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# Laboratory study for pore pressures in sandy deposit under wave loading

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ABSTRACT

This paper presents the results of an experimental study of wave-induced pore pressures in marine sediments. A one-dimensional facility was set up with a vertical cylinder, and a 1.8 m thick sandy deposit with 0.2 m of water above the deposit. Unlike the previous experiments of Zen and Yamazaki (1990a), additional static water pressures were added onto the harmonic dynamic wave pressure, which allowed us to simulate a case with a greater depth of water. Furthermore, more pore pressure gauges were installed to better resolve the distribution of the pore pressure. A series of experiments with 3000 wave cycles in each test were conducted under various wave and soil conditions. This allowed examining the influence of the wave and soil parameters on the wave-induced pore pressures as well as liquefaction. The experimental results show the significant influence of liquefaction on sandy deposits in shallow water. Furthermore, the thickness of the sandy deposit was usually considered to be unchanged in theoretical calculations, while the thickness was observed to change periodically with the loadings.



## 1. Introduction

The phenomenon of seabed stability around marine infrastructures, such as breakwaters, pipelines, platforms, and seawalls, has attracted great attention from coastal geotechnical engineers, due to the growing activities in marine environments. Previous studies have shown that the wave-induced liquefaction in the vicinity of the structures has been considered significantly important in the design of coastal structures (Silvester and Hsu, 1989). Furthermore, the pore pressure in a porous seabed is the key factor in estimating the liquefaction (Okusa, 1985; Sumer and Fredsøe, 2002). Therefore, an accurate evaluation of wave-induced pore pressures is desired for engineers involved in the design of the foundations of marine infrastructures.

Numerous theoretical studies for wave–seabed interactions have been reported in the literature (Jeng, 2003). Among these, based on Biot's consolidation theory (Biot, 1941), Yamamoto et al. (1978) proposed an analytical solution for the wave-induced soil response in an infinite seabed. This framework was further extended to the three-dimensional short-crested wave-induced seabed response in a seabed of finite thickness (Hsu and Jeng, 1994) and a layered seabed (Hsu et al., 1995). Later, many theoretical studies for more complicated wave and seabed conditions were reported: for example, cross-anisotropic soil behavior (Jeng, 1997a; Kitano and Mase, 1999); non-homogenous seabed (Jeng and Seymour, 1997; Kitano and Mase, 2001); inertial effects (Jeng et al., 1999; Jeng and Rahman, 2000; Ulker et al., 2009); fully dynamic soil behavior (Jeng and Cha, 2003; Ulker et al., 2009); and, recently, combined wave and current loadings (Ye and Jeng, 2012; Wen et al., 2012; Zhang et al., 2013; Liu et al., 2014).

In addition to theoretical studies, several laboratory studies have been reported in the literature. Experimental studies mainly include two-dimensional wave flume tests (Tsui and Helfrich, 1983; Sumer et al., 1999; Zhang et al., 2009) and one-dimensional compressive tests (Zen and Yamazaki, 1990a,b; Chowdhury et al., 2006) and centrifuge tests (Sassa and Sekiguchi, 1999; Sassa et al., 2001). The purpose of two-dimensional flume experiments is mainly to capture the pore pressure buildup, while the purpose of one-dimensional tests has generally been to capture the response of the soil to an oscillatory pore pressure. In addition, the drawback of two-dimensional experiments (including wave flume tests and centrifuge tests) is the limited number of measurable points in a shallow soil layer (about 3–4 measurement points in a cross section of 10 cm). On the other hand, the advantages of one-dimensional laboratory experiments have been their thick soil layer, which allows us to have more measurable points in the vertical profile of the pore pressures, especially in the region near the seabed surface. Thus, a one-dimensional facility was used in this study to resolve the vertical profile of the pore pressure distributions. All these

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one-dimensional compressive tests (Zen and Yamazaki, 1990a,b; Chowdhury et al., 2006) were performed with 500 cycles.

The aim of this paper is to present the results of a series of one-dimensional tests to have a better understanding of the wave-induced pore pressures in the vertical direction. In this study, 10 pore pressure gauges were installed and more wave cycles were loaded in each tests (3000 cycles), compared to the previous studies. Furthermore, additional static water pressures were added onto the harmonic dynamic wave pressure in this study, to simulate a greater water depth. The experimental results will be first compared with the previous analytical solution (Hsu and Jeng, 1994). Then, the influence of the wave parameters (wave cycles, wave period, and wave height) and soil parameters (relative density and soil saturation) on the wave-induced pore pressure will be investigated. Later, the influence of the wave and soil parameters on sandy deposit liquefaction will be examined. Finally, the variation of the sandy deposit thickness observed in the experiment will be discussed in detail.

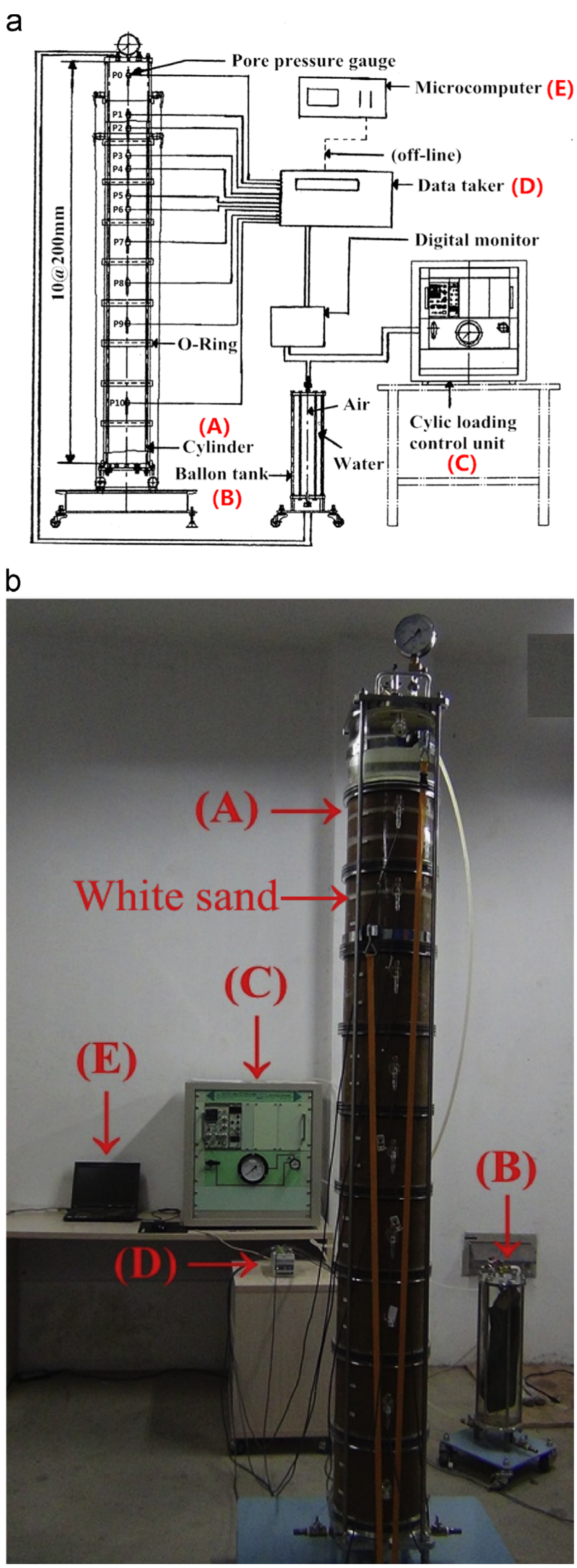


**Fig. 1.** One-dimensional cylinder experimental equipment: (a) schematic diagram of the equipment and (b) photo of the equipment.

## 2. Laboratory experiment

### 2.1. Experimental facility

The one-dimensional cylinder model facility used in this study was improved, based on the one originally designed by Zen and Yamazaki (1990a). The details of this facility are shown in Fig. 1, in which Fig. 1(a) presents the schematic diagram of the equipment and Fig. 1(b) presents a photograph of the equipment as a whole. As shown in the figure, the facility consists of the following five parts: the cylinder (A), the balloon tank (B), the cyclic loading


**Fig. 2.** Grain size distribution curve of sandy deposit.

**Table 1**
Basic properties of sandy deposit.

| | |
|---|---|
| Specific gravity, $G_s$ | 2.67 |
| Maximum void ratio, $e_{max}$ | 0.92 |
| Minimum void ratio, $e_{min}$ | 0.53 |
| Mean grain size, $D_{50}$ (mm) | 0.157 |
| *Loose sand* | |
| Soil permeability, $K$ (cm/s) | $1.8\times10^{-4}$ |
| Shear modulus, $G$ (N/m$^2$) | $1.27\times10^{7}$ |
| Coefficient of consolidation, $c_v$ (m$^2$/s) | 0.00505 |
| Relative density, $D_r$ | 46.7 |
| Average void ratio, $e_{avg}$ | 0.74 |
| *Dense sand* | |
| Soil permeability, $K$ (cm/s) | $1.8\times10^{-4}$ |
| Shear modulus, $G$ (N/m$^2$) | $1.27\times10^{6}$ |
| Coefficient of consolidation, $c_v$ (m$^2$/s) | 0.000253 |
| Relative density, $D_r$ | 73.8 |
| Average void ratio, $e_{avg}$ | 0.63 |
| Soil saturation, $S$ – with de-aired water | 99.6% |
| Soil saturation, $S$ – with normal water | 95.1% |