Wave-induced liquefaction and flow deformation in sand beds

Liquéfacton générée par les vagues et conséquences sur la déformation de l'ecoulement dans un fond sableux

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ABSTRACT: This paper examines wave-induced liquefaction and flow deformation in soil beds below granular slopes based on centrifuge wave tank tests. The experimental program focuses on the measurements of deformation in sand beds undergoing liquefaction, in addition to pore pressure measurements. The development of the liquefaction-related flow deformation in level beds of sand due to wave loading was clearly observed, and this observation led the authors to propose an extended two-fluid-layer theory for describing marked fluidity of the liquefied sand. Then the behaviour of granular slopes resting on liquefiable sand beds was investigated in a separate set of centrifuge wave tank tests. The test results showed that the toe of a granular slope started settling upon the occurrence of liquefaction at shallow soil depth in the free field, and that the rate of settlement of the granular slope increased markedly in association with the spread of liquefied zone.

RÉSUMÉ: Cet article examine la liquéfaction générée par la houle et la déformation de l'écoulement des lits constitués de sols granulaires en se basant sur des essais en centrifugeuse. Précisément, le programme expérimental met l'accent les mesures de déformations des lits sableux en passant par la liquéfaction, en plus des mesures de pressions des pores. Le développement de la relation entre la liquéfaction et la déformation de l'écoulement du fond sableux dues à la houle a été clairement observé, et cette observation a conduit les auteurs a proposer une théorie étendue de celle de la double-couche-fluide pour décrire la fluidité marqué du sable liquéfié. Ensuite, le comportement des pentes granulaires reposant sur les lits sableux et liquéfiable a été étudié dans un ensemble séparé d'essais en centrifugeuse. Les résultats des essais montrent que le haut de la pente commence à se tasser suite à la liquéfaction du sol de surface, et que le taux de tassement de la pente granulaire augmente nettement avec la dispersion de la zone liquéfiée.

1 INTRODUCTION

The performance of permeable granular systems such as detached breakwaters under storm waves has become an increasingly important subject of research in relation to the preservation of sand beaches as well as coastal environments (Isobe, 1994). The wave-induced instability of detached breakwaters on sandy seabeds is one of the long standing problems that needs closer examination (Fig.1). For example, the detached breakwaters on the Niigata Coast of Japan settled by more than 10m due to wave attack over a period of several stormy seasons (Nishida et al., 1985). Possible factors that may be responsible for such a large settlement include wave pressures, oscillating flow through concrete blocks that compose the body of a detached breakwater and currents around the toe of the breakwater. These factors may trigger processes resulting in wave-induced liquefaction, scouring or sediment transport (Fig. 1). The question arises as to how these processes are linked with each other. In order to address this issue, it is important to achieve a full understanding of the wave-induced liquefaction of seabed soil near coastal structures and its consequences.

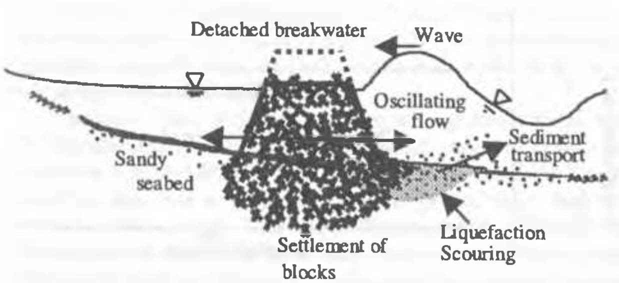

Figure 1. Factors affecting stability of detached breakwater on sandy seabed.

With the above-mentioned in mind, this paper will compare the wave-induced instability of level beds of sand with the performance of sand beds below much coarser granular slopes. Specifically, it starts with a discussion of characteristics of wave-induced liquefaction and flow deformation in level beds of sand, emphasizes the progressive nature of liquefaction and discusses the influence of the presence of granular slopes on the process of instability of seabed soil.

2 WAVE-INDUCED LIQUEFACTION AND FLOW DEFORMATION IN LEVEL BEDS OF SAND

2.1 *Observation of the flow deformation of level beds of sand in a centrifuge*

The process of wave-induced liquefaction of a level bed is essentially of a progressive nature (Sassa & Sekiguchi, 1999). In this section, this process and the flow deformation in level beds of sand will be related from the results of centrifuge wave tank tests. All of the tests were performed under a centrifugal acceleration of 30 gravities. The cross section of the wave tank used is shown in Fig. 2. Silicone oil with a viscosity of 30 cSt was used in order to match the time scaling laws of soil consolidation and fluid-wave propagation. The sand used was silica sand #7 (G_s=2.69, D_{50}=0.14mm). The sand beds formed had an average relative density, Dr, of 34%. The wave paddle was excited at a frequency of 8 Hz. A high-speed CCD camera (250 frames/sec) permitted us to observe the vertical movement of the soil surface during wave loading.

A typical set of experimental results from test WJ12 are discussed here. The measured amplitude of fluid pressure acting on the soil surface, u_0, was 3.0 kPa (Fig. 3(a)). The cyclic stress ratio, $\chi_0(=u_0 \cdot \kappa / \gamma'$; κ: wave number, γ': submerged unit weight of soil), was equal to 0.15, which was severe enough to induce liquefaction (Sassa & Sekiguchi, 1999). The measured time histories of the excess pore pressures at three different soil

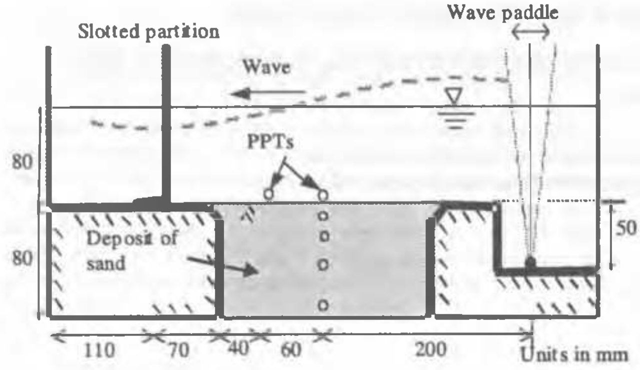

Figure 2. Cross section of a wave tank for use in centrifuge wave testing on soil bed.

depths are shown in Figs. 3(b), (c) and (d). Let us first look at the response at a shallow soil depth (z= -13mm) as shown in Fig. 3(b). After 3 cycles of wave loading, the residual pore pressure reached the level of the initial vertical effective stress σ'_{v0}, indicating the occurrence of liquefaction at this soil depth. During the course of wave loading, the liquefaction front advanced downward (Figs. 3(c), (d)).

The associated vertical movement of the soil surface observed using the high-speed CCD camera is shown in Fig. 4. It is seen that no significant movement occurred before the soil bed underwent liquefaction. However, upon the occurrence of liquefaction at shallow soil depth, the soil surface exhibited a significant vibratory motion. The amplitude of the soil vibratory motion, a_0, increased markedly in association with the progress of the liquefaction front. After the entire soil bed was brought into liquefaction, the amplitude of the soil vibration remained practically constant throughout the subsequent period of wave loading. These observations emphasize the relation between the development of flow deformation in the sand bed and the downward advance of the liquefaction front.

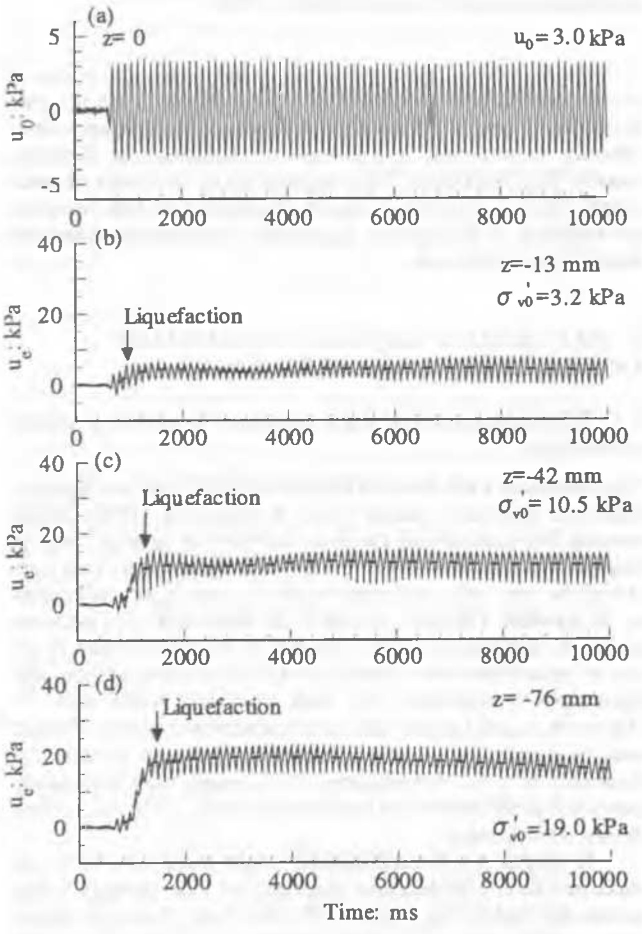

Figure 3. Measured performance of a sand bed under severe wave conditions: (a) Time history of wave pressure acting on the soil surface; (b)-(d) Time histories of excess pore pressures at three different soil depths.

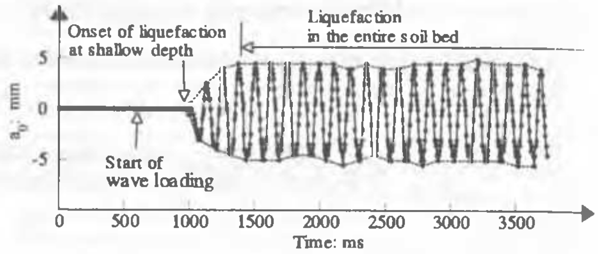

Figure 4. Measured time history of vertical movement of soil surface.

2.2 *Proposal of a theoretical model for describing the development of flow deformation in level beds of sand*

The above-mentioned experimental findings led the authors to propose a simple yet workable analytical model. This model is an extension of Lamb's (1932) two-fluid-layer theory in the sense that the liquefied soil is regarded as an inviscid fluid with a particular mass density and is underlain by a deposit of sub-liquefied, elastoplastic soil that obeys a simple law of cyclic plasticity (Fig. 5). Note that in this model, the liquefaction front is treated as a moving boundary.

Applying the theory of wave propagation in a two-layer fluid (Lamb, 1932) to the two-layer fluid region above the liquefaction front, the amplitude of the vertical displacement of the soil surface, a_0, may be related to u_0 as a function of the location of the liquefaction front, z_L, as follows:

$$a_0 = -\frac{\kappa \cdot \tanh(\kappa z_L)}{\rho_2 \omega^2 + (\rho_2 - \rho_1) N g \kappa \cdot \tanh(\kappa z_L)} u_0 \tag{1}$$

where ω is the angular frequency of waves, ρ_1 is the density of

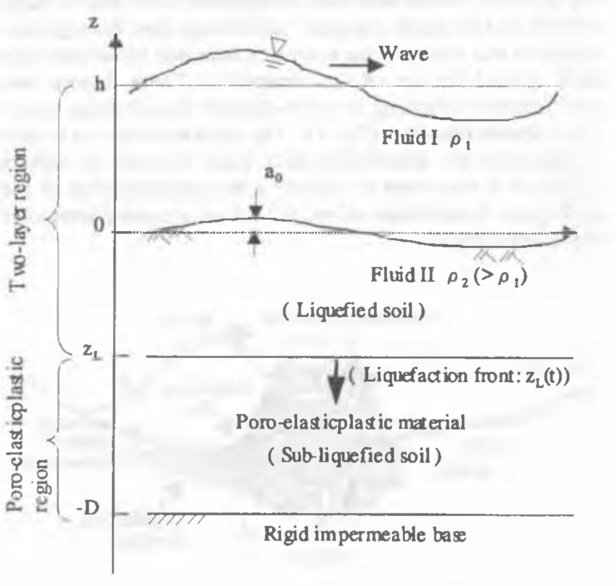

Figure 5. Proposed theoretical model: two-layer fluid underlain by a poro-elasticplastic material

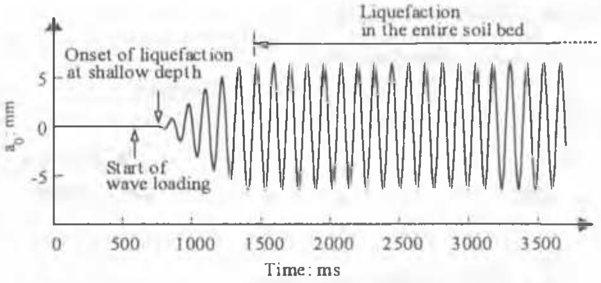

Figure 6. Predicted time history of vertical movement of soil surface.

the exterior fluid, ρ_2 is the mass density of the liquefied soil, z_L is the location of liquefaction front and Ng is the centrifugal acceleration. Note that in this problem the quantity z_L also is an unknown. To determine the evolution of z_L, the development of residual pore pressures in the elastoplastic soil layer has to be solved on the basis of the following storage equation relevant to the elastoplastic soil (Sassa & Sekiguchi, 1999):

$$\frac{\partial u_e^{(2)}}{\partial(\omega t)} = \frac{k_D}{m_v \cdot \gamma_f \cdot \omega} \cdot \kappa^2 \cdot \frac{\partial^2 u_e^{(2)}}{\partial(\kappa z)^2} + \frac{1}{m_v} \cdot \frac{\partial \varepsilon_{vol}^{(2)}}{\partial(\omega t)} \qquad (for\ z_L > z > -D) \qquad (2)$$

where $u_e^{(2)}$ is the residual pore pressure, $\varepsilon_{vol}^{(2)}$ is the plastic volumetric strain, k_D is the coefficient of permeability, m_v is the coefficient of compressibility of the soil skeleton and γ_f is the unit weight of the pore fluid. The second term on the right-hand side of Eq. (2) corresponds to the "source term" due to the cyclic plasticity of the soil. In this study Eq. (2) was solved under the moving boundary conditions ($z_L=z_L(t)$) by adopting an implicit finite-difference scheme.

The proposed model can predict the vertical displacement of the fluid/soil interface during wave loading. The predicted displacement is shown in Fig. 6. Note that the amplitude of the soil vibration increases markedly with time in accordance with the downward advance of the liquefaction front. The predicted amplitude of the soil surface vibration, when the entire soil bed is in a state of liquefaction, is 6.4 mm. This value is consistent with the measured value of 4.5mm, emphasizing the high fluidity of liquefied soil.

3 LIQUEFACTION-RELATED FLOW DEFORMATION IN SOIL BEDS BELOW GRANULAR SLOPES

The wave-induced liquefaction and flow deformation in soil beds below granular slopes will subsequently be discussed based on centrifuge wave tank tests. The aim of the experiments was to gain an insight into the wave-induced instability of detached breakwaters on sandy seabeds.

The wave tank tests were performed under a centrifugal acceleration of 30 gravities. The properties of gravel used to form a granular slope were as follows: Gs = 2.7, D_{50}= 3 mm. The gradient of the granular slope formed was 1: 2.7. The other test conditions were kept the same as in the wave tank tests on the level beds of sand described in Section 2.

The wave field with the presence of a granular slope was first investigated by conducting a separate series of centrifuge wave tests with the fluid and the granular slope overlying a rigid base. The distributions of measured wave pressures on the rigid base are shown in Fig. 7 for two different phases. In these figures, the dotted lines represent wave pressures calculated using small-amplitude wave theory. It is seen that the measured results conform well to the theoretical lines in the range between x = 0 and x = 200 mm. The measured reflection coefficient of the granular slope was equal to 0.26. Considering these observations,

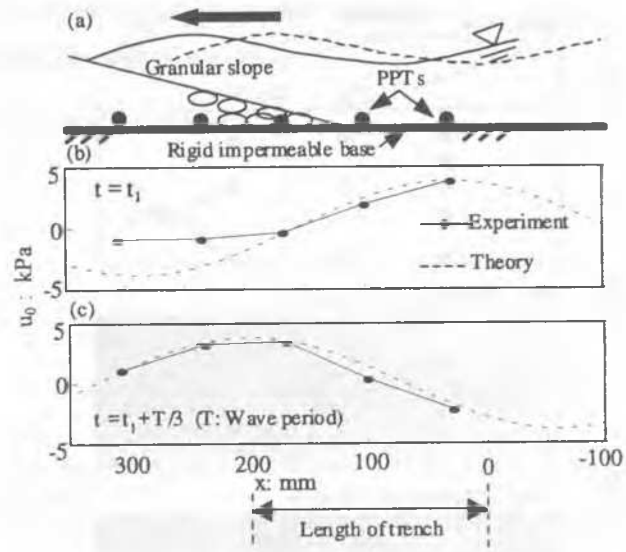

Figure 7. Measured distributions of wave pressure on a rigid base outside and within a granular slope.

it can be said that the wave field concerned approximates to a travelling-wave field.

Then, three wave tank tests (test Nos. WJ13, WJ14, WJ15) with the same χ_0-value of 0.19 were performed on the three identical sand beds that were overlain by granular slopes. The measured excess pore pressures in these tests were compiled to give an idea as to the spread of the liquefied zone during the wave loading (Fig. 8). In this figure, the distribution of excess pore pressure ratios ($u^{(2)}/\sigma_{v0}'$) in the foundation soil is shown in terms of the gray levels indicated. The initial effective vertical stress, σ_{v0}', was estimated from the elasticity theory with consideration of the effective weight of the granular slope.

It is seen from Figs. 8 (b), (c) and (d) that the process of liquefaction in the free field was essentially the same as that observed in the level beds of sand. In short, the liquefaction front advanced downward with increasing wave-loading cycle. However, in the sand bed below the granular slope, the liquefied zone spread two-dimensionally, except for the soil directly below the granular slope. This part of soil remained sub-liquefied due to the overburden effect that resulted from the effective weight of the granular slope. While wave loading was continued, the excess pore pressure started dissipating gradually in the soil directly below the granular slope, thereby reducing the extent of the liquefied zone (Fig. 8(e)).

The corresponding process of flow deformation in the foundation soil below a granular slope is shown in Fig. 9. This figure was constructed based on a typical set of deformation measurements in test WJ15. The dots marked on Fig. 9(a) represent the locations of the coloured sand markers that were used for visual observation. In Figs. 9(b), (c) and (d), it is noteworthy that the interface between the granular slope and the sand surface remained sharp. This observation indicates that essentially no transport of sand particles through the granular slope occurred. Therefore, the significant settlement of the granular slope as shown Fig. 9(d) may be ascribed to the liquefaction-related flow deformation that occurred in the foundation sand.

It is important here to note that the flow deformation manifested itself in the form of two-dimensional, residual shear deformation. A closer examination of its development tells us that the residual shear deformation initially concentrated in the soil below the toe of the granular slope and then became severe in wider areas in association with the spread of the liquefied zone.

Interestingly, the rate of development of such flow deformation was much slower than the rate of development of the liquefied zone. This observation suggests the existence of a certain complex form of resistance of the liquefied soil to shearing, calling for future studies focusing on the physics of liquefied soil.

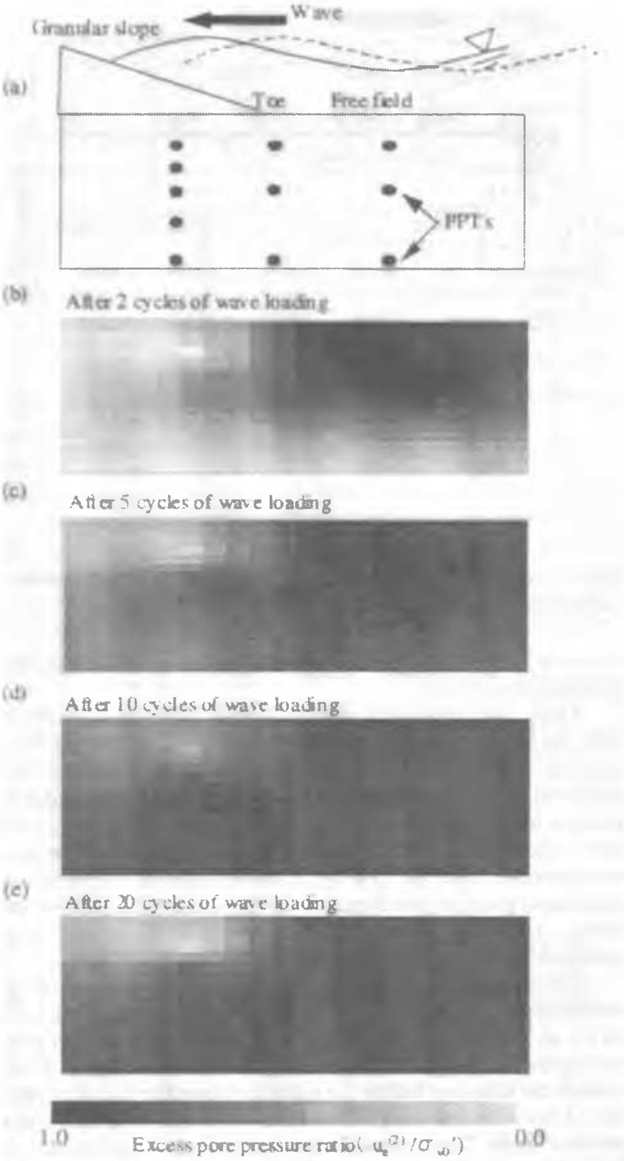

Figure 8. Measured spread of liquefied zone due to wave loading.

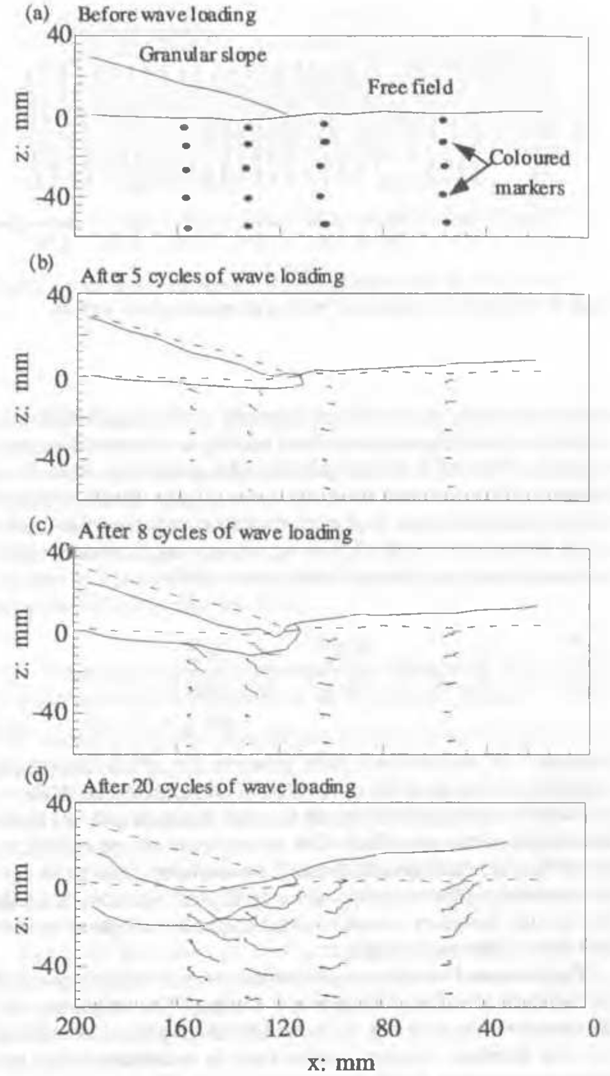

Figure 9. Measured flow deformation of a sand bed below a granular slope due to wave loading.

4 CONCLUSIONS

The characteristics of wave-induced instability of the level beds of sand with and without granular slopes have been compared, in terms of pore pressure measurements as well as deformation measurements. The principal results obtained may be summarized as follows:

(1) The progressive nature of liquefaction is well reflected in the flow deformation of the liquefied soil. In fact, when liquefaction occurred, the sand surface started vibrating and the amplitude of the soil vibration increased markedly with the downward advance of the liquefaction front.

(2) These observations led the authors to propose an extended two-fluid layer model. The proposed model can capture the progressive nature of liquefaction and the high fluidity of the liquefied sand.

(3) The spread of the liquefied zone in the free field outside the toe of the granular slope was essentially the same as that observed for the level beds of sand without granular slopes. However, in the soil bed below the granular slope, the liquefied zone spread two-dimensionally.

(4) The residual shear deformation in the soil below the granular slope developed at much slower rates than the rates of spread of the liquefied zone. This observation may relate to a form of shear resistance of the liquefied soil undergoing plastic flow.

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