1 Overview

1.1 Background

Marine geotechnics is a multidisciplinary research theme covering conventional civil engineering disciplines such as fluid mechanics, coastal engineering, geotechnical engineering and structural engineering. This research area has attracted great attention among coastal and geotechnical engineers due to the growing activities in marine environments worldwide. An appropriate design for the foundations of marine infrastructures such as breakwaters, offshore pipelines, platforms and offshore wind turbine systems plays an important role in the success of offshore engineering projects. The evaluation of the soil response to hydrodynamic loading such as waves and currents around foundations of marine structures and the resultant seabed instability is one of the key factors in the design of foundations.

When a coastal structure is installed in a marine environment, the presence of the structure will alter the flow patterns in its immediate neighbourhood. The flow condition around the structure not only affects the wave force acting on the structure, but also induces seafloor instability. The former has been the main concern in the design of coastal structures, and has been intensively studied by coastal and structural engineers in the past. However, the latter involves the foundations of the structure, and has attracted attention from coastal geotechnical engineers in recent years.

In the past few decades, considerable efforts have been devoted to the phenomenon of wave-soil-structure interactions. The major reason for the growing interest is that many coastal structures (such as vertical walls, caissons, offshore monopiles and pipelines) have been damaged by the wave-induced seabed response, rather than from construction deficiencies (Christian, Taylor, Yen & Erali 1974; Smith & Gordon 1983; Lundgren, Lindhardt & Romold 1989). It has been reported in the literature that concrete armour blocks at the toes of a marine structure subsided into the seabed, and wave-induced liquefaction and shear failure have been identified as the culprit for this problem (Silvester & Hsu 1989). Another reason is that poro-elastic theories for wave-soil interactions have been applied to field measurements, such as the determination of the shear modulus of soil (Yamamoto & Trevorrow 1991) and the directional spectra of ocean surface waves (Nye & Yamamoto 1994), as well as acoustic waves propagating through porous media (Yamamoto & Turgut 1988).

When water waves propagate in the ocean, they generate significant dynamic pressures on the seafloor. These dynamic pressures further induce pore-water pressure and

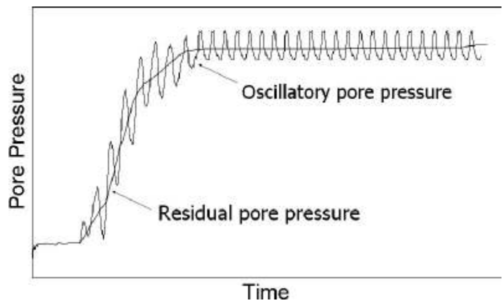

Figure 1.1 Conceptual sketch of two different mechanisms of pore pressure (not to scale).

effective stresses within the seabed. With excess pore-water pressure and diminishing vertical effective stress, part of the seabed may become unstable or even liquefied. Once liquefaction occurs, the soil particles are likely to be carried away as a heavy fluid by any prevailing bottom current or mass transport owing to the action of ocean waves.

The occurrence of seabed instability is a widespread phenomenon in ocean environments. There is evidence of ocean floor instability in a wide variety of offshore regions, from shallow water, near-shore zones, continental slopes and beyond to deep ocean floors. Seabed instability has been responsible for the damage and destruction of offshore structures (Christian et al. 1974; Bea, Wright, Sircar & Niedoroda 1975; Barends 1991).

1.2 Recent Advances in Theoretical Models for Wave-Seabed Interactions (WSIs)

1.2.1 An Overview of Theoretical Models

This section is an attempt to give a comprehensive review of wave-seabed interactions around marine structures. It also takes into consideration all state-of-the-art knowledge. We start off with the basic models, including a detailed review and summary of existing work.

In general, two mechanisms of the wave-induced soil response have been reported in the literature, as shown in Figure 1.1, based on the observations in the laboratory and field measurements, depending on the manner in which the pore pressure is generated (Zen & Yamazaki 1990*a*, 1990*b*; Nago, Maeno, Matsumoto & Hachiman 1993). The first mechanism 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 (Madsen 1978; Yamamoto, Koning, Sellmeijer & Hijum 1978). This mechanism is particularly important for small-amplitude waves and the seabed could only liquefy momentarily in the seabed under wave troughs (Jeng 2013). 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 (Seed & Rahman

1978; Sumer & Fredsøe 2002). As reported in Jeng (2013), the residual mechanism is important for large wave loading.

Numerous models for the wave-induced seabed response have been developed with various assumptions since the 1940s. The assumptions and the leading governing equations for each model are summarised here.

- **Simplified model:** In the model, both pore fluid and soil are considered incompressible media, and the accelerations due to fluid and soil motion are ignored. This leads to an uncoupled model, in which Laplace's equation is the governing equation (Putnam 1949). Another similar approach is to include the compressibility of pore fluid, which leads to the diffusion equation (Nakamura, Onishi & Minamide 1973).
- **Biot's poro-elastic models:** Basically, three types of poro-elastic models have been developed since the 1970s, including the consolidation model (or quasi-static model), the *u–p* approximation and dynamic models.
  - *Consolidation model:* In the consolidation model, both the pore fluid and the soil skeleton are considered to be compressible, but the accelerations due to fluid and soil motion are ignored. Since the acceleration has been ignored, this model is also called the quasi-static model (Biot 1941). The classic analytical solutions for this model were reported in Yamamoto et al. (1978), Madsen (1978) and Okusa (1985*b*) for an infinite seabed, in Hsu & Jeng (1994) for a seabed of finite thickness, and in a layered seabed (Hsu, Jeng & Lee 1995).
  - *u–p approximation:* In this model, the acceleration due to pore fluid is ignored in the general formulation. This model was first proposed by Zienkiewicz, Chang & Bettess (1980) with a one-dimensional analysis, and was extended to a two-dimensional analysis for wave-seabed interaction (Jeng, Rahman & Lee 1999; Jeng & Rahman 2000).
  - *Dynamic model:* The full set of governing equations established by Biot (1956*a*) is employed in the analysis, in which accelerations due to both pore fluid and soil motion are included. Since this model is rather complicated, only a few investigations are available in the literature (Jeng & Cha 2003; Ulker, Rahman & Jeng 2009).
- **Inelastic models for residual soil response:** Biot's poro-elastic models have been commonly used to model the wave-induced oscillatory soil response. For a residual mechanism that is involved with the build-up of pore pressures in a seabed, Seed & Rahman (1978) proposed adding a source term in the conventional consolidation equation. In the source term, the wave-induced oscillatory shear stresses are applied to generate cyclic loading. This one-dimensional model was first used to study earthquake-induced liquefaction, and Seed & Rahman (1978) further applied it to wave-induced residual soil response. Some analytical solutions and numerical codes have been reported in the literature (McDougal, Tsai, Liu & Clukey 1989; Cheng, Sumer & Fredsøe 2001; Sumer & Fredsøe 2002; Jeng, Seymour & Li 2007).
- **Poro-elastoplastic model:** In addition to linear poro-elastic models, some advanced models such as the poro-elastoplastic model for the wave-induced seabed response have been developed recently (Sassa & Sekiguchi 2001; Jeng & Ou 2010). The

poro-elastoplastic models will provide better predictions of the potential of the wave-induced seabed instability, which is normally a large deformation.

In the aforementioned models, both analytical and numerical models have been employed to obtain the wave-induced soil response. In the following sections, most previous theoretical investigations for the wave-induced seabed response are reviewed under the heading of each model.

1.2.2 Simplified Models

Uncoupled models have been used as the first approximation in the area of wave-seabed interactions. In these models, either the pore fluid or the soil skeleton has been considered as an incompressible medium. The accelerations due to both pore fluid and soil motion are ignored in this approach. The governing equation is either Laplace's equation or the diffusion equation, for which the analytical solutions have been well developed.

Models with the Laplace Equation

Based on the assumptions of a rigid, permeable sandy seabed and incompressible pore fluid, Laplace's equation is the governing equation for wave-induced pore pressure, i.e.,

$$\nabla^2 p = \frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial z^2} = 0, \tag{1.1}$$

where p is the pore pressure in the seabed.

Using a linear wave theory, Putnam (1949) presented a simple solution for an isotropic porous seabed of finite thickness and concluded that a significant loss of wave energy occurred in the presence of a porous sandy seabed due to viscous percolation of fluid. The percolation was activated by the pressure variation at the interface between seawater and seabed. The solution indicated that pressure distribution within the seabed depended only on the wave characteristics and geometry of the sand layer, and not on the properties of the seabed. However, a possible error in the wave height was observed in Putnam's paper, which overestimated the dissipation function by a factor of four (Reid & Kajiura 1957).

Based on the same assumptions as Putnam (1949), the wave-induced pore pressure for a porous seabed of finite thickness with anisotropic permeability was examined by Sleath (1970). He also conducted laboratory experiments to verify the theory, resulting in the discovery of a phase lag (less than 10°) in the pore pressure. However, these experimental results were inconsistent with his theoretical results. This inconsistency is due to the assumptions of his theoretical approach.

Considering the viscous effect of the boundary layer and energy balance, Liu (1973) modelled the flow in a permeable bed and determined the damping rate for an infinite seabed. Continuity of pressure and velocity is required because boundary conditions at the interface are up to order $O(\sqrt{\nu})$, where ν is the viscosity of the pore fluid. His results indicated that there was no relationship between pore pressure and permeability, whilst fluid velocity depended on the porosity and permeability. However, Liu's solution (1973) only considered the pressure condition whilst neglecting shear stress. Thus, it may not be a complete analysis of viscous flow. With the same framework, Liu (1977) further developed a solution for the damping of the wave-induced pressure in a two-layered porous seabed. Compared with the solution for an infinite seabed (Liu 1973), the pore pressure was found to depend on both the permeability and the thickness of the upper layer only to a small degree. However, he only considered the case of a two-layered seabed with uniform permeability in each layer. Later, based on the generalised Darcy equation (Dagan 1979), the boundary layers between the seabed surface and the impervious stratum were included in the model of Liu & Dalrymple (1984). It was concluded that the spatial damping rate depended strongly on the permeability and the water depth when the physical wave number remained approximately the same over the rigid bottom layer.

From a different aspect, Massel (1976) took into account the non-linear damping and the inertia terms in the momentum equation for a rigid porous seabed. His results indicated that the effect of permeability on the pressure distribution in the seabed was negligible and that they were essentially the same as that from Laplace's equation.

In describing the wave-induced soil response, Mallard & Dalrymple (1977) used a set of elastic displacement equations based on the assumption of stress equilibrium within the soil, neglecting the effect of soil inertia on the response. Dawson (1978) considered the soil inertia term in the model of Mallard & Dalrymple (1977) and concluded that the effect of the soil inertia term could not generally be ignored without committing a serious error, in the case of an incompressible soil.

Models with the Diffusion Equation

Another type of uncoupled model was proposed by Nakamura et al. (1973) and Møshagen & Tørum (1975), based on assumptions of compressible pore fluid and a non-deformable porous soil skeleton. This model results in the heat conduction equation or diffusion equation for pore pressure, i.e.,

$$\nabla^2 p - \frac{\gamma_w n \beta}{k_z} \frac{\partial p}{\partial t} = 0, \tag{1.2}$$

where γ_w is the unit weight of water, n is the porosity, β is the compressibility of pore water that is defined in Equation 1.4 and k_z is the permeability in the z direction.

Among these studies, Nakamura et al. (1973) compared theoretical results of pore pressure with laboratory experiments in fine and coarse sandy beds. The experimental results for the latter showed no phase lag and agreed reasonably well with the solution from Laplace's equation. The data for fine sand exhibited a large pressure attenuation and phase lag, which agreed reasonably well with the diffusion theory. However, an unexplainable pressure discontinuity exists near the seabed surface in their experimental data. As Yamamoto et al. (1978) pointed out, the waves generated in the experiments of Nakamura et al. (1973) were too steep. Therefore, the state of stresses in sandy beds under wave crests and troughs might have reached the limit of equilibrium or a state of liquefaction, thus causing a large pressure drop. Furthermore, a critical error was found in their calculations. The compressibility of the water used in the calculation was 980 times that of real water. Yamamoto et al. (1978) showed that the false agreement reported might be explained by the existence of a small amount of air in the sand used. However, no further confirmation has been reported in the literature.

Møshagen & Tørum (1975) considered wave-induced flow in a porous medium under the assumption of compressible pore fluid and an incompressible soil skeleton. They found that the inclusion of pore-fluid compressibility in the analysis of wave-induced pore pressures in a porous soil significantly altered the vertical seepage forces acting on the soil. However, the assumption made by Møshagen & Tørum (1975) regarding the relative compressibility of the pore fluid and the soil skeleton appeared somewhat unrealistic (Prevost, Eide & Anderson 1975). Thus, serious doubts arise about the validity of Møshagen & Tørum's (1975) conclusion.

All the aforementioned theories assumed that the seabed is a rigid porous medium. Because these approaches do not permit the coupling of pore-fluid motion and soil-skeleton motion, the governing equation for the pore pressure is Laplace's equation for an incompressible fluid, or the diffusion equation for compressible pore fluid for hydraulically isotropic (i.e., isotropic permeability) seabeds. However, such solutions for pore pressures are limited to the particular case of soil and wave conditions, i.e., Laplace's equation for very permeable beds such as coarse sand, or a diffusion equation for poorly permeable beds such as clay. Furthermore, these approaches provide no information for the effective stresses and soil displacements in the seabed.

1.2.3 Biot's Poro-Elastic Models for Oscillatory Mechanism

Coupled models generally treat both the soil and the pore fluid as compressible media. They can more precisely describe the mechanical properties of porous media and the interaction between soil and pore fluid. Recently, investigations of the consolidation or the dynamic response of seabed soil were conducted using these coupled models. Biot's theory is the model most widely used for soil-fluid interaction. Depending on the inclusion of the acceleration of soil particles and pore fluid, and the displacement of pore fluid relative to soil particles, there are three types of expression of Biot's equation: the quasi-static model, the 'u–p' approximation and the full dynamic models.

Consolidation Models (Quasi-Static Model)

Biot (1941) first formulated a three-dimensional consolidation equation, treating the soil as an isotropic poro-elastic porous medium with compressible pore water and deformable soil particles. The quasi-static model was established (Biot 1941) with the following assumptions: (1) the soil is homogeneous and isotropic; (2) the stress-strain relation is reversible (linearly elastic) under final equilibrium; (3) small deformations of solid and pore fluid are considered; (4) the pore fluid in porous media and soil particles are compressible; and (5) the water flows steadily within the porous medium, e.g. Darcy's flow ($R_e \leq 1$; Gu & Wang 1991).

Based on the conservation of mass, we have the two-dimensional consolidation equation

$$\frac{k_x}{k_z}\frac{\partial^2 p}{\partial x^2} + \frac{\partial^2 p}{\partial z^2} - \frac{\gamma_w n\beta}{k_z}\frac{\partial p}{\partial t} = \frac{\gamma_w}{k_z}\frac{\partial \epsilon_s}{\partial t} \tag{1.3}$$

for a hydraulically anisotropic seabed. In Equation 1.3, n is the soil porosity; γ_w is the unit weight of the pore water; k_x and k_z are the soil permeabilities in the x and z directions, respectively; the compressibility of the pore fluid (β) and the volume strain (ϵ_s) are defined as

$$\beta = \frac{1}{K_w} + \frac{1 - S_r}{P_{w0}} \quad \text{and} \quad \epsilon_s = \frac{\partial u_s}{\partial x} + \frac{\partial w_s}{\partial z}, \tag{1.4}$$

where K_w is the true bulk modulus of elasticity of the pore water that is taken as 1.95×10^9 N/m^2 (Yamamoto et al. 1978); u_s and w_s are the soil displacements in the x and z directions, respectively; S_r is the degree of saturation; and P_{w0} is the absolute water pressure. If the soil skeleton is fully saturated, then $\beta = 1/K_w$ since $S_r = 1$.

Based on the force balance and Hooke's law, we have the governing equations for overall equilibrium in a pore-elastic medium as

$$G\nabla^2 u_s + \frac{G}{1 - 2\mu}\frac{\partial \epsilon_s}{\partial x} = \frac{\partial p}{\partial x}, \tag{1.5}$$

$$G\nabla^2 w_s + \frac{G}{1 - 2\mu}\frac{\partial \epsilon_s}{\partial z} = \frac{\partial p}{\partial z}, \tag{1.6}$$

where G is the shear modulus of soil and μ is Poisson's ratio.

In Biot's poro-elastic theory, the flow of fluid in a porous medium is generally treated as steady flow. Therefore, Darcy's law is used in formulating the equations. Furthermore, only the small deformation problem could be applicable. It is worth noting that Biot's consolidation equations (1941) ignored the inertia terms of the solid and fluid. This kind of simplification is acceptable for a consolidation process with small permeability or low-frequency loading problems as the acceleration of the solid or fluid is apparently small in this situation.

Most previous investigations with Biot's consolidation equations have been directly solved to obtain the wave-induced pore pressure, soil displacements and effective stresses. This approach was first developed by Yamamoto et al. (1978) and Madsen (1978), who considered compressible pore fluid in a compressible porous medium. A three-dimensional general consolidation equation (Biot 1941) was adopted in these studies, in which only progressive waves were examined. Among these, Madsen (1978) considered a hydraulically anisotropic and unsaturated porous bed, whilst Yamamoto et al. (1978) studied an isotropic medium. Both considered only an infinite thickness. Moreover, Yamamoto (1977) investigated soil response in a homogeneous soil of finite thickness under isotropic and partially saturated conditions. However, Yamamoto's solution (1977) was cast in a semi-analytical manner that did not have a closed form.

Yamamoto et al. (1978) concluded that when the stiffness of a porous medium is much less than that of the pore fluid (for example, saturated soft soils), the soil response is independent of the permeability and has no phase lag. On the other hand, when the stiffness of a porous medium is much greater than that of the pore fluid (for example, partially saturated dense sands), pore pressure attenuates rapidly. In the latter case, the phase lag increases linearly with the distance from the seabed surface.

Madsen (1978) investigated a hydraulically anisotropic and partially saturated seabed. He found that these conditions had an appreciable effect on the nature of wave-induced effective stresses in coarse sand. As reported in Madsen (1978), the effect of partial saturation on soil response may be significant.

Yamamoto (1981) developed a semi-analytical solution for a non-homogeneous layered porous seabed, together with a comprehensive verification using data obtained from the Mississippi Delta. Yamamoto (1981) pointed out that a layer of concrete blocks had a significant effect on the wave-induced soil response. However, the assumption of treating the concrete blocks as soils seems unrealistic because the properties of concrete blocks are quite different from those of soil.

Okusa (1985*b*) used the compatibility equation under elastic conditions and reduced the governing equation of Yamamoto et al. (1978) to a fourth-order differential linear equation. It is noted that Okusa's (1985*b*) study was based on plane-stress conditions, whereas Yamamoto et al.'s (1978) study was based on plane-strain conditions. Okusa (1985*b*) found that the wave-induced pore pressure and effective stresses consisted of two parts. The first depends only on the wave characteristics and the second is related to both the sediment and the wave characteristics. It was reported that the wave-induced soil response depended only on the wave conditions, not on the soil characteristics, for a fully saturated and isotropic sandy seabed of infinite thickness. However, this conclusion is invalid for an isotropic seabed of finite thickness, even under a saturated condition (Gatmiri 1990; Jeng & Hsu 1996).

Rahman, El-Zahaby & Booker (1994) summarised the previous work with the direct analytical framework in a semi-analytical analysis. In their model, a general layered seabed is considered, which is particularly important for the design of a cover layer for seabed protection.

The aforementioned investigations have been limited to an isotropic homogeneous seabed, which may be an idealised case. The major difficulty in analysing the wave-induced soil response in a seabed with variable permeability has been the governing equation includes variable coefficients. By employing the Varley–Seymour (VS) function (Varley & Seymour 1988), Seymour, Jeng & Hsu (1996) derived an analytical solution for such a condition. In their study, only fine sand is considered. In their model, the first-order derivation of permeability with respect to vertical distance was excluded, which has been reported to play an important role in the evaluation of wave-induced soil response in coarser material (Lin & Jeng 1997). Later, Jeng & Seymour (1997*a*, 1997*b*) further developed analytical solutions for general soils in seabeds of both infinite and finite thicknesses. They concluded that the relative difference of the wave-induced pore pressure between variable and uniform permeability might be up to 23 per cent of the amplitude of the wave pressure at the seabed surface.

Recently, another analytical solution for the wave-induced seabed response with variable permeability was proposed by Kitano & Mase (2001). However, their one-dimensional (1D) model is limited to exponential distribution decay of the permeability, although it provides a simpler formulation than that of Jeng & Seymour (1997*a*, 1997*b*).

The inclusion of cross-anisotropic soil behaviour in the wave-seabed interaction can also be handled analytically. Jeng (1996*b*, 1997*b*) may have been the first to derive the analytical solution for wave-seabed interactions in a cross-anisotropic seabed. His numerical results show that the conventional solution with the assumption of isotropic soil behaviour may overestimate the pore pressure, but underestimate the effective stresses. The consideration of cross-anisotropic soil behaviour is particularly important in determining the wave-induced soil displacements. An identical approximation was also proposed by Yuhi & Ishida (1997).

A simplified analytical solution for the soil response in a cross-anisotropic seabed was proposed by Yuhi & Ishida (2002). In the model, two parameters related to boundary-layer thickness and stiffness ratio were introduced. However, their model is only applicable in the region near the seabed surface (for example, $|z|/L < 0.02$, where L is the wavelength of ocean waves); outside this region, the relative difference between the simplified solution (Yuhi & Ishida 2002) and the exact solution (Jeng 1996*b*) is not negligible, as reported in Jeng (2003*a*).

Since the direct analytical solution for the wave-induced seabed response involves complicated mathematical presentations, especially for a seabed of finite thickness, it is impossible to have a closed-form solution for a layered seabed. An alternative approximation, the boundary-layer approximation, was proposed by Mei & Foda (1981). The principle of the boundary-layer approximation is to divide the whole soil domain into inner and outer regions. In the inner region (near the seabed surface, defined by the boundary-layer thickness), a full solution is required. On the other hand, a simplified solution is sufficient in the outer region. This solution agrees well with that of Yamamoto (1977) for fine sand. However, it may lose accuracy for all soils in unsaturated conditions and for coarse-sand, undersaturated conditions (Yamamoto 1977). This shortcoming may be attributed to the solution being only suitable for a seabed with low permeability, for which a scaling was carried out (Huang & Song 1993). However, the solutions are more convenient for engineering applications due to their much simpler form compared to those of Hsu & Jeng (1994).

Although the boundary-layer approximation proposed by Mei & Foda (1981) is a simple yet fairly accurate analysis, it was restricted to low-frequency waves only. Huang & Chwang (1990) investigated Biot's equation for the acoustic problem and obtained three uncoupled Helmholtz equations to represent each of the three kinds of waves. Their approach is applicable for the complete range of wave frequencies.

Later, Huang & Song (1993) applied this approach to investigate the problem of linear water waves in a channel of constant depth propagating over a horizontal poro-plastic bed of infinite thickness. In the general solution presented, five non-dimensional physical parameters were defined. One of them represents the relative stiffness of solids and fluids and another expressed penetrability, whilst the other three revealed Mach numbers for two longitudinal waves and one transverse wave of a porous medium of low soil permeability. However, their solution was restricted to a porous seabed of infinite thickness. Later, Song & Huang (2000) further applied the boundary-layer approximation to examine the mechanism of laminar poro-elastic media flow under wave loading.

Kitano & Mase (1999) derived another set of analytical solutions to investigate the influence of cross-anisotropic soil behaviour on wave-induced soil response through the boundary-layer approximation. Their model agreed well with the exact solution proposed by Jeng (1997*b*) for fine sand.

Besides the direct analytical solution and boundary-layer approximation, Sumer & Cheng (1999) proposed a random-walk model for the wave-induced pore pressure in marine sediments. The principle of the model is based on the shear stress in the marine sediment to determine the built-up pore pressure. However, the determination of the shear stress relied on other analytical solutions such as that of Hsu & Jeng (1994).

Madga (1996, 1997, 2000) developed a one-dimensional finite-difference model of wave-induced pore pressure in a highly saturated sandy bed and applied it to the case of buried pipelines. It was concluded that the time phase in pressure generation is dominated by the degree of saturation, the compressibility of the soil skeleton and the soil permeability.

Zen & Yamazaki (1990*a*, 1990*b*) simplified a two-dimensional boundary-value problem to one dimension, based on the assumption of the seabed thickness being very small compared with the wavelength. A numerical model (finite-difference method) was also established, which was only applicable to a single layer of porous seabed.

Gatmiri (1990) developed a simplified finite element model of wave-induced effective stresses and pore pressure in an isotropic and saturated permeable seabed. Two important conclusions were drawn from this paper. First, there exists a critical bed thickness about 0.2 times the wavelength, in which the horizontal movement of the soil skeleton is a maximum and where the unstable state occurs. Second, the soil response is affected by soil characteristics even in a hydraulically isotropic and saturated seabed of finite thickness. This result complemented the solution for a seabed of infinite thickness reported by Okusa (1985*b*). However, the general trend of pore pressure distribution versus seabed thickness in Gatmiri (1990) was found to be inconsistent (Jeng & Hsu 1996).

Gatmiri (1992) further extended the numerical model to consider the soil response in a cross-anisotropic saturated seabed. The numerical results showed that the effects of cross-anisotropic soil parameters are significant and the soil response was affected by the combined parameters in different ways. Compared with the cross-anisotropy, the effect of hydraulic anisotropic permeability on the variation of effective stresses may be insignificant.

A possible error in the results of Gatmiri's model (1990, 1992) may have stemmed from the boundary condition used. The lateral boundaries at $x/L = 0$, 1 in that paper, '$v = 0$, $p = 0$ and u free', were used in the model. However, it has been proved that there is a phase lag in the soil response in a fully saturated seabed of finite thickness (Jeng & Hsu 1996). This implies that the lateral boundary conditions, $v = 0$ and $p = 0$, are invalid in a porous seabed of finite thickness. Thus, the numerical results of Gatmiri (1990, 1992) seem doubtful. In fact, this obstacle can be overcome by using the principle of repeatability (Zienkiewicz & Scott 1972), as suggested by Lin & Jeng (2000*a*).

Thomas (1989, 1995) developed a one-dimensional finite element method for a two-layered unsaturated seabed. His results agreed well with the analytical solutions of Yamamoto (1977) and Okusa (1985*b*). It was also suggested that the stiffer sediment in the top layer dominated the response of the bottom layer in a two-layered seabed.