21}(-K_2(A_{12}A_{31} - A_{11}K_2 + A_{12}K_2 \\ & + A_{31}K_2 + A_{31}K_2) + A_{32}(A_{41}K_1 \\ & + A_{11}K_2 + K_2^2)) + K_2(A_{32}(1 + A_{22}K_1 \\ & - 2K_2 - A_{11}K_2 + 2K_1\Pi_2) + K_2(1 + \\ & A_{12}A_{31} + A_{22}K_1 + A_{41}K_1 - 2K_2 - A_{11}K_2 \\ & + A_{12}K_2 + A_{31}K_2 + K_2^2 - 2K_1\Pi_2))] \end{aligned} \quad (23)$$

$$\alpha_4 = K_1(A_{21} - K_2)K_2(A_{32} + K_2) \quad (24)$$

where

$$A_{11} = \Pi_2 - \frac{2(1-\mu)}{1-2\mu} K_1 - K_2 \quad (25)$$

$$A_{12} = \Pi_2 \beta - K_2 \quad (26)$$

$$A_{21} = \frac{2(1-\mu)}{1-2\mu} K_1 \quad (27)$$

$$A_{22} = K_2 - K_1 + \Pi_2 \quad (28)$$

$$A_{31} = \beta \Pi_2 - K_2 \quad (29)$$

$$A_{32} = \frac{\beta \Pi_2}{n} - K_2 + \frac{i}{\Pi_1} \quad (30)$$

$$A_{41} = \frac{\beta \Pi_2}{n} + \frac{i}{\Pi_1} \quad (31)$$

These roots of the characteristics equation (25) can be expressed as

$$\lambda_1 = -\frac{\alpha_3}{3\alpha_4} + \frac{\sqrt[3]{2}(\alpha_3^2 - 3\alpha_2\alpha_4)}{3\alpha_4\alpha_5^{1/3}} + \frac{\alpha_5^{1/3}}{3\sqrt[3]{2}\alpha_4} \quad (32)$$

$$\begin{aligned} \lambda_2 = & -\frac{\alpha_3}{3\alpha_4} + \frac{(1+i\sqrt{3})(\alpha_3^2 - 3\alpha_2\alpha_4)}{3\sqrt[3]{4}\alpha_4\alpha_5^{1/3}} \\ & + \frac{(1+i\sqrt{3})\alpha_5^{1/3}}{6\sqrt[3]{2}\alpha_4} \end{aligned} \quad (33)$$

$$\begin{aligned} \lambda_3 = & -\frac{\alpha_3}{3\alpha_4} + \frac{(1-i\sqrt{3})(\alpha_3^2 - 3\alpha_2\alpha_4)}{3\sqrt[3]{4}\alpha_4\alpha_5^{1/3}} \\ & + \frac{(1-i\sqrt{3})\alpha_5^{1/3}}{6\sqrt[3]{2}\alpha_4} \end{aligned} \quad (34)$$

where

$$\begin{aligned} \alpha_5 = & -2\alpha_3^3 + 9\alpha_2\alpha_3\alpha_4 - 27\alpha_1\alpha_4^2 \\ & + [4(-\alpha_3^2 + 3\alpha_2\alpha_4)^3 \\ & + (9\alpha_2\alpha_3\alpha_4 - 2\alpha_3^3 - 27\alpha_1\alpha_4^2)^2]^{1/2} \end{aligned} \quad (35)$$

The coupled equation can be solved. The general solution satisfied the bottom boundary condition (7) can be expressed as

$$\bar{U}_1 = a_1 e^{\lambda_1 z} + a_3 e^{\lambda_2 z} + a_5 e^{\lambda_3 z} \quad (36)$$

$$\bar{U}_2 = b_1 a_1 e^{\lambda_1 z} + b_3 a_3 e^{\lambda_2 z} + b_5 a_5 e^{\lambda_3 z} \quad (37)$$

$$\bar{W}_1 = c_1 a_1 e^{\lambda_1 z} + c_3 a_3 e^{\lambda_2 z} + c_5 a_5 e^{\lambda_3 z} \quad (38)$$

$$\bar{W}_2 = d_1 a_1 e^{\lambda_1 z} + d_3 a_3 e^{\lambda_2 z} + d_5 a_5 e^{\lambda_3 z} \quad (39)$$

where λ_i coefficients are the roots of the characteristics equation form the coupled equation. The b_i , c_i and d_i coefficients can derived from (15)-(18) as,

$$b_i = \begin{bmatrix} -i\lambda_i & iK_2 & K_2\lambda_i^2 + \Pi_2 \\ -A_{31} & A_{32} & iK_2\lambda_i \\ -i\lambda K_2 & i\lambda_i K_2 & K_2\lambda_i^2 + A_{41} \end{bmatrix} / \Delta_i \quad (40)$$

$$c_i = \begin{bmatrix} A_{21}\lambda_i^2 + A_{22} & -i\lambda_i & K_2\lambda_i^2 + \Pi_2 \\ iK_2\lambda & -A_{31} & iK_2\lambda_i \\ K_2\lambda_i^2 + \Pi_2 & -i\lambda_i K_2 & K_2\lambda_i^2 + A_{41} \end{bmatrix} / \Delta_i \quad (41)$$

$$d_i = \begin{bmatrix} A_{21}\lambda_i^2 + A_{22} & iK_2 & -i\lambda_{i2} \\ -A_{31} & A_{32} & -A_{31} \\ -i\lambda K_2 & i\lambda_i K_2 & -i\lambda_i K_2 \end{bmatrix} / \Delta_i \quad (42)$$

where,

$$\Delta_{ii} = \begin{bmatrix} A_{21}\lambda_1^2 + A_{22} & iK_2 & K_2\lambda_1^2 + \Pi_2 \\ -A_{31} & A_{32} & iK_2\lambda_1 \\ -i\lambda K_2 & i\lambda_1 K_2 & K_2\lambda_1^2 + A_{41} \end{bmatrix} \quad (43)$$

Based on the wave-induced soil and fluid displacements, we can obtain the wave-induced pore pressure, effective stresses and shear stress as

$$\begin{aligned} \bar{P} = & -\frac{k_f}{n} k \{ (i + b_1\lambda_1 + ic_1 + d_1\lambda_1) a_1 e^{\lambda_1 z} \\ & + (i + b_2\lambda_2 + ic_2 + d_2\lambda_2) a_3 e^{\lambda_2 z} \\ & + (i + b_3\lambda_3 + ic_3 + d_3\lambda_3) a_5 e^{\lambda_3 z} \} \end{aligned} \quad (44)$$

$$\begin{aligned} \bar{\sigma}'_{11} = & \frac{2Gk}{1-2\mu} \{ [(1-\mu)i + \mu\lambda_1 b_1] a_1 e^{\lambda_1 z} \\ & + [(1-\mu)i + \mu\lambda_2 b_3] a_3 e^{\lambda_2 z} + [(1-\mu)i + \mu\lambda_3 b_5] a_5 e^{\lambda_3 z} \} \end{aligned} \quad (45)$$

$$\begin{aligned} \bar{\sigma}'_{22} = & \frac{2Gk}{1-2\mu} \{ [\mu i + (1-\mu)\lambda_1 b_1] a_1 e^{\lambda_1 z} \\ & + [\mu i + (1-\mu)\lambda_2 b_3] a_3 e^{\lambda_2 z} \\ & + [\mu i + (1-\mu)\lambda_3 b_5] a_5 e^{\lambda_3 z} \} \end{aligned} \quad (46)$$

$$\begin{aligned} \bar{\sigma}'_{12} = & Gk \{ (\lambda_1 + ib_1) a_1 e^{\lambda_1 z} + (\lambda_2 + ib_3) a_3 e^{\lambda_2 z} \\ & + (\lambda_3 + ib_5) a_5 e^{\lambda_3 z} \} \end{aligned} \quad (47)$$

The three unknown coefficients, a_i ($i=1, 3, 5$) involved in (44)-(47) can be solved with the boundary condition at the seabed surface (6). Once we obtain the a_i coefficients, we can calculate the wave-induced soil response parameters.

NUMERICAL EXAMPLES AND RESULTS

Based on the analytical solution presented above, a parametric study will be carried out. In this section, the effects of soil characteristics (the degree of saturation), and wave characteristics (wave period and water depth) on the wave-induced seabed response will also be discussed. The input details of the following numerical examples are tabulated in Table 1.

Effects of Degree of Saturation

To demonstrate the difference between the dynamic and quasi-static solutions, the wave-induced pore pressure, effective normal stress and shear stress with various degrees of saturation are presented in Figs. 2-4. In these figures, the solid lines represent the results from the present solution, and dotted lines are the quasi-static solutions (Jeng 1997). As shown in Fig. 2, the degree of saturation significantly affects the wave-induced pore pressure. It clearly shows that the degree of saturation increases as pore pressure increases. The figure also indicates a slight difference between the quasi-static and dynamic solutions.

Table 1: Input data for numerical examples

| Wave Characteristics | |
|----------------------------|------------------------------------|
| Wave Period T | 15 sec or various |
| Water d | 50m or various |
| Soil Characteristics | |
| Degree of saturation S_r | 0.95 or various |
| Porosity n | 0.35 |
| Permeability | 1.0×10^{-2} m/sec |
| Shear modulus G | 5.0×10^6 N/m ² |

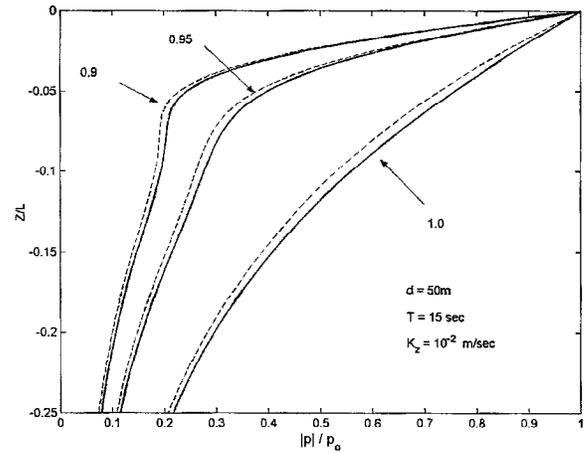


Fig. 2: Vertical distribution of the wave-induced pore pressure (p/p_0) versus the soil depth (Z/L) for various values of degree of saturation (S_r) in a seabed.

Figs. 3 and 4 illustrate the comparison between dynamic and quasi-static on the effective stresses and shear stresses for various values of degree of saturation. Figure 3 clearly show the significant difference of effective normal stress between the two solutions, when soil depth becomes deeper values. Its influence will increase as the degree of saturation decreases. Figure 4 indicate that the shear stress does not affected by the degree of saturation. However, the differences between dynamic and quasi-static analytical solutions are clearly shown this figure.

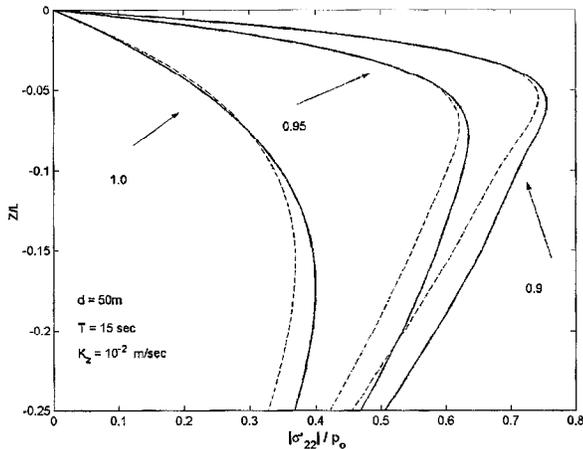


Fig. 3: Vertical distribution of the vertical effective normal stresses ($|\sigma'_{22}|/p_0$) versus the soil depth (Z/L) for various values of degree of saturation (S_r) in a seabed.

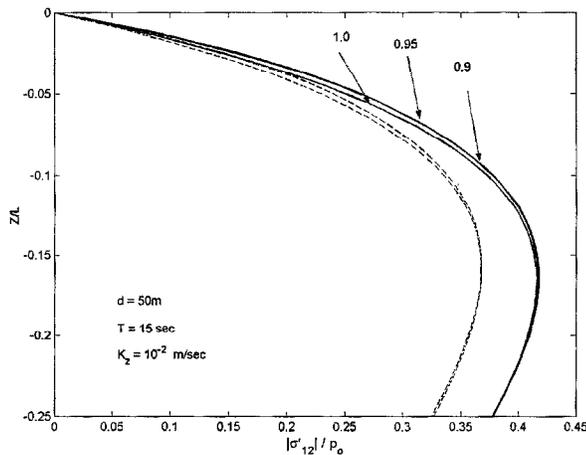


Fig. 4: Vertical distribution of the shear stresses ($|\sigma'_{12}|/p_0$) versus the soil depth (Z/L) for various values of degree of saturation (S_r) in a seabed.

Effects Of Water Depth

Besides the degree of saturation, the wave characteristics are also important in the determination of the wave-induced soil response. In this section, we further examine the effects of dynamic soil behavior with various values of water depths.

Figs. 5-7 illustrate the comparison between two solutions on the wave-induced pore pressure, effective stresses and shear stresses for various values of water depths. As shown in Fig. 5, the magnitude of the wave-induced pore pressure increases as water depth increases. However,

only a slight difference of pore pressure between dynamic and quasi-static analytical solutions is observed.

Fig. 6 indicates that water depth significantly affects the vertical effective normal stresses. It also indicates that there are more significant differences between dynamic and quasi-static analytical solutions in deeper regions of the soil column.

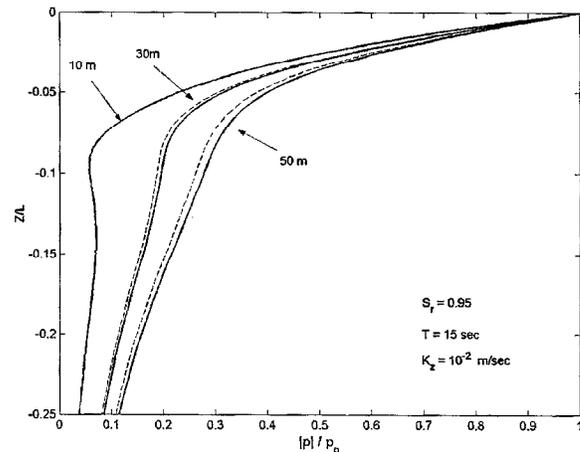


Fig. 5: Vertical distribution of the wave-induced pore pressure ($|p|/p_0$) versus the soil depth (Z/L) for various values of water depth (d) in a seabed.

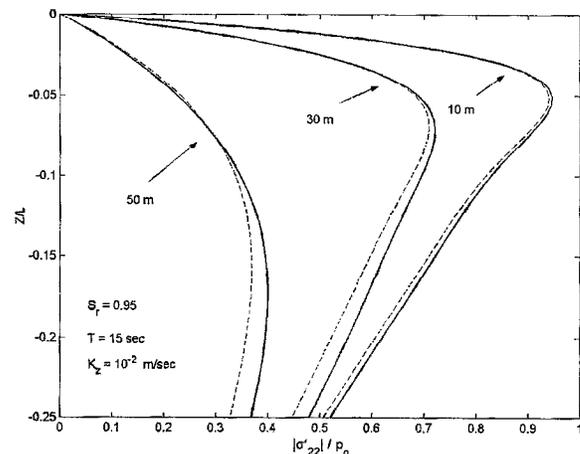


Fig. 6: Vertical distribution of the vertical effective normal stresses ($|\sigma'_{22}|/p_0$) versus the soil depth (Z/L) for various values of water depth (d) in a seabed.

Effects Of Wave Period

Wave period is another important wave parameters in the determination of wave-induced soil response. In this section, we investigate the influence of dynamic soil behavior on the wave-induced soil response with various wave periods.

Figs. 8-10 present the vertical distribution of the wave-induced pore pressure, effective normal stresses, and shear stress for various values

of the wave period between dynamic and quasi-static analytical solutions. Fig. 8 indicates that the maximum amplitude of the wave-induced pore pressure decreases as the wave period increases. Compared with the pore pressure, soil depth significantly effects the wave period on the effective stress, especially in the 15sec wave period. Similar trends for the shear stress are observed in Fig. 10.

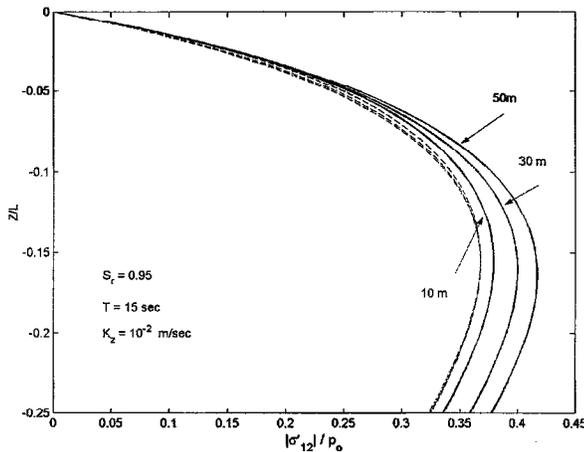


Fig. 7: Vertical distribution of the shear stresses ($|\sigma'_{12}|/p_0$) versus the soil depth (Z/L) for various values of water depth (d) in a seabed

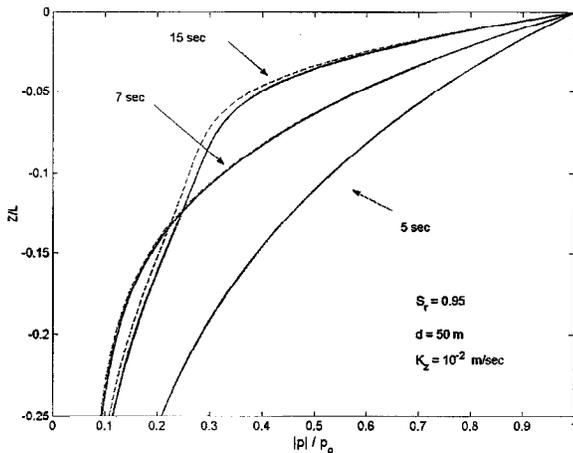


Fig. 8: Vertical distribution of the wave-induced pore pressure ($|p|/p_0$) versus the soil depth (Z/L) for various values of wave period in a seabed.

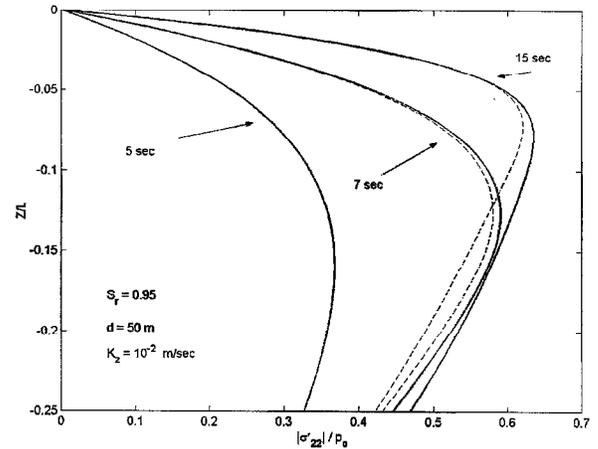


Fig. 9: Vertical distribution of the vertical effective normal stresses ($|\sigma'_{22}|/p_0$) versus the soil depth (Z/L) for various values of wave period in a seabed.

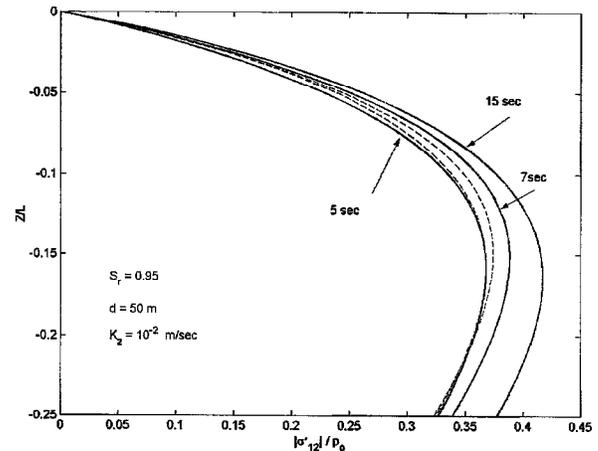


Fig. 10: Vertical distribution of shear stress ($|\sigma'_{12}|/p_0$) versus the soil depth (Z/L) for various values of wave period in a seabed.

CONCLUSIONS

In this paper, the phenomenon of ocean waves propagating over a porous seabed was re-examined by including dynamic soil behavior. Unlike previous one-dimensional analysis, we directly solve the wave-seabed interaction problem by a two-dimensional analysis. Based on the numerical results presented, the following conclusions can be drawn:

- (1) The dynamic soil behavior cannot always be ignored in the estimation of the wave-induced seabed response for certain combination of wave and soil characteristics.
- (2) The wave-induced pore pressures are affected by the degree of saturation in both dynamic and quasi-static solution. The degree of saturations significantly affects to the effective stress and shear stresses. There are more significant in deeper area of the seabed.
- (3) There is a significant difference of effective stress and shear stress between dynamic and quasi-static solution with various water depth, compared with other soil response.
- (4) The effective stress and shear stress are significantly affected by wave period, while the wave-induced pore pressure is not.

This paper only presents some preliminary results, more advanced results and the application of the solution is currently undertaken, and will be available in the future.

ACKNOWLEDGEMENTS

This study was supported by ARC IREX Award (2001), ARC Large Grant (2001-2003) and AAS-JSPS Exchange program (2001-2002).

REFERENCES

- Biot, MA (1941). "General theory of three dimensional consolidation." *Journal of Applied Physics*, Vol. 12, pp155-164.
- Biot, MA (1960). "Mechanics of deformation and acoustic propagation in porous media." *Journal of Applied Physics*, Vol. 33, pp1483-1498.
- Jeng, DS (1997). *Wave-Induced Seabed Response in Front of a Breakwater*, PhD thesis, The University of Western Australia.
- Jeng, DS, Rahman, MS and Lee, TL (1999). "Effects of inertia forces on wave-induced seabed response." *International journal of Offshore and Polar Engineering*, Vol. 9, No. 4, pp307-313.
- Madsen, OS (1978). "Wave-induced pore pressures and effective stresses in a porous bed." *Geotechnique*, Vol. 28, No. 4, pp377-393.
- Silverster, R, and Hsu, JRC. (1989). "Sines Revisited," *J Waterways, Port, Coastal and Ocean Engineering, ASCE*, Vol 115, No3, pp 327-344.
- Zienkiewicz, OC, Chang, CT and Bettess, P (1980). "Drained, undrained, consolidating and dynamic behaviour assumptions in soils." *Geotechnique*, Vol. 30, No 4, pp385-395