



## Letter

## A modified criterion for wave-induced momentary liquefaction of sandy seabed



Wen-Gang Qi, Fu-Ping Gao\*

Key Laboratory for Mechanics in Fluid Solid Coupling Systems, Institute of Mechanics, Chinese Academy of Sciences, Beijing 100190, China

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## ABSTRACT

The assessment of the wave-induced soil liquefaction plays a key role in the geotechnical design for offshore foundations. The underlying shortcomings of the existing momentary liquefaction criteria are identified and clarified by mechanism analyses and the recent field observations. A modified criterion for the wave-induced momentary liquefaction of a sandy seabed is given to describe the vertical pore-pressure distributions. An improved approximation of the momentary liquefaction depth is further presented. Parametric study of the effects of the saturation degree of soils indicates that this modification is significant for the evaluation of wave-induced momentary liquefaction.

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Liquefaction is the act or process of transforming any substance into a liquid. In the offshore environments, soil liquefaction may occur and the soil particles would become unbound. The resulting loss of the soil strength could produce catastrophic consequences, such as sinking and floatation of pipelines, failure of breakwater, and instability of offshore foundations. Thus in offshore engineering practices, the occurrence of soil liquefaction and its effect on the structures should be well assessed.

Based on the observations in laboratory experiments and field measurements, two mechanisms for wave-induced pore-pressure response have been identified [1], which are oscillatory pore-pressure and residual pore-pressure. Soil liquefaction can be generated by either the oscillatory or residual pore-pressure with different mechanisms. Oscillatory pore-pressure is found to propagate into seabed with certain damping and phase lag. At the wave-trough phase, wave-induced pore-pressure exerts an upward seepage force onto the particles and can momentarily liquefy the sandy soils if the seepage force is larger than the submerged weight of the soil particle [2,3]. Residual pore-pressure refers to the build-up of excess pore-pressure caused by contraction of the soil under the action of cyclic wave loading similar to the earthquake-induced pore-pressure accumulation. Under the continuous action of the cyclic wave loading, the excessive pore-pressure may build up gradually and the soil is liquefied when the residual pore-pressure exceeds the effective overburden pressure.

Much attention has been devoted to calculating the pore-pressure response and deducing the liquefaction depth, mostly based on Biot's theory for poro-elastic media [4]. Some approximations for the prediction of wave-induced liquefaction potential have been previously proposed for engineering practice [2,5,6]. However, some underlying shortcomings of the momentary liquefaction criteria still remain, which could generate unreasonable results in assessing the momentary liquefaction depth. In this study, the pore-pressure based liquefaction criteria are discussed and some underlying shortcomings are clarified. A modified momentary liquefaction criterion is proposed and pore-pressure distribution is calculated correspondingly, which provides a simple approximation of momentary liquefaction depth.

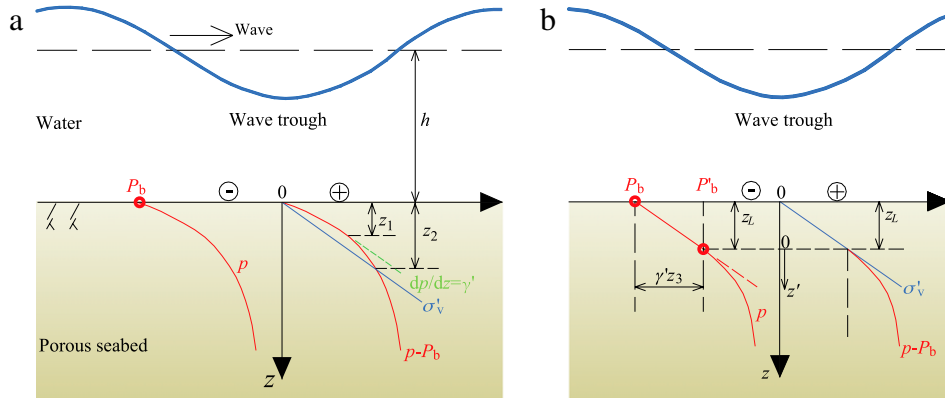
An appropriate liquefaction criterion is crucial to evaluation of the liquefaction depth of soils under the wave loading. The criterion commonly adopted for residual liquefaction is [7]  $p_{res} = \sigma'_{v0}$ , where  $p_{res}$  represents the residual pore-pressure and  $\sigma'_{v0}$  denotes the initial vertical effective stress, which can be calculated by  $\sigma'_{v0} = \gamma'z$ . Here  $\gamma'$  is the submerged unit weight of the soil and  $z$  is the soil depth. This criteria for residual liquefaction can be easily comprehended by viewing the balance of forces acting on the liquefied soil layer during the event.

Regarding momentary liquefaction, Jeng [8] summarized three kinds of pore-pressure-based criteria. It is concluded that the liquefaction criteria based on the concept of effective stress [9,10] would not be applicable due to inaccuracies in determining the effective stress at shallow soil depth. A liquefaction criterion was ever suggested by Zen and Yamazaki [11] for the two-dimensional (2-D) case:

$$\sigma'_{v0} \leq p - P_b \quad (1)$$

\* Corresponding author.

E-mail addresses: [qwq1989@foxmail.com](mailto:qwq1989@foxmail.com) (W.-G. Qi), [fpgao@imech.ac.cn](mailto:fpgao@imech.ac.cn) (F.-P. Gao).



**Fig. 1.** Distribution of pore-pressure response and definitions of momentary liquefaction depths: (a) employing Eqs. (1) and (2); (b) employing Eqs. (3) and (12) ( $z_1$ : liquefaction depth calculated with Eq. (2),  $z_2$ : liquefaction depth calculated with Eq. (1),  $z_L$ : liquefaction depth considering the redistribution of pore-pressure in the liquefied soil).

where  $P_b$  is the wave pressure at the seabed surface (negative value at wave trough phase),  $p$  is the wave-induced oscillatory pore-pressure in the soil.

The physical nature of the momentary liquefaction is the vanishing of the contact between the soil particles due to the seepage force resulting from the pore-pressure gradient along the soil depth. When the soil is liquefied, the vertical seepage force exceeds the submerged weight of the soil particles [12]. Accordingly, another criterion for momentary liquefaction was proposed by Xu et al. [13] as:

$$\gamma' \leq j_z \quad (2)$$

where  $j_z = dp/dz$  (pore-pressure vertical gradient) is the seepage force acting on the soil skeleton.

Figure 1(a) gives the distribution of the pore-pressure response calculated from elastic solution [14]. The distributions of the excess pore-pressure  $p$  and the vertical effective stress  $\sigma'_v$  are also shown to illustrate the momentary liquefaction depth. The liquefaction depth determined with Eq. (1) ( $z_2$  in Fig. 1(a)) is not equal to that determined with Eq. (2) ( $z_1$  in Fig. 1(a)), as obviously shown in Fig. 1. The liquefaction criteria expressed with Eq. (1) are not equivalent to that expressed with Eq. (2). This is due to the fact that Eq. (1) cannot be derived by directly integrating Eq. (2) from 0 to  $z_2$ , and Eq. (2) is essentially an inequality rather than equality.

When the liquefaction criterion expressed with Eq. (1) is adopted, the soil particles located from depth  $z_1$  to  $z_2$  are practically in a partially-suspended state but not fully liquefied because Eq. (2) is not satisfied (see Fig. 1(a)). Thus, the liquefaction depth determined with Eq. (1) is inaccurate. If the liquefaction criterion expressed with Eq. (2) is employed, the soil particles located from depth 0 to  $z_1$  will be in an over-suspension state, and a resultant upward movement would be expected because a vertical net force would be exerted ( $\gamma' \leq j_z$ , see Fig. 1(a)). Nevertheless, once the soil have been liquefied, it will behave like a fluid and thus no more excess pressure gradient can be sustained in it except the essential one (seepage force) for maintaining the soil liquefied. Thus the over-suspension state of the soil particles located from depth 0 to  $z_1$  is unlikely to occur.

In a field study of momentary liquefaction, Mory et al. [3] observed that the pressure difference between two pore-pressure sensors located at the mudline ( $z = 0$ ) and the depth of  $z = 0.3$  m below mudline did not typically exceed the critical value for liquefaction to occur (breaking the static equilibrium of the sediment layer contained between two sensors) during all the recorded momentary liquefaction events. The explanation provided is that when momentary liquefaction occurs liquefaction produces a dilated suspension and the pore-pressure is rapidly adjusted to the

value on top of the soil. This observation provides some evidence for the conclusion that the over-suspended state of the soil particles in the liquefied layer is not likely.

According to the aforementioned discussion and the field observation of pore-pressure distribution in the liquefied layer, it can be reasonably assumed that the soil particles in the liquefied layer should be in an exact suspended state (neither over-suspended nor partially-suspended). Thus following conditions should be satisfied at any position in the momentary liquefaction soil layer

$$\sigma'_{v0} + P_b - p = 0 \quad (0 \leq z \leq z_L) \quad (3a)$$

$$\gamma' - j_z = 0 \quad (0 \leq z \leq z_L) \quad (3b)$$

where  $z_L$  is the depth of the actual liquefaction soil (see Fig. 1(b)). In fact, Eqs. (3a) and (3b) are equivalent, and Eq. (3a) can be derived by directly integrating Eq. (3b) from 0 to  $z_L$ . Under this assumption, the liquefied soil acts like a heavy liquid and there is no extra pressure gradient in this liquid.

Below the liquefied soil layer, the pore-pressure distribution can be described on the basis of poroelastic solutions [14]. It was pointed out by Zen and Yamazaki [15] that for the case of relatively long waves wave-seabed interaction can be analyzed with a one-dimensional (1-D) approach because, under the action of long period waves, the flow and deformation in the horizontal direction are typically negligible because the steepness of the wave is relatively low. Chowdhury et al. [16] also showed that the 1-D solution of pore-pressure can reasonably be used for the purpose of momentary liquefaction analysis of the seabed under waves, particularly under shallow water wave condition. For 1-D analysis of wave-induced seabed response, Chowdhury [17] presented the following governing differential equations by assuming pore fluid and soil mass both to be compressible and adopting Darcy's law

$$\left( \frac{1}{m_v} + \frac{1}{n\beta} \right) \frac{\partial^2 u}{\partial z^2} - \frac{1}{n\beta} \frac{\partial^2 w}{\partial z^2} = 0 \quad (4a)$$

$$\left( \frac{\partial^2 u}{\partial z^2} - \frac{\partial^2 w}{\partial z^2} \right) = - \frac{\gamma n \beta}{k_s} (i\omega) w \quad (4b)$$

where  $m_v = \frac{1}{E} \left( 1 - \frac{2\nu^2}{1-\nu} \right)$  is the volumetric compressibility of soil,  $E$  is the soil elastic modulus,  $\nu$  is the Poisson ratio of the soil,  $n$  is the soil porosity,  $\beta = \frac{1}{K'}$  is the compressibility of pore water,  $K' \left( \frac{1}{K'} = \frac{1}{K} + \frac{1-S_r}{P_0} \right)$  is the apparent bulk modulus of pore-water,  $K$  is the true bulk modulus of water,  $S_r$  is the degree of saturation,  $P_0 = \gamma h$  is the absolute hydrostatic pressure,  $\gamma$  is the unit weight of the water,  $h$  is the water depth,  $u$  denotes the displacement of soil skeleton,  $w$  denotes the displacement of pore water,  $k_s$  is the

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