

Chapter 2

Recent Advances

Abstract In this chapter, a comprehensive literature review for the wave-seabed interactions in marine sediments, including wave-seabed-structure interactions, was provided. In this review, the major outcomes of the existing research were summarized. The objective of this chapter is to provide readers an overall understanding of the topic.

Keywords Literature review · Pore pressure · Seabed instability · Wave-seabed interactions

2.1 Introduction

The present book is an attempt to give a comprehensive account of wave-seabed interaction around marine structure. It also takes into consideration all state-of-the-art knowledge. We shall start off with the literature review (the present chapter). These include a detailed review and summary of existing work.

2.2 Waves Propagating over a Porous Seabed: Theoretical Models (Transient Mechanism)

Numerous models for the wave-induced seabed response have been developed with various assumptions since 1940's. The assumptions and the leading governing equations for each model will be summarized here:

- *Un-coupled model (or drained model):*

In the model, both pore fluid and soil are considered as incompressible medium, and the accelerations due to fluid and soil motion are ignored. This leads to an un-coupled model, in which the *Laplace's Equation* is the governing equation.

Another similar approach is to include the compressibility of pore fluid, which leads to *Diffusion Equation*.

- *Consolidation model (or quasi-static model):*

In this model, both pore fluid and soil are considered to be compressible, but the accelerations due to fluid and soil motion are ignored. This assumption leads to Biot's consolidation equation.

Since the acceleration has been ignored, this model is also called as quasi-static model. This model has been commonly used since 1978 [130].

- *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 *et al.* [141] with a one-dimensional analysis, and been extended to two-dimensional analysis for wave-seabed interaction [46, 48].

- *Dynamic model:*

The full set of governing equations established by Biot [5, 6] is employed in the analysis, in which both accelerations due to pore fluid and soil motion are included. Since this model is rather complicated, only a few investigations have been available in the literature [8, 38, 47, 121].

- *Poro-elastoplastic model:*

Besides linear poro-elastic models, some advanced models such as the poro-elastoplastic model for the wave-induced seabed response have been developed recently [101–105]. The poro-elastoplastic models will provide better predictions of the potential of the wave-induced seabed [135] 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 will be reviewed under the heading of each model.

2.2.1 *Un-coupled Models (or Drained Models)*

Un-coupled models have been used as the first approximation in the area of wave-seabed interaction. In the model, either pore fluid or soil has been considered as incompressible medium. The accelerations due to both pore fluid and soil motion are ignored in this approach. Since the governing equation is either *Laplace's Equation* or the *Diffusion Equation*, for which the analytical solutions have been well developed, most previous investigations with these assumptions are analytical approximations.

Based on the assumptions of a rigid, permeable sandy seabed and incompressible pore fluid, Laplace's equation is the governing equation for the wave-induced pore pressure. Using a linear wave theory, Putnam [89] 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 sea water 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 of the wave height was observed in Putnam's paper, which overestimated the dissipation function by a factor of four [97].

Based on the same assumptions as Putnam [89], the wave-induced pore pressure for a porous seabed of finite thickness with anisotropic permeability was examined by Sleath [108]. He also conducted laboratory experiments to verify the theory, resulting in the discovery of a phase lag (less than 10 degrees) 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 [63] modeled the flow in a permeable bed and determined the damping rate for an infinite seabed. Continuity of pressure and velocity are required as boundary conditions at the interface 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 [63] only considered the pressure condition whilst neglecting shear stress [28]. Thus, it may not be a complete analysis of viscous flow.

With the same framework of Liu [63], Liu [64] 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 [63], the pore pressure was found to depend on both the permeability and 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 generalized Darcy's equation [11], the boundary layers between the seabed surface and the impervious stratum are included in the model [65]. 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 its rigid bottom.

Because the aforementioned continuum approach disregards the seepage flow and pore pressure development in soils, they provide no information on the effective stress, which is essential when elastic waves in soils are considered. However, it has been reported that the internal friction form is independent of strain rate [110]. The experimental data suggested that the soil internal friction is not of viscous friction form, but of Coulomb friction type.

From a different aspect, Massel [72] 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 [71] 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 [12] considered

the soil inertia term in the model of Mallard [71], and concluded that the effect of the soil inertia term could not generally be ignored without committing a serious error for the case of an incompressible soil.

Another type of un-coupled model has been proposed by Nakamura *et al.* [79] and Moshagen and Torum [76], based on assumptions of compressible pore fluid and a non-deformable porous seabed. This results in the heat conduction equation or diffusion equation for pore pressure. Among these, Nakamura *et al.* [79] 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 the 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.* [130] pointed out, the waves generated in the experiments of Nakamura *et al.* [79] were too steep. Therefore, the state of stresses in sandy beds under wave crests and troughs might have reached the limit of equilibrium or the 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 the real water. Yamamoto *et al.* [130] showed that the false agreement reported might be explained by the existence of a small amount of air in the sand used.

Moshagen and Torum [76] considered the wave-induced flow in a porous medium under the assumption of compressible pore fluid and an incompressible soil. They found that the inclusion of pore fluid compressibility in the analysis of the wave-induced pore pressures in a porous soil significantly altered the vertical seepage forces acting on the soil. However, the assumption made by Moshagen and Torum [76] regarding the relative compressibility of the pore fluid and the soil skeleton appeared somewhat unrealistic [88]. Thus, serious doubts arise about the validity of Moshagen and Torum's 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 motion, the governing equation for the pore pressure is Laplace's equation for incompressible fluid, or the diffusion equation for compressible pore fluid for hydraulically isotropic (i.e., isotropical permeability) seabeds. However, such solutions for pore pressure are limited to a 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.

2.2.2 Biot's Consolidation Model (Quasi-Static Model)

The second model for the wave-induced seabed response is based on the assumption of compressible pore fluid and soil, but ignoring the accelerations due to pore fluid

and soil motion. This model has been widely used to investigate the wave-induced soil response since 1970's. The methodologies in solving the governing equations can be summarized into three divisions: direct analytical solution, boundary-layer approximation and numerical modeling.

2.2.2.1 Direct Analytical Solution

Most previous investigations with Biot's consolidation equations have been directly solved and obtain the wave-induced pore pressure, soil displacements and effective stresses. This approach was first developed by Yamamoto *et al.* [130], Yamamoto [126] and Madsen [67], who considered compressible pore fluid in a compressible porous medium. A three-dimensional general consolidation equation [4] and storage equation [124] were adopted in these studies, in which only progressive waves were examined. Among these, Madsen [67] considered a hydraulically anisotropic and unsaturated porous bed, whilst Yamamoto *et al.* [130] studied an isotropic medium. Both considered only an infinite thickness. Moreover, Yamamoto [126] investigated soil response in a homogeneous soil of finite thickness under isotropic and partially saturated conditions. However, Yamamoto's solution [126] was cast in a semi-analytical manner that did not have a closed form.

Yamamoto *et al.* [130] concluded that when the stiffness of a porous medium is much smaller 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 larger 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 [67] investigated a hydraulically anisotropic and partially saturated seabed. He found that this had an appreciable effect on the nature of the wave-induced effective stresses in coarse sand. For all soils, the effect of partial saturation on soil response may be significant.

Yamamoto [127] developed a semi-analytical solution for a non-homogeneous layered porous seabed, together with a comprehensive verification using data obtained from Mississippi Delta. Yamamoto [127] pointed out that a layer of concrete blocks had 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 soil.

Okusa [83] used the compatibility equation under elastic conditions and reduced the governing equation of Yamamoto *et al.* [130] to a fourth-order differential linear equation. It is noted that Okusa [83] was based on plane stress conditions, while Yamamoto *et al.* [130] was based on plane strain conditions. Okusa [83] found that the wave-induced pore pressure and effective stresses consisted of two parts. The first depended only on the wave characteristics and the second was related to both the sediment and the wave characteristics. He 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 [18, 39].

Rahman *et al.* [93] summarized 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 idealized case. The major difficulty to analysis the wave-induced soil response in a seabed with variable permeability has been the governing equation includes variable coefficients. By employing the VS function [123], Seymour *et al.* [107] 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 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 [61]. Later, Jeng and Seymour [49, 50] further developed analytical solutions for general soils in seabed of both infinite and finite thickness. They concluded that the relative difference of the wave-induced pore pressure between variable and uniform permeability might up to 23 % 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 and Mase [52] and Kitano *et al.* [53]. However, their one-dimensional model is limited to exponential distribution decay of the permeability, although it provides a simpler formulation than that of [49, 50].

The inclusion of cross-anisotropic soil behavior in the wave-seabed interaction can also be handled analytically. Jeng [30, 32] may have been the first to derive the analytical solution for the wave-seabed interaction in a cross-anisotropic seabed. His numerical results show that the conventional solution with the assumption of isotropic soil behavior may overestimate the pore pressure, but underestimate the effective stresses. Consideration of cross-anisotropic soil behavior is particular important in determining the wave-induced soil displacements. An identical approximation is also proposed by Yuhi and Ishida [134].

A simplified analytical solution for the soil response in a cross-anisotropic seabed was proposed by Yuhi and Ishida [136]. In the model, two parameters related to boundary layer thickness and the stiffness ratio were introduced. However, their model is only applicable in the region near the seabed surface (for example, $|z|/L < 0.02$, L is the wavelength of ocean waves), outside this region, the relative difference between the simplified solution [136] and exact solution [30] is not negligible, as reported in Jeng [37].

2.2.2.2 Boundary-Layer Approximation

Since the direct analytical solution for the wave-induced seabed response is involved with complicated mathematical presentations, especially for a seabed of finite thick-

ness, it is even impossible to have a closed-form solution for a layered seabed. An alternative approximation, the boundary-layer approximation, was proposed by Mei and Foda [73]. The principle of the boundary-layer approximation is to divide the whole soil domain into two regions: inner and outer regions. In the inner region (near the seabed surface, defined by the boundary layer thickness), the 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 [126] for fine sand. However, it may lose accuracy for all soils in unsaturated conditions and for coarse sand under saturated conditions [126]. This shortcoming may be attributed to the solution being only suitable for a seabed with low permeability, for which a scaling was carried out [28]. However, their solutions are more convenient for engineering application due to their much simpler form, compared to those of Hsu and Jeng [26].

Although the boundary-layer approximation proposed by Mei and Foda [73] is a simple yet fairly accurate analysis, it was restricted to low frequency waves only. Huang and Chwang [27] 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 and Song [28] 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 represented the relative stiffness of solid and fluid and another expressed penetrability, whilst the other three revealed Mach numbers for two longitudinal waves and one transverse wave of the porous medium of low soil permeability. However, their solution was restricted to a porous seabed of infinite thickness. Later, Song and Huang [109] further applied the boundary-layer approximation to examine the mechanism of laminar poro-elastic media flow under wave loading.

Recently, Kitano and Mase [51] derived another set of analytical solution to investigate the influence of cross-anisotropic soil behavior on the wave-induced soil response through the boundary-layer approximation. Their model agreed well with the exact solution proposed by Jeng [32] for fine sand.

Besides the direct analytical solution and boundary layer approximation, Sumer and Cheng [112] 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 build-up pore pressure. However, the determination of the shear stress relied on other analytical solutions such as that of Hsu and Jeng [26].

2.2.2.3 Numerical Methods

Numerical methods, including a finite difference method, a finite element method and a boundary element method, is another type of approximation which renders numerical results at discrete points for the wave-induced soil response and seabed instability.

Finite Difference Method Madga [66] developed a one-dimensional finite difference model for the wave-induced pore pressure in a highly saturated sandy bed. He concluded that the time phase in pressure generation is dominated by the degree of saturation, compressibility of the soil skeleton and soil permeability.

Zen and Yamazaki [138, 139] 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.

Finite Element Method Gatmiri [18] developed a simplified finite element model for the wave-induced effective stresses and pore pressure in an isotropic and saturated permeable seabed. Two important conclusions were drawn from his 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 [83]. However, the general trend of pore pressure distribution versus the seabed thickness in [18] was found to be inconsistent [39].

Gatmiri [19] further extended his 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 [18, 19] may have stemmed from the boundary condition used. The lateral boundaries at $x/L = 0, 1$ in his paper, " $v = 0$, $p = 0$ and u free", were used in his model. However, it has been proved that there is a phase lag in soil response in a fully saturated seabed of finite thickness [39]. This implies that the lateral boundary conditions, $v = 0$, $p = 0$, are invalid in a porous seabed of finite thickness. Thus, the numerical results of [18, 19] seem doubtful. In fact, this obstacle can be overcome by using the principle of repeatability [142], as suggested by the author [62].

Thomas [115, 116] developed a one-dimensional finite element method for a two layered unsaturated seabed. His result agreed well with the analytical solutions of Yamamoto [126] and Okusa [83]. It also suggested that the stiffer sediment in the top layer dominated the response of the bottom layer in a two-layered seabed.

With a similar framework, a series of one-dimensional finite element models was developed for the wave-induced seabed response in a non-homogeneous seabed [42, 43, 60, 61]. The results of these numerical models agree well with previous two-dimensional experimental data and analytical solutions. In the models, the permeability and shear modulus are considered to vary with burial depth. [43] have also examined the influence of non-linear wave components on the soil response. In addition, the combined effect of cross-anisotropic soil behavior and non-homogeneous

soil characteristics on the wave-induced soil response was examined by Jeng and Lin [44].

The advantage of the above one-dimensional finite element models is the reduced computational time. However, such one-dimensional models cannot apply to the case with a structure. Thus, a two-dimensional finite element model, was proposed [62] by employing the principle of repeatability [142].

Boundary Element Method Raman-Nair and Sabin [96] proposed a boundary element technique for computing the wave-induced effective stresses and pore pressures within the slope by using Biot's theory of poro-elasticity. The boundary element method was verified with analytical solutions for flat beds. The results indicated that there was no significant difference in the extent of the failure zones for fine and coarse sand, although in some cases the failure zone in fine sand was slightly deeper.

2.2.3 u - p Approximation

Based on Biot's poro-elastic theory [5, 6], Zienkiewicz *et al.* [141] proposed a one-dimensional so-called u - p approximation for waves propagating over a porous media. Their paper provided a simplified approximation for such a problem. Based on the numerical examples, Zienkiewicz [141] concluded that neither the u - p approximation nor the dynamic approximation is required for the case of ocean waves over a seabed. However, in their examples, the compressive wave velocity under ocean wave loading was fixed at 1000 m/s, which is applicable during an earthquake, but may not be a reasonable value for most ocean environments. Thus, the conclusion drawn from the examples was in doubt.

Sakai *et al.* [100] extended Mei and Foda's boundary-layer approximation [73] to examine the effect of inertia and gravity force of the ocean waves on the seabed response, and verified it with results of their numerical model (finite element method). They concluded that the inertia term (acceleration) could be neglected in normal wave conditions except for breaking, whilst the gravity term can practically be ignored.

Later, Sakai *et al.* [99] modified the boundary-layer approximation to take into account the effect of wave-induced bottom shear stress which cannot be neglected in the surf zone. In his paper, it was simply assumed that the amplitude of the wave-induced bottom shear stresses was proportional to the wave pressure, without any phase lag. However, it has been reported that the wave-induced bottom shear stress has a phase lag of 45° to the wave-induced pressure at the seabed surface [23].

Recently, the one-dimensional u - p approximation [141] was further extended to two-dimensional waves over a porous seabed [46, 48]. The seabed was considered to be either infinite or finite thickness. It was found that the relative differences between u - p approximation and quasi-static solution may reach 17 % of the amplitude of wave pressure.

2.2.4 Dynamic Models

The major difference between dynamic analysis and u - p approximation has been the inclusion of acceleration due to pore fluid. With a similar framework to [141], Jeng and his co-workers [8, 38, 47] investigated the effects of dynamic soil behavior on the wave-induced soil response through a two-dimensional analysis. Although their dynamic solution can provide a better prediction of the wave-induced soil response, their solutions were lengthy and difficult to be applied in engineering practice.

Based on the boundary-layer approximation, Huang and his co-workers [9, 25, 28] further developed a series of analytical solutions for waves propagating over a soft poro-elastic bed. Both linear and non-linear wave loading were considered. Their approximation provided a simpler formulation, compared with the full closed-form solution.

Alternatively, based on the governing equations derived by Mei and Foda [73], Yuh and Ishida [135] directly solved the boundary value problem, rather than using a boundary-layer approximation. The seabed was considered to be of infinite thickness. The characteristics of three waves within the soil column, including two compressive waves and one shear wave, was discussed in detail. This work has been later extended to the case of finite thickness [40].

Besides the linear poro-elastic models, Yamamoto and his co-workers [129, 131] carried out a series of studies on the non-linear mechanism and the Coulomb type friction failure for the interaction between waves and porous seabed. With similar framework, the non-linear interaction between waves and soil was examined [128]. Yamamoto and Takahashi [131] applied their solutions to the case of acoustic waves propagating through porous media. Recently, these solutions have also been applied to back calculate the shear modulus of marine sediments [132]. Marine geotechnical engineers have used this procedure, named as Bottom Shear Modulus Profile (BSMP), in field measurements.

Lee *et al.* [56] extended Yamamoto's model with Coulomb-damping friction [129] to a case of finite thickness. They demonstrated the significant of Coulomb-damping friction on the wave-induced soil response in fine sand, not in graveled bed. An identical work has also been reported by Lin [58].

Detailed of comparative evaluation of various analytical solutions for the wave-induced seabed response is given in [8, 38].

2.2.5 Poro-Elastoplastic Models

All aforementioned poro-elastic models have been limited to small deformations, which is an idealized condition. However, large deformations will be a more important issue for engineering problems, especially under the action of a storm. In that case, poro-elastoplastic models are required to provide a better estimation of the soil response. Since a poro-elastoplastic model is more complicated than a poro-elastic

model, especially in the problem of wave-seabed interaction, to date, only a few investigations have been published.

Sekiguchi *et al.* [105] may have been the first to derive an analytical solution for waves propagating over a poro-elastoplastic seabed with Laplace's transformation. Their model clearly demonstrated the difference between poro-elastic and poro-elastoplastic models. Also, overall agreement between their model and centrifugal tests was reported. However, their model was based on the assumption of the seabed thickness being much less than the wavelength, which is questionable [31].

With the non-associative bounding surface plastic constitutive model, Yang and Poonoshasb [133] developed a numerical code to investigate the standing wave-induced soil response in a seabed. Their model demonstrated the significant influence of permeability on the wave-induced seabed response. However, no comparison between elastic and their plastic models was made in their paper. Thus, the importance of plastic model was not observed.

Recently, Li *et al.* [57] established a poro-elastoplastic model for wave propagation in saturated media, using the Drucker-Prager criterion to describe non-linear constitutive behavior of pressure-dependent elastoplasticity for the solid skeleton. Based on their model, they further investigated the stationary discontinuities and flutter instability of wave propagation in a saturated seabed, and concluded that the flutter instability may occur prior to stationary discontinuity only if the deviative derivative effective stress normal to the surface of discontinuity is compressive.

Besides the poro-elastoplastic models, discontinuous deformational analysis (DDA) is another alternative approach for larger deformation. However, the conventional DDA models have been based on total stress analysis, which may be suitable for rock mechanics, rather than wave-soil interaction. Thus, the conventional DDA model needs to be modified by including the pore pressure and become effective stress analysis, as proposed by Jeng *et al.* [45]. The relative difference between discontinuous deformation and conventional continuous approaches was reported to be significant under large wave loading [45].

2.3 Waves Propagating over a Porous Seabed: Theoretical Model (Residual Mechanism)

Based on the analytical solution proposed by Sumer and Cheng [112], Cheng *et al.* [10] developed a finite difference model to investigate the wave-induced build-up pore pressure in marine sediments. They concluded that the pore pressure build-up is more sensitive to shear stress near the seabed surface in a deep soil column. It is noted that the governing equation for the pore pressure used in the model is one-dimensional. However, the shear stress required in their pore pressure model was obtained from two-dimensional Biot's consolidation. This inconsistency may lead to some doubts on regarding their model.

2.4 Waves Propagating over a Porous Seabed: Physical Modeling

Theoretical investigations usually involve some assumptions to simplify the realistic problem to become solvable. However, physical models, including laboratory experiments and field measurements, have commonly been conducted by researchers in the early stage of a series of studies on this topic. From these, a limited number of data can be chosen to verify the theoretical models.

2.4.1 Field Measurements

Using conventional techniques, properties of seabed deposits have been estimated from laboratory tests of bored samples. However, these are usually accompanied by many difficulties such as: (1) disturbances from remolding and swelling caused by sampling; (2) difficulty in controlling the degree of saturation of the sand samples and (3) performing compression tests at low confining pressure in obtaining mechanical properties of the surface sediment on the seabed. Therefore, in-situ measurements are recommended to estimate the properties of seabed deposits.

Bennett [2] measured pore pressure and hydrostatic pressure of silty clay in the Mississippi Delta. Piezometric measurements in submarine sediments at a selected site revealed the presence of high excess pore-water pressures. Its presence was not only due to short-term rapid fluctuations in excess pore pressure during active storm conditions, but also due to long-term changes in excess pore pressure following the passage of Hurricane Eloise through the Gulf of Mexico. Relatively rapid fluctuations were observed in hydrostatic and dynamic pore-pressures during storm activity, with the maximum pore pressure variations measured approximately one-half of that observed in hydrostatic conditions. This suggested an energy loss or damping effect of the wave through the sediments during the storm.

Bennett and Faris [3] implanted a shallow-water piezometer in the Mississippi Delta silty clay to measure pore water and hydrostatic pressures for a period of approximately eight months (March to November 1977). The field measurements indicated that water surface activity by tides and short period waves were observed to produce significant fluctuations in pore pressure.

Okusa and Uchida [85] measured the pore-water pressure in disturbed silty sand sediment at 1.5 m below the sea floor in about 12 m deep water. Concurrently, they monitored wave pressure in the water at about 10 m above the sea floor, over a period 1976–1978 in Shimizu Harbor, central Japan. They reported that the damping for a long-period wave was smaller than for a short-period one. The time lag was relatively clear for the former and not for the latter. The curves of wave pressure were slightly different from the fluctuations of the differential pore-water pressure.

Later, based on the field measurements of pore pressure in submarine sediments under various marine conditions, including mixtures of silt, sand and gravel, Okusa *et al.* [84] concluded that the rate of damping increased with shorter period waves. The pore pressures measured in the sediments decreased faster than that predicted

by the theories available (for example, [67, 130]). This might have come from the existence of pore gas in the sediment or other damping mechanisms.

Okusa [82] has also reported several observations of the wave-induced pore pressure, which were measured in sand and gravel sediment at a depth of about 4 m from the seabed surface in the water about 1 m deep. The results revealed that the wave-induced sea floor pressure was transferred into the sediment with damping, which was slightly greater than that predicted by Madsen [67], coupled with a time lag. Furthermore, permeability and deformation properties of submarine sediments would appear to play an important role in the damping transfer of wave pressure at the sea floor to the pore pressure with a time lag.

Maeno and Hasegawa [69] proposed a new method for predicting physical and mechanical properties of the deposits near the surface of a seabed by means of in-situ measurements of pore pressure in Nabae beach, Japan. They pointed out that low frequency components of the wave-induced pressure fluctuations propagate into the seabed more easily than the high frequency components. Furthermore, the time lag between the pore pressure in the seabed and the wave pressure at the seabed surface increased as the frequency decreased.

Zen and Yamazaki [140] also conducted field observations of wave-induced pore pressure and effective stress at Hazaki, Japan. They concluded that the effective stress in the seabed varied periodically in accordance with the propagation of ocean waves. Furthermore, wave-induced liquefaction was closely related to the upward seepage flow induced in the seabed during the passage of wave troughs.

2.4.2 Laboratory Experiments

Laboratory experiments have been extensively used for estimating the soil behavior under wave action. In general, three different experimental approaches have been used. They are: wave tank experiments, compressive tests, and centrifugal wave modeling. Wave tank experiments have been commonly used by coastal engineers [13, 68, 108, 119], which provide the spatial distribution of the wave-induced pore pressure within a seabed, but lack accuracy in determining soil parameters. Compressive tests have been widely used by geotechnical engineers [138, 139], which can provide a better estimation of pore pressure, but are unable to provide the spatial distribution of pore pressure. Both wave tank experiments and compressive tests have the scale problem under one gravity acceleration environment. Centrifugal wave modeling is a new advanced experimental approach, in which the experiments are conducted under N times gravitational acceleration. Such an experiment does not only provide the identical stress distribution to that in the field, but also provides the spatial distribution of soil response. However, the development of wave experiments in centrifuges is still not mature, and is a challenging experimental and instrumentation task.

The previous experimental approaches are summarized in the following sections.

2.4.2.1 Wave Tank Experiments

There have been numerous investigations for the wave-induced pore pressure based on water tank experiments. Among these, Sleath [108] conducted an experiment to verify his theory. He pointed out that the wave-induced pore pressure was out of the phase with the wave profile by about 10 degrees.

Later, Tsui and Helfrich [119] conducted experiments for loose and dense sands and confirmed the existence of a phase lag. They discovered that the maximum phase lag might reach one-third of the wave period.

Maeno and Hasegawa [68] proposed an empirical equation for the wave-induced pore pressure in some sandy beds, using second-order Stokes wave theory. In this, the pore pressure from the seabed surface to the bottom of the sand layer was expressed as a function of the wave steepness and two experimental parameters, which depended on the permeability of the bed. This empirical equation involved both wave characteristics and drainage conditions of the sandy bed.

Demars [13] measured the wave-induced pore pressure and stresses in a sandy bed. From their laboratory data, they concluded that elastic theories provided a reasonable method for estimating the total vertical and horizontal stresses in a sandy bed in which dilation of the grain matrix is small. Furthermore, their theoretical solution of the total stress for a seabed of infinite depth provided a lower bound for stresses in a sandy bed of finite depth.

Tzang [120] conducted a series of wave tank experiments to study the fluidization of silty soil under wave loading. His experimental results demonstrated the relationship between the build-up pore pressure and fluidization in a soft seabed.

2.4.2.2 Compressive Tests

The compressive test has been commonly used by geotechnical engineers, due to a better estimation of soil characteristics. Zen and Yamazaki [138, 139] also conducted a series of experiments to investigate the wave-induced pore pressure. They reported that the oscillatory pore pressure could be represented by two non-dimensional parameters, namely, coefficients of drainage and propagation. It was also observed that the number of waves did not affect the oscillatory pore pressure significantly. However, the amplitude of input pressure in their model tests is too large, compared with a realistic ocean environment. This is a shortcoming of compressive tests, and has been questioned by coastal engineers.

2.4.2.3 Centrifugal Wave Experiment

Centrifugal modeling is a new technology in geotechnical engineering. It has been widely applied to simulate various geotechnical problems and solute transport phenomena. Sekiguchi and his co-workers [87, 106] may have been the first to attempt

conducting wave experiments in centrifuge. They developed the fundamental framework of the centrifugal wave experiments, and their experimental data has been commonly used for the verification of theoretical results. Later, the technology of wave generation in a centrifuge has been further improved and under better controlled experimental environments [101–104]. The phenomenon of wave-induced liquefaction can be physically observed under multi-gravity acceleration. Recently, centrifugal modeling has further applied to investigate the wave-induced progressive liquefaction [103]. However, sample preparation is a difficult task for centrifugal modeling, while a well controlled wave generation system is another challenge.

2.5 Waves Propagating over a Porous Seabed: Wave Damping and Seepage Flux

2.5.1 Wave Damping in a Porous Seabed

When ocean wave propagates on a porous seabed, not only the wave pressure penetrates into the seabed, but also the soil column modifies the wave characteristics. This phenomenon has attracted attention from ocean engineers since 1958 [15, 17, 20, 24, 78]. However, these early researches were based on the analytical solution with assumption of rigid permeable seabed. Under such a condition, the wave dispersion equation needs to be modified as

$$\omega^2 - gk \tanh kd = -i\omega \frac{k_z}{g} (gk - \omega^2 \tanh kd) \tanh kd. \quad (2.1)$$

It is noted that the modified wave dispersion equation consists of soil permeability and wave characteristics. Thus, the wavelength will be modified by the soil permeability. However, this modification in the wavelength is minor, because the soil model is an un-couple approach. Viscous effects in the wave damping problem, based on the similar assumptions, have been summarized in [16].

Recently, the effects of soil characteristics on the wave characteristics such as wavelength, wave profile and wave pressure have been discussed [34–36, 55]. Among these, both quasi-static soil behavior [34, 36] and dynamic soil behavior [35, 55] have been considered. The new linear wave dispersion equation was derived as

$$\tanh kd = \frac{\omega^2 - gkF}{gk - \omega^2 F} \quad (2.2)$$

where F is the function of soil characteristics. The above equation indicates that the wave characteristics will be modified by soil parameters. Detailed information for wave damping in a porous seabed will be given in Chap. 7.

The phenomenon of wave damping in Coulomb friction seabed was also studied by Yamamoto and others [128, 129, 131]. However, only the damping ratio (k_c/k_r)

was investigated in their studies. Later, the modification of wave characteristics due to soil was examined by Lee *et al.* [56]. They concluded that the response of soil characteristics on the wave field is important in graveled seabeds, which agrees with the conclusions drawn from the solution without Coulomb-damping friction. Detailed comparison of various models for wave damping is given in Lin and Jeng [59] and summarized in Chap. 7.

2.5.2 Wave-Driven Seepage Flux in Sediments

Sediments in bays, estuaries, and in the seabed near river inlets are often contaminated. Many inorganic contaminants (notably heavy metals) do not decompose. Under certain conditions, these accumulated substances can be released back into the receiving body of water through mass transfer processes at the seabed. The mass transfer rate is largely controlled by the seepage flux exchange between the sediment and the seawater [86]. Increased wave action and higher sediment hydraulic conductivity generally cause larger transfer rates. Clearly, quantification of the mass transfer rate is a key factor in water quality modeling.

Mu *et al.* [77] may be the first to investigate the wave driven seepage flux at the interface of water and sediments, based on quasi-static models. The volume of seepage exchange between seawater and seabed driven by the water waves, that is, the volume exchange at $z = 0$. This can be calculated by

$$V_z = \int_{-T/4}^{T/4} \int_{-T/4}^{T/4} \frac{k_z}{\gamma_w} \frac{\partial p}{\partial z} dx dt. \quad (2.3)$$

It is noted that in (2.3) is the “volume displacement” per unit width at interface between water and sediment.

The wave-driven seepage flux in marine sediment was also studied by Jeng *et al.* [41] with a dynamic model. The wave damping was also included. The numerical results concluded that the seepage flux across the seabed boundary is larger in shallow water and for shorter waves. Also, the magnitude of the seepage flux is also affected by the soil type and degree of saturation. In general, it is larger in coarse sands than in fine sands. The framework has been later extended to the case of cross-anisotropic seabed [40].

2.6 Wave-Induced Seabed Instability

Evaluation of seabed instability is an important part of the foundation design for many marine facilities, because some structures might have failed owing to seabed instability and concomitant subsidence. In general, the sea floor instability has been classified into three mechanisms: shear failure, liquefaction and scour [90, 91, 113, 114]. Since the mechanism of scour around coastal structures has been recently

reviewed by Sumer and Fredsøe [113], we only discuss the mechanism of shear failure and liquefaction.

2.6.1 *Shear Failure*

The shear stresses at a point within the marine sediment, induced by gravity forces and storms, may be significant enough to overcome its shear resistance, resulting in seabed instability. This type of seabed instability, referred to as shear failure, may produce a horizontal movement of sediment.

Henkel [21] may have been the first to identify the important role of water waves on submarine landslides. He proposed a total stress analysis for the Birdfoot Delta of Mississippi, based on the principle of limiting equilibrium and the assumption of a circular failure surface under the action of a standing wave. He pointed out that the pressure changes on the bottom were large enough to cause shear failure in soft sediments. Based on this concept, [125] used a finite element method to compute the wave-induced stresses and displacements appropriate to an almost horizontal slope type of failure. However, in the limiting equilibrium analysis, no information concerning the deformation or the extent of failure was available.

Mitchell [75] demonstrated that wave action was instrumental in initiating mass movement of submarine sediments in model studies, but indicated little correlation between the results of model tests and analytical predictions. From this study, they concluded that the driving force in submarine land slides was primarily gravitational, and that a progressive wave train initiated failure by reducing the soil strength.

As pointed by Mitchell [74], cyclic subsurface pressures due to wave action caused remolding and loss of strength in fine grained submarine sediments. The depth of remolding generally increased with subsurface pressure. Furthermore, in sloping offshore sediments, the remolding due to wave action leads to slope instability. The depth of potential slope failure increased with increase subsurface pressure, in general accordance with the relation proposed by Henkel [21].

Bea [1] described failure of a major offshore platform caused by a sea floor slide and hurricane wave forces. As they reported, it was developed during the passage of Hurricane Camille (August, 1969) through South Pass Block 70, Mississippi River Delta. Large soil movements extended to depths in excess of 30 m and lateral soil translations exceeded 1000 m. Three offshore platforms failed as a result of soil movements and hurricane forces. They compared the results of the previous models, such as limit equilibrium, finite element and layered continuum analytical models. Ultimate strength and plastic design methods employing load factors on specified loading conditions were suggested to supplement or replace other elastic design methods and allowable stresses in design of a slide-resistant platform.

Lee and Edwards [54] proposed a regional method to assess offshore slope stability using properties of short core sample analyzed in the laboratory with wave parameters. Gravity cores were taken with conventional oceanographic sampling equipment in which pipes penetrated the sea floor as a result of momentum gained during

a free-fall phase. However, this method was not applicable to all offshore situations, but was shown to be useful in certain simple depositional environments [54].

Rahman and his co-workers [90, 95] using simplified analyses, formulated the necessary conditions for wave-induced instability. The results showed that a seabed with cohesive sediments might suffer from shear failure, leading to slope instability. However, a seabed with non-cohesive sediments was unlikely to become unstable due to shear failure [90].

From a different aspect, Sulisz [111] considered the effect of permeability on the stability of rubble-mound bases. Their approach consisted of theoretical modeling of the interaction of water waves with a caisson-type breakwater and its rubble foundation, and then employing predicted flow field in a stability model to determine the required stone size. Their results showed that the permeability of a rubble-base had substantial effect on its stability. In general, the stability increases with the increase of the rubble-base permeability at intermediate water depths and increase with the decrease of the rubble-base permeability in shallow water. However, the permeability of the armor layer has only little effect on the rubble-base stability.

Despite aforementioned investigations, Hoeg [22] presented a review of key geotechnical issues in offshore engineering. He pointed out trends and needs for improvements in theory and practice related to offshore site investigations, seafloor instability and pipeline safety, single- and multi-based gravity structures, pile foundations and platform anchorage, jack-up platforms and arctic offshore constructions.

Based upon the analytical solution proposed by Hsu and Jeng [26] and Mohr-Coulomb's failure criterion, Jeng [33] investigated the wave-induced shear failure in front of a breakwater. The influence of wave and soil characteristics on the wave-induced maximum shear failure depth was examined through a parametric study. The results are summarized in Chap. 4.

2.6.2 Liquefaction

When excess pore pressure is equal to the downward effective soil weight, liquefaction may occur. This causes a quick sand effect in which external loads cannot be supported. As this occurs, the soil particles in a suspended state may be readily transported as fluid. The liquefaction is an extreme form of seabed instability, which may cause a vertical movement of sediment.

Liquefaction is the state where the material has lost all its structural strength. It behaves as a heavy liquid with no rigidity, and can therefore flow. It has been submitted that when pore water pressure increases until the effective stress becomes zero, then liquefaction can occur in non-cohesive soils.

In summary, two different criteria for liquefied state have been used in the past: the first criterion is based on the concept of effective stress, as given by Okusa [83] for two-dimensional cases, in which a liquefied state is reached when the vertical effective normal stress is greater than the submerged weight of the soil deposits. This criterion was extended to a three-dimensional case by Tsai [117]. The second

criterion is based on the concept of excess pore pressure, as suggested by Zen and Yamazaki [138, 139] for two-dimensional case. This was modified by Jeng [33] to a three-dimensional case.

The possibility of wave-induced liquefaction occurring in saturated seabed sediments was first recognized and analyzed by Bjerrum [7], in connection with the foundation design for deep-water structures in the North Sea.

Nataraja *et al.* [81] suggested a simplified procedure for ocean wave-induced liquefaction analysis. Based on the concepts proposed, they concluded that the existing data on cyclic shear strength of liquefiable soils under seismic loading could be used to estimate the cyclic shear strength under wave loading conditions. Later, Nataraja and Gill [80] summarized the features of the simplified procedure for ocean wave-induced liquefaction analysis and examined the procedure by using data from four projects.

Employing the theoretical solution for a porous seabed of infinite thickness [130], Ishihara and Yamazaki [29] proposed a methodology for evaluating the magnitude of cyclic stress and wave-induced liquefaction on the basis of design storm parameters specified at an offshore location in deep water. This methodology for liquefaction analysis was applied to some typical cases in which several storm conditions were considered. They pointed out that: “For a medium-dense deposit of sand with 70 percent relative density, liquefaction could extend down to a depth of 17.7 m at a location of 14 m water depth.”

Finn *et al.* [14] reviewed previous methods for estimating the stability of an ocean floor under wave loading and the determination of the wave-induced pore-water pressures. They developed two computer programs for computing transient and residual soil response and estimated liquefaction potential in each. The numerical results indicated that the distribution of the transient pore-water pressures, induced by the wave loading and the associated effective stress field, might be investigated by Biot’s theory of poro-plastic soil.

Umchara *et al.* [122] proposed a method to evaluate liquefaction resistance for partially drained conditions. Their results indicated that soil strength increases with partial drainage and can be well represented by the relative density and a coefficient of drainage effect. The authors also suggested that a thin silty layer located at the sub-surface may play an important role in causing liquefaction of the deposits.

Based on the solution for an infinite seabed, Okusa [83] suggested two simple criteria to gauge the liquefaction depth. First, the “liquefied state” was reached when the effective vertical normal stress was greater than the submerged weight of the soil deposit. This criterion is not valid for cohesive soils because the lateral earth pressure was not accounted for. The second criterion was given by that the geostatic pressure being less than that of the wave-induced effective mean normal stress. These criteria were applied to examine the liquefaction potential for marine deposits in the North Sea and Mississippi Delta.

A case study of a damaged breakwater was demonstrated by Zen *et al.* [137]. It was suggested that the wave-induced liquefaction of the foundation with its subsequent slip circle failure was one of the possible causes of the collapse. The case study showed that the liquefaction might be induced by ocean waves only under the

condition that the pore pressure dissipation or redistribution was restricted by the existence of layers with low permeability.

Maeno *et al.* [70], employing the empirical formula of Maeno and Hasegawa [69], discussed the effects of wave steepness on the wave-induced liquefaction depth. They pointed out that the liquefaction depth predicted by the theoretical solution of Yamamoto *et al.* [130] was considerably less than this empirical formula for small waves. In the case of large waves (for wave steepness greater than 0.2) the former is consistent with the latter. They also concluded that wave height and wave period play an important role on the wave-induced liquefaction.

Tsotsos *et al.* [118] developed a numerical model for the evaluation of pore pressure generation and liquefaction potential in the sea floor due to cyclic wave action. The analysis included the development of both transient and residual pore-water pressures, and the simultaneous partial pressure dissipation. The paper revealed that soil permeability had a very significant influence on pore-water pressure generation and liquefaction because high permeability prevented the development of excess pore-water pressure.

Sakai *et al.* [98], using Mei and Foda's boundary-layer approximation [73] examined the wave-induced momentary liquefaction. They concluded that it increases as the soil stiffness ($G\beta$) increases. The maximum liquefaction depth reached around half of the wave height in surf zone conditions.

According to Rahman [90, 91], a fully saturated seabed with non-cohesive sediments could experience liquefaction (associated with progressive build-up of pore pressure) in the region of vulnerable water depth, the liquefaction potential increased as the wave period increased. However, a partly saturated seabed may experience transient liquefaction (associated with oscillatory pore pressure). The liquefaction potential increased with a decrease in degree of saturation and with an increase of wave period. In his paper, the critical values of the relative water depth (d/L) in which the wave-induced seabed instability occurred were also presented in graphs. In fact, the conclusions of his papers were drawn from Okusa's criteria for liquefaction [83].

Jeng [33] examined these two criteria for several different cases, together with the field data from Zen and Yamazaki [140]. He concluded that the criterion suggested by Okusa [83] and Tsai [117] may only be suitable for a seabed with large thickness. He also found that no liquefaction occurs in a saturated seabed, except under the following conditions: (i) very shallow water, (ii) large wave, (iii) a seabed with very low permeability.

Despite the aforementioned theoretical approaches, an alternative approach, probabilistic liquefaction risk analysis, including fuzzy variables, was proposed by Rahman and Jabery [92, 94]. In their study, the combined effect of fuzzy variables is introduced into the basic model by evaluating an average fuzzy factor, and then transforming it into equivalent random variables. Thereafter, the general framework of probabilistic risk analysis is used to formulate the probability of liquefaction. The inclusion of fuzzy variables into the probabilistic liquefaction analysis represents an enhancement of estimation of liquefaction potential in practical circumstances.

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