CHAPTER 243

Standing wave induced soil response in a porous seabed

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ABSTRACT

This paper presents the experimental results of the pore water pressures within the soil under the action of standing waves. Both the spatial and time variation of pore pressures are generally well agreement between the theory and the experiment. The experiments also show some nonlinear phenomenon of the pore pressures within the soil.

INTRODUCTION

When incident waves arrive to a breakwater they will be reflected, then normally or obliquely standing waves are formed in front of the structure. The forces of standing waves play a very important role for designing the coastal structures. There were a lot of theories concerning this subject. It was known from the previous reports (Smith and Gordon, 1983; Silvester and Hsu, 1989) that some breakwaters have failed possibly due to soil instability and trench scouring, rather than structural causes. The investigations of the soil response in front of such a structure under the action of waves are thus become important.

As gravity waves exert cyclic pressures on a sedimentary seabed, they penetrate into the porous medium and induce pore pressure within the soil column. The soil will lose its strength in bearing any load if the induced pore pressures

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become excessive in the soil skeleton. A number of theories have been developed for the wave-induced pore pressure in the porous medium, such as Yamamoto et al. (1978), Madsen (1978), Mei and Foda (1981) and Okusa (1985). There were many previous works investigated the wave-induced pore pressures in the laboratory as well as prototype conditions. However, only the case of the progressive wave was considered.

For the cases in front of a breakwater, Hsu et al. (1993) has developed a general three-dimensional solution of the soil response induced by the short-crested wave system. Tsai (1995) extended to the case of partially reflected waves and evaluated the potential liquefaction under the action of such a wave system. Their solutions can be reduced to the case of two-dimensional standing wave that the incident-wave is normally to a vertical wall. Mase et al. (1994) proposed a numerical model for calculating the soil response under the action of standing waves. This paper shows the experimental results of the standing wave induced pore pressure in the sand bed in front of a breakwater.

EXPERIMENTS

The measurements of pore pressures within the sand bed due to the action of standing waves were conducted in a wave flume. The experimental set-up is shown in Fig. 1. The dimension of the wave flume is 2 m in width, by 2 m in height and by 100 m in length. Sand soil of medium firmness was filled 50 cm in depth and 2 m in length in front of the wall.

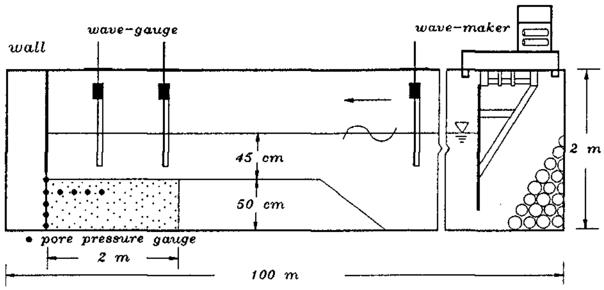

Fig. 1 Sketch of the experimental set-up.

The physical properties of the sand are measured: the specific gravity of soil G_s is 2.64, the mean grain size $D_{50} = 0.187$ mm, the uniformity coefficient $D_{60}/D_{10} = 1.8$, the specific gravity is 2.64, permeability is 1.2 x 10^{-4} cm/sec, the water content w is 0.23 , the porosity $n = 0.38$ and the void ratio e is 0.62. Then the degree of saturation of the soil is 98% which is calculated from the formula $S_r = wG_s/e$.

The water depth d above the mudline is 45 cm. The wave periods T generated in the experiment are varied from 1.0 to 3.0 seconds. The wave heights of incident waves are ranged from 4 cm to 16 cm, from which the sand drifting is not obviously observed.

Under the position of anti-nodal point of the standing wave, the time variation of pore pressures at various distances from the mudline were measured by five pore-water pressure transducers. Also five transducers were placed horizontally below the mudline to measure the spatial variations of pore pressures.

RESULTS AND DISCUSSION

The solution of pore pressures of the standing wave can be reduced from the three-dimensional solutions given by Tsai (1995), from which the solution of the two-dimensional standing wave are given by

$$P = \frac{p_o}{1 - 2\mu} \{(1 - 2\mu - \lambda)C_1 e^{kz} + (1 - \mu) \frac{(\delta^2 - k^2)}{k} C_2 e^{\delta z}\} \cos kx \, e^{-i\omega t} \tag{1}$$

in which P is the wave-induced pore pressure in excess of the hydrostatic condition, k is the wave number, ω is the wave frequency, t is the time; the x-direction is measured normal to the wall, and the z-axis is positive upward from the mudline. The parameter μ is the Poisson's ratio of soil. The parameters δ and λ are coupled with soil properties and wave conditions, they are given by

$$\delta^2 = k^2 [\frac{K_x}{K_z}] - \frac{i\omega\gamma_w}{K_z} [n\beta + \frac{1 - 2\mu}{2G(1 - \mu)}] \tag{2}$$

$$\lambda = (1 - 2\mu)\{k^2 (1 - \frac{K_x}{K_z}) + \frac{i\omega\gamma_w n\beta}{K_z}\}\{k^2 (1 - \frac{K_x}{K_z}) + \frac{i\omega\gamma_w}{K_z}[n\beta + \frac{1 - 2\mu}{2G}]\}^{-1} \tag{3}$$

respectively, in which K_x and K_z are the coefficients of soil permeability in the x- and z-directions, γ_w is the unit weight of the pore water, n is the soil porosity and G

is the shear modulus. While β is the compressibility of the pore fluid which can be related to the apparent bulk modulus of the pore water K' and the degree of saturation S_r (Verruijt, 1969), such that

$$\beta = \frac{1}{K'} = \frac{1}{K} + \frac{1 - S_r}{P_{wo}} \tag{4}$$

where K is the true bulk modulus of elasticity of water and P_{wo} is the absolute pore-water pressure. Coefficients C_1 and C_2 in the equation (1) can be determined from

$$C_1 = \frac{\delta - \delta\mu + k\mu}{\delta - \delta\mu + k\mu + k\lambda} \tag{5}$$

$$C_2 = \frac{k\lambda}{(\delta - k)(\delta - \delta\mu + k\mu + k\lambda)} \tag{6}$$

The above solutions are based on the assumptions that (i) The porous bed is homogenous, unsaturated, hydraulically anisotropic and of infinite thickness, and (ii) The soil skeleton and the pore fluid are compressible.

For comparisons of theory and experiment, some values of parameters are needed besides the physical properties of the sand stated above. These are the stiffness of soil, $G\beta$, in which G is the shear modulus of soil and β is the compressibility of the pore water, and the Poisson's ratio of soil, μ. The value of β can be determined from the relationship of the apparent bulk modulus of the pore water and the degree of saturation S_r (see Yamamoto et al., 1978), from which it is given by $\beta = 3.7 \times 10^{-6}$ N/m^2 for this study. From the best fit made by the series experimental data, it is adopted reasonably by $G\beta = 5$ and $\mu = 0.3$ for the soil of medium firmness. The permeability of soil in the experiment is considered as hydraulically isotropic, i.e. $K_x = K_z = 1.2 \times 10^{-4}$ cm/sec.

Unlike those of progressive waves, the pore pressures under the action of standing waves are spatial and time variations which are shown in Fig. 2 for both theoretical and experimental results measured below the mudline z = −10 cm. The maximum pore pressure occurs at the position under the anti-nodal point of the standing wave ($kx = 0$), while the almost zero value is at the nodal point ($kx = \pi/2$). The pore pressures, under the position of the anti-nodal point, at various depths from the mudline are depicted in Fig. 3. It is found that the pore pressure decreases and a time-phase delay appears as the depth increases. It is noted that there is no

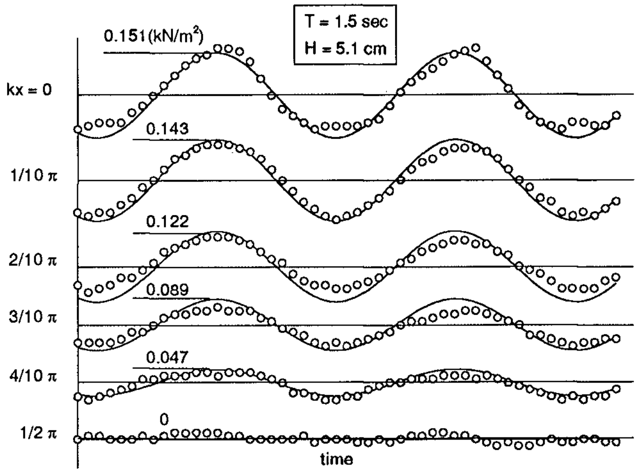

Fig. 2 Spatial and time variations of pore pressures below the mudline z = −10 cm.

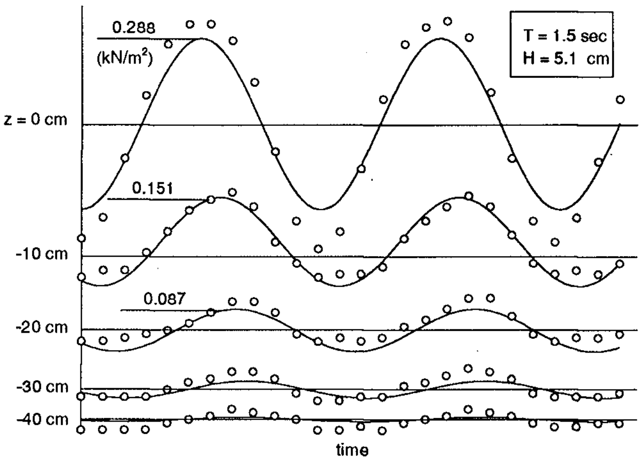

Fig. 3 Time variations of pore pressures at various depths, at $kx = 0$.

phase lag in the saturated soil theoretically, because the coefficient of C_2 in equation (1) is zero.

The depth profiles of pore pressures ratio P/P_o are given in Fig. 4, in which P is the measured amplitude and P_o is the amplitude of bottom pressure of the incident wave estimated by linear wave theory. It is seen from the theoretical curves that the fully saturated condition ($G\beta$ = 0) gives the larger values than those of partially saturated soil. The measured amplitudes at the mudline are smaller than the theoretical values, as the larger wave height actions. In contrary, the measured values near bottom are slightly larger than those of theory. The former may be caused by the energy damping from water column through the porous bed, while the latter is due to the effect of finite thickness of soil in experiments as opposed to infinite thickness of the theory. Nevertheless, the experiments are good in agreement with the theory over all.

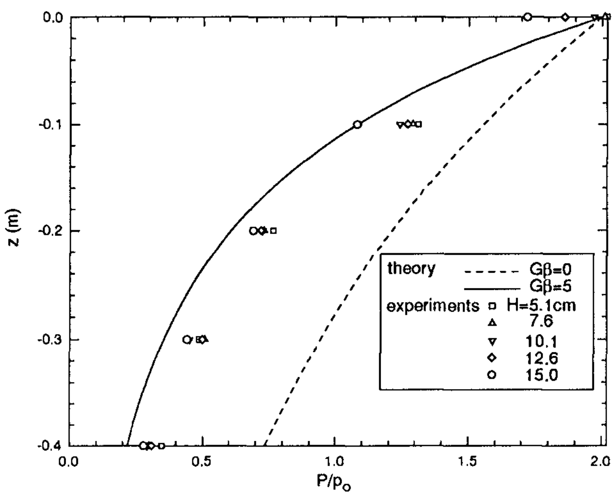

Fig. 4 The depth profiles of pore pressures ratio P/P_o, at $kx = 0$.

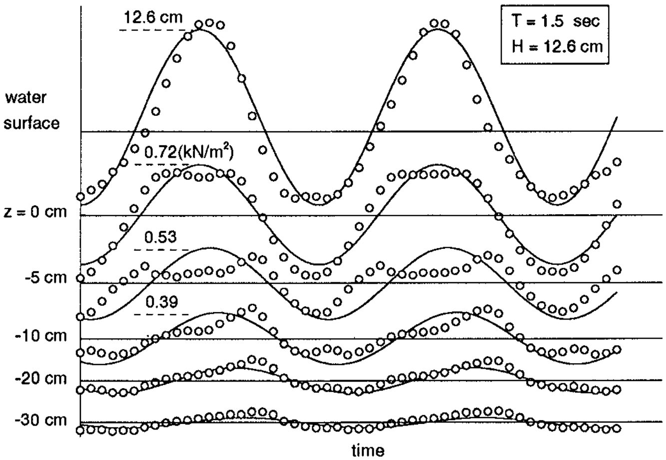

Fig. 5 Double humps of time variations of pore pressures, at $kx = 0$.

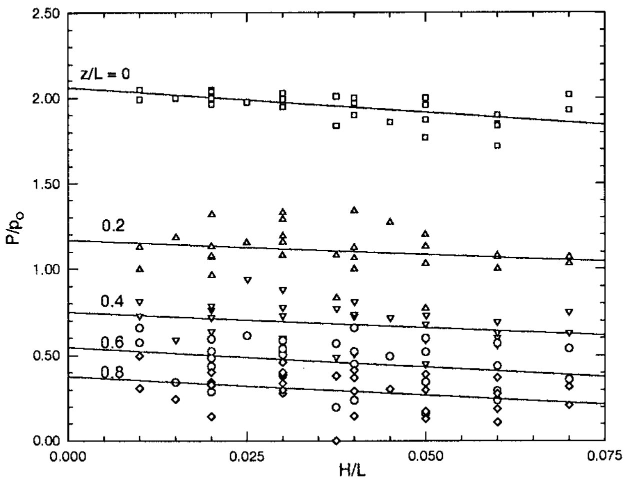

Fig. 6 Effect of the wave steepness on the pore pressures ratio P/P_0, at $kx = 0$.

It was known that the double humps appear in the time-history of pressures of nonlinear standing waves. This nonlinear phenomenon also influences the pore pressures within the soil column, which is shown in Fig. 5. Due to the effect of phase lag, the double humps become asymmetric as depth increases. However, it is obviously seen that the nonlinear effect can not be observed in the linear theory.

The pressure ratio P/P_0 at the mudline and within the soil skeleton is independent on wave steepness theoretically. Nevertheless, it can be seen in Fig. 6 that the pressure ratio slightly decreases as the wave steepness increases, although the plotted points are scatter over a range.

CONCLUSIONS

1. Standing waves are not only important for their forces on a maritime structure, but they have been received attention for their conducive scouring in front of a breakwater.
2. This study presented the experimental results of pore pressures under the action of standing waves.
3. In general, the spatial and time variations of pore pressures induced by the standing waves are well agreement between the experiment and theory.
4. It was found that some nonlinear characteristics of pore pressures exists for the larger wave cases.

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