

## Article

# Experimental Study on Mechanism of Wave-Induced Liquefaction of Sand-Clay Seabed

Jun Zhang <sup>1</sup>, Qin Jiang <sup>1,\*</sup>, Dongsheng Jeng <sup>2</sup>, Changkuan Zhang <sup>1</sup>, Xindi Chen <sup>1</sup> and Lizhu Wang <sup>3</sup>

<sup>1</sup> College of Harbor, Coastal and Offshore Engineering, Hohai University, Nanjing 210098, China; joezhang@hhu.edu.cn (J.Z.); zhangck@hhu.edu.cn (C.Z.); chenxindi1991@hhu.edu.cn (X.C.)

<sup>2</sup> School of Engineering and Built Environment, Griffith University Gold Coast Campus, GoldCoast 4222, Australia; d.jeng@griffith.edu.au

<sup>3</sup> School of Marine Science and Engineering, Nanjing Normal University, Nanjing 210046, China; wlzh2026@163.com

\* Correspondence: qjiang@hhu.edu.cn; Tel.: +86-1395-161-7121

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**Abstract:** In this study, a series of laboratory experiments for the response of wave induced clay-sand seabed were carried out to clarify the mechanism of liquefaction of clayey seabed. The experiments were conducted in an 80 m long wave flume. In the tests, the sand-clay beds were mixed with various clay contents (CC) from 0.5% to 15% and were tested for given wave conditions. The pore water pressure and the water elevation were measured in each test. Soil properties tests and scanning electron microscope (SEM) experiments on different seabed samples were carried out to further explore the mechanism of liquefaction. The experimental results indicated that the amplitude and accumulation of the excess pore water pressure (EPP) varied with different CC in the sand-clay bed. With the introduction of CC, micro-structure and properties (such as permeability and compressibility) of bed soils changed. Sand-clay bed presented more susceptibility to liquefy compared with pure sand bed. CC promoted seabed liquefaction, even if the added amount was very small (CC is 0.5%), however when CC exceeded a certain value (10% in this study), the mixed bed will not be liquefied. This phenomenon can be well explained by the micro-structure of sand-clay bed. CC within a sandy seabed, does not only affect the permeability, but also change the compressibility of seabed soils. For example, the microfabric of seabed vulnerable to liquefaction is loose. Clay aggregations generally gathered at the sand particle contact points. This microfabric is easily compressed under wave loads and allowed pore water to flow, resulting in the accumulation of pore water pressure. On the other hand, the microfabric of seabed that was resistant to liquefaction appeared to be more compact. Due to clay-filled gaps between the sand particles, the pore water is more difficult to flow when seabed was compressed. Furthermore, the tendency of seabed liquefaction is closely related to CC.

**Keywords:** liquefaction; clay content; sand; waves; microfabric; mechanism

## 1. Introduction

Wave-induced liquefaction of seabed is an important phenomenon in coastal engineering, because it will cause reduction of effective stress between the soil grains, and then uneven settlement of foundation caused by the reduction will result in the damage of structures. As reported in the literature, there are two main mechanisms of seabed liquefaction under waves: residual vs. momentary liquefaction. Residual liquefaction is associated with the accumulated pore-water pressure gradient, whereas the momentary liquefaction is associated with the amplitude of pressure oscillations [1]. For most of the coastal zones, seabed soils are generally classified into three categories, i.e., sand, silt

and clay. In the past 30–40 years, the phenomenon of wave induced liquefaction has been extensively investigated and numerous factors affecting this process were studied on non-cohesive sand and silt seabed [2–6]. To date, only limited research has been carried out on wave induced clayey soils because of the widely accepted supposition that clayey soils are in general non liquefiable [7].

The phenomenon of liquefaction of clayey soils has been observed during earthquakes [8–10]. The effects of several factors on the seismic-induced liquefaction of clayey soils, such as cyclic shear stress and frequency [11,12], initial static shear stress [13], and the over-consolidation ratio [14] have been relatively well-studied. Recently, Gratchev et al. [7] investigated the mechanism of liquefaction of soils with different clay contents at the microscopic level. They also examined the influence of CC and soil plasticity on the clayey soils, and concluded that the liquefaction potential of soil was strongly related to specific particle arrangements, and the open microfabric, in which clay aggregations at the sand particle contact points is vulnerable to liquefaction. Moreover, it was found that the presence of a small amount of bentonite (<7%) could cause rapid liquefaction, while a further increase in bentonite content (>11%) lead to the opposite effect of raising soil resistance to liquefaction.

The phenomenon of liquefaction of clayey soils was found after storm surge [15]. However, there are only a few works available in the literature regarding the behavior of clayey soils under wave actions, which requires further clarification. To the authors' best knowledge, among the limited studies on clayey soils, most of them focused on the response of seabed, e.g., stress and strain in the bed [16], seabed scouring [17,18] and seabed liquefaction [19], with pure clay and sand-clay mixture. Recently, Kirca et al. [20] conducted a series of wave flume experiments to examine how different CC influenced the process of wave-induced liquefaction. In their study, only blue clay (commercially available in Denmark) was used. Their experimental results showed that the susceptibility of silt to liquefaction was increased with increasing CC up to 30%, beyond which the mixture of silt and clay was not liquefied. This work focused on the wave-induced pore pressure response of silt-clay bed. Later, Liu and Jeng [21,22] studied influence of clayey soils on liquefaction by a one-dimensional experimental. In their experiments, a vertical cylinder was set up with a 1.8 m thick deposit and 0.2 m thick water depth plus the static water pressure of 5 m water depth. Three different clays, kaolin, illite and bentonite, were considered in their tests. It has been confirmed that the deposit will become prone to liquefaction with the increase of CC, when CC up to 33% for kaolin-sand and illite-sand, 16% for bentonite-sand, the mixture will almost never liquefy. A balloon tank was used in this study to ensure that wave condition close to real sea could be tested. Depth of seabed liquefaction was investigated in this study. Chavez et al. [23] performed three sets of experiments to reproduce the sinking of structures due to liquefaction on the clayey soils. The threshold for bed composition was determined that the soil of 40% or more CC may liquefy with a high initial water content. Moreover, the distribution of pore water pressure in the seabed under the structure was also analyzed. The study also pointed out the effect of initial water content on liquefaction of composite soils. High initial water content led to a different threshold of sand-clay for liquefaction compared to Gratchev et al. [7], Kirca et al. [20] and Liu and Jeng [21,22] where the similar median particle size of the sand was used. Their researches provided guidance for the effect of clay content on the wave-seabed interaction.

In addition, Gratchev et al. [7] explained the mechanism of liquefaction of clayey soils on the basis of research findings of Osipov et al. [24], Georgiannou et al. [25] and Ovando Shelley and Perez [26]. Addition of clay promoted the formation of loose structures as the clay particles reduced the number of contact points between the sand grains. Micro-structure of clayey soil was observed before, during and after cyclic loading and it was found that soil microstructure was not ruptured during the shear process and disruption of some structural bonds (named as "clay bridge" in Gratchev et al. [7]) was followed by their rapid restoration. Gratchev et al. [7] considered that an open microfabric (loose structure) with low strength "clay bridges" and the low plasticity created the vulnerability to liquefaction. However, this theory cannot fully explain the experimental phenomena that the seabed with 0.5% CC could rapidly liquefy which the seabed was almost no plasticity and had the same permeability as sandy bed.

Based on the aforementioned researches of seabed samples by SEM, a hypothesis was proposed to explain the mechanism of clayey seabed liquefaction.

Natural seabed commonly contains various types of soil particles, especially clay, and a small amount of clay in the seabed could lead to a significant impact on its dynamic behavior. To date, most of the researches mainly focused on the critical value of CC that liquefaction would not occur when the CC exceeded it [7,20–22]. However, there are a few aspects concerning whether CC will affect seabed when it is very small. Both Gratchev et al. [7] and Kirca et al. [20] tested the seabed with 5% CC (sand-clay and silt-clay seabed), but they didn't consider the seabed with less than 5% CC. It is helpful to understand the liquefaction mechanism of clayey soils with studying seabed with very low CC. Therefore, this study endeavored to understand the influence of CC on the dynamic response of sand-clay bed. Laboratory experiments were carried out by changing of the concentration of clay from 0.5% to 15% in the mixed bed. The pore water pressure and the corresponding wave height were measured and analyzed.

## 2. Experimental Setup

### 2.1. Wave Flume

The experiments were conducted in a wave flume, 80.0 m long, 0.5 m wide, and 1.5 m high, with a pit filled with mixed clay-sand seabed for the investigation of seabed response (Figure 1). Waves were generated by a piston-type wave-generator located at upstream end of the flume. At the near side of the wave paddle, a fixed bed with 1:10 slope adjacent to a 7.5 m long platform was set in front of a soil trench. Behind the sand trench, a 6 m long platform was set to retain the trench. The soil trench, 0.25 m deep, 1.0 m long and 0.5 m wide, is used to contain the mixed clay-sand seabed. A sloping wave absorber was located at downstream end to eliminate wave reflection. In total, 3 wave gauges (Yu Fan Technology Co, Chengdu) were set up in the flume, one is in front of the trench to measure the incident wave height, other one is behind the trench to measure the wave height after passing through the soil bed and the third gauge is located above sensors.

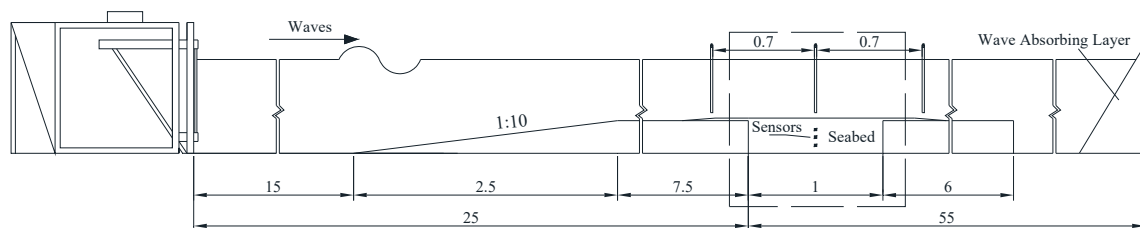


Figure 1. Wave flume setup (unit: m).

### 2.2. Soil Trench

Pore-water pressure sensors (HC-25, RuiHeng Chang Tai Technology Co, Beijing) were located in the middle of the trench at 3 different depths (Figure 2),  $z = 0.09, 0.16$  and  $0.23$  m, where  $z$  was the vertical distance measured downward from the soil surface. Soil samples used for experiments were made indoor, by mixing natural sand and commercial kaolin with the determined content ratios.

Preparation of the sand-clay mixtures was as follow: sand-clay mixtures were made by using a handheld mixer that allowed mixing without any significant grinding of the mixture. First, fine sand was poured into the mixing tank in a nearly dry condition. Then, clay was separately poured into a small tank and mixed with appropriate water until it reached the liquid state. Subsequently, the liquid clay was added into the mixing tank and running the mixer to form a uniformed soil sample. Finally, the prepared soil sample was placed in the trench of the flume.

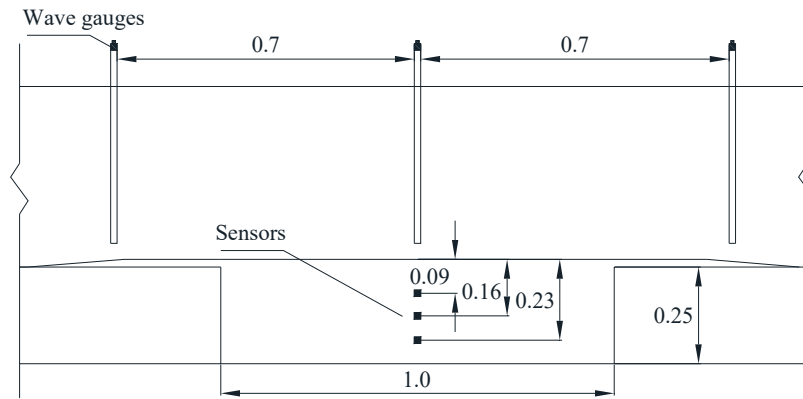


Figure 2. Soil trench (unit: m).

### 2.3. Soil Mechanics Test

The particle size distribution of materials was obtained by the Mastersizer 3000 laser particle size analyzer (Figure 3). The  $d_{50}$  of two kinds of soils are 0.163 mm (fine sand) and 0.009 mm (clay). The physical properties of seabeds are summarized in Table 1, CC was defined as the ratio of the weight of clay to the weight of sand-clay mixture, where  $\rho_s$  is the density of soil,  $k$  is the permeability coefficient,  $\omega$  is the rate of water content,  $e$  is the void ratio of the soil,  $\rho_d$  is dry density of soil,  $\rho_{dmax}$  is maximum dry density and  $\rho_{dmin}$  is minimum dry density,  $D_r$  is the relative density and is given by Equation (1).  $k_0$  is the coefficient of lateral earth pressure at rest and is given by  $k_0 = 1 - \sin\varphi$ ,  $\varphi$  is the friction angle. In this study,  $\varphi$  of all soil samples are 31 and  $k_0$  is 0.48. It should be mentioned that  $k$  is obtained by variable head experiments because the seepage pipe would be blocked by CC in constant head tests.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{\rho_{dmax}(\rho_d - \rho_{dmin})}{\rho_d(\rho_{dmax} - \rho_{dmin})} \quad (1)$$

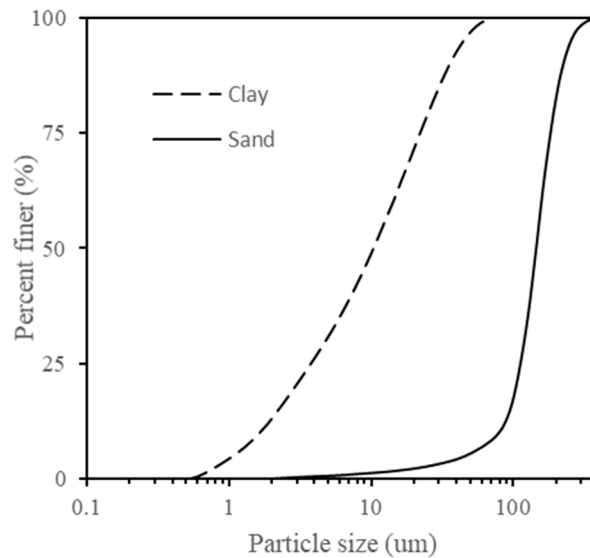


Figure 3. Particle size distributions of soil samples.

**Table 1.** Properties of soil samples.

| Soil Type | CC (%) | $\rho_s$ (g/cm <sup>3</sup> ) | k (cm/s) | $\omega$ (%) | e     | $\rho_d$ (g/cm <sup>3</sup> ) | $\rho_{dmax}$ (g/cm <sup>3</sup> ) | $\rho_{dmin}$ (g/cm <sup>3</sup> ) | $D_r$ |
|-----------|--------|-------------------------------|----------|--------------|-------|-------------------------------|------------------------------------|------------------------------------|-------|
| Sand      | 0      | 1.785                         | 0.000615 | 25           | 0.856 | 1.387                         | 1.760                              | 1.311                              | 0.215 |
| CC0.5     | 0.5    | 1.775                         | 0.000613 | 29.5         | 0.933 | 1.372                         | 1.756                              | 1.307                              | 0.218 |
| CC1.0     | 1      | 1.771                         | 0.000604 | 29.1         | 0.932 | 1.365                         | 1.752                              | 1.257                              | 0.281 |
| CC2.4     | 2.4    | 1.714                         | 0.000541 | 29.5         | 1.002 | 1.352                         | 1.745                              | 1.244                              | 0.279 |
| CC4.9     | 4.9    | 1.698                         | 0.000417 | 33.3         | 1.080 | 1.326                         | 1.740                              | 1.134                              | 0.407 |
| CC7.7     | 7.7    | 1.673                         | 0.000363 | 31.6         | 1.085 | 1.288                         | 1.734                              | 1.095                              | 0.416 |
| CC9.9     | 9.9    | 1.676                         | 0.000348 | 34.0         | 1.119 | 1.271                         | 1.727                              | 1.015                              | 0.488 |
| CC14.2    | 14.2   | 1.601                         | 0.000080 | 30.1         | 1.153 | 1.192                         | 1.704                              | 0.976                              | 0.425 |

Note: “CC0.5” represents a mixture of 0.5% clay content and 99.5% fine sand, and so on.

Compression modulus,  $E_s$ , was also measured by compression test. The compression coefficient,  $a_v$ , is obtained by the height change,  $\Delta h_i$ , of the confined soil sample after loading different loads,  $p$ .  $a_v$  is calculated by Equation (2),  $e_i$  is void ratio of soil under loading and is given by Equation (3).  $E_s$  is given by  $E_s = (1 + e)/a_v$  and results show in Table 2.

$$a_v = \frac{e_i - e_{i+1}}{p_{i+1} - p_i} \quad (2)$$

$$e_i = e - (1 + e) \frac{\Delta h_i}{h_0} \quad (3)$$

**Table 2.** Compression modulus of soil samples.

| Soil Type | $\rho_s$ (g/cm <sup>3</sup> ) | $\omega$ (%) | e     | $a_v$ (MPa) | $E_s$ (MPa) |
|-----------|-------------------------------|--------------|-------|-------------|-------------|
| Sand      | 1.785                         | 25           | 0.856 | 0.16        | 11.70       |
| CC0.5     | 1.775                         | 29.5         | 0.933 | 0.18        | 10.53       |
| CC1.0     | 1.771                         | 29.1         | 0.932 | 0.19        | 10.47       |
| CC2.4     | 1.714                         | 29.5         | 1.002 | 0.23        | 8.47        |
| CC4.9     | 1.698                         | 33.3         | 1.080 | 0.25        | 7.97        |
| CC7.7     | 1.673                         | 31.6         | 1.085 | 0.27        | 7.60        |
| CC9.9     | 1.676                         | 34.0         | 1.119 | 0.34        | 6.06        |
| CC14.2    | 1.601                         | 30.1         | 1.153 | 0.39        | 5.63        |

#### 2.4. Test Condition

The water depth above the soil surface was kept as 0.45 m for all tests. The percentage of clay content in the experiments are 0, 0.5, 1, 2.5, 5, 7.7, 9.9 and 14.2. The wave height in these tests are all 0.14 m and the wave period are all 1.6 s.

#### 2.5. Procedure

The procedures in these experiments were described below:

- (1) Place the well-mixed soil in the trench;
- (2) Fill up the flume with water gradually to a depth of 0.45 m;
- (3) Wait 20–22 hours to allow the soil to consolidate;
- (4) Calibrate water gauges and preheating pressure transmitters to record the pore pressure;
- (5) Switch on the waves 40 minutes;
- (6) Switch off the waves, and wait until the suspended sediment settles;
- (7) Empty the flume slowly;
- (8) Empty the sediment pit, and re mix soil sample if the next experiment is for the clay-sand soil with the same clay content. Prepare for the next test.

### 3. Results and Discussions

#### 3.1. Pore Pressure Response of Seabed

The relationship between clay and seabed liquefaction had been investigated by Gratchev et al. [7], Liu and Jeng [21,22], Kirca et al. [20] and Chavez et al. [23] through cyclic load simulation experiments and wave flume experiments. It was found that susceptibility of seabed liquefaction was increased with the addition of CC and there is a critical value of CC which liquefaction would not occur when the CC exceeded the critical value. However, it was neglected what would happen with low CC. To get further insight on mechanism of clayey seabed liquefaction, the effects of low CC on seabed liquefaction was investigated in this study.

Table 3 gives an overview of experiments. The parameters of excess pore water pressure (EPP) are listed in Table 3.  $D$  is the distance from seabed surface to the EPP sensor.  $P_{ACC}$  is the maximum of accumulated EPP.  $P_{AMP}$  is the amplitude of EPP.  $N_W$  is the number of waves required for EPP reaching  $P_{ACC}$ .  $\sigma_0$  is the critical mean normal effective stress that used to discriminate seabed liquefaction. It is given by Equation (4) in which  $\gamma'$  is the submerged specific weight of the soil and  $k_0$  is the coefficient of lateral earth pressure.  $\sigma'_{v,0}$  is the initial vertical effective stress that used to discriminate seabed liquefaction in engineering practice [27]. It is given by Equation (5).  $T_d$  is the time of pressure dissipation, the time point when EPP starts to drop under wave action.  $T_c$  is the time of soil consolidation, the time point when EPP drop to the lowest point, which represents the completion of the consolidation.  $T_d$  is the time of pressure dissipation.  $T_c$  is the time of soil consolidation.

**Table 3.** Seabed responds under wave actions.

| CC/% | D/m  | $P_{ACC}$ /kPa | $P_{AMP}$ /kPa | $N_W$ | $\sigma_0$ /kPa | $\sigma'_{v,0}$ /kPa | $T_d$ /s | $T_c$ /s |
|------|------|----------------|----------------|-------|-----------------|----------------------|----------|----------|
| 0    | 0.09 | 0.0284         | 0.3288         | 10    | 0.4528          | 0.3734               | $N_D$    | $N_D$    |
|      | 0.16 | 0.0275         | 0.1520         | 10    | 0.8050          | 0.6639               |          |          |
|      | 0.23 | 0.0671         | 0.0547         | 10    | 1.1571          | 0.9543               |          |          |
| 0.5  | 0.09 | 0.6133         | 0.7043         | 10    | 0.4470          | 0.3540               | 689      | 805      |
|      | 0.16 | 1.1557         | 0.7180         | 10    | 0.7947          | 0.6293               | 551      | 818      |
|      | 0.23 | 1.6245         | 0.7377         | 12    | 1.1424          | 0.9046               | 465      | 855      |
| 1.0  | 0.09 | 0.5303         | 0.5898         | 9     | 0.4447          | 0.3523               | 1901     | 2107     |
|      | 0.16 | 0.9589         | 0.6815         | 10    | 0.7906          | 0.6264               | 1450     | 2114     |
|      | 0.23 | 1.5704         | 0.9363         | 11    | 1.1365          | 0.9004               | 1090     | 2153     |
| 2.4  | 0.09 | 0.5676         | 0.7452         | 10    | 0.4119          | 0.3149               | $N_D$    | $N_D$    |
|      | 0.16 | 1.2606         | 1.2816         | 9     | 0.7322          | 0.5598               |          |          |
|      | 0.23 | 1.7048         | 0.6950         | 11    | 1.0525          | 0.8047               |          |          |
| 4.9  | 0.09 | 1.1134         | 1.2807         | 15    | 0.4026          | 0.2963               |          |          |
|      | 0.16 | 1.1605         | 0.9768         | 36    | 0.7158          | 0.5267               |          |          |
|      | 0.23 | 1.5187         | 0.5735         | 46    | 1.0289          | 0.7572               |          |          |
| 7.4  | 0.09 | 0.6207         | 0.5003         | 70    | 0.3882          | 0.2850               |          |          |
|      | 0.16 | 0.9852         | 0.5488         | 81    | 0.6901          | 0.5066               |          |          |
|      | 0.23 | 1.5443         | 0.2373         | 99    | 0.9921          | 0.7283               |          |          |
| 9.9  | 0.09 | 0.3481         | 0.1825         | 80    | 0.3899          | 0.2817               |          |          |
|      | 0.16 | 0.5829         | 0.1258         | 100   | 0.6932          | 0.5007               |          |          |
|      | 0.23 | 0.9277         | 0.2290         | 113   | 0.9965          | 0.7198               |          |          |
| 14.2 | 0.09 | 0.1941         | 0.0750         | 130   | 0.3467          | 0.2465               |          |          |
|      | 0.16 | 0.4392         | 0.1230         | 231   | 0.6163          | 0.4381               |          |          |
|      | 0.23 | 0.6221         | 0.2210         | 178   | 0.8859          | 0.6298               |          |          |

Note: " $N_D$ " represents that consolidation did not happen in these experiments.

From the Table 3, it can be concluded that the addition of CC caused liquefaction of seabed which would not liquefy without it. Even if only 0.5% clay was added to the seabed, a large accumulation of EPP occurred under the wave action. The accumulated EPP dissipated when the clay content is low, 0.5%–1%.

$$\sigma_0 = \gamma' D \frac{1 + 2k_0}{3} \quad (4)$$

$$\sigma'_{v,0} = \gamma' D(1 - n) = \gamma' D(1 + e) \quad (5)$$

Figure 4 shows EPP time series at  $z = 0.23$  m in sand-clay cases. With the introduction of waves, EPP begins to build up. Sumer et al. [6] described the mechanism of buildup of EPP with following words. The waves generate cyclic shear strains in the soil. If the soil is loose, the cyclic shear strains will gradually rearrange the soil grains at the expense of the pore volume of the soil. The latter effect will pressurize the water in the pores, and presumably this will lead to the buildup of EPP in the case of nearly undrained soil.

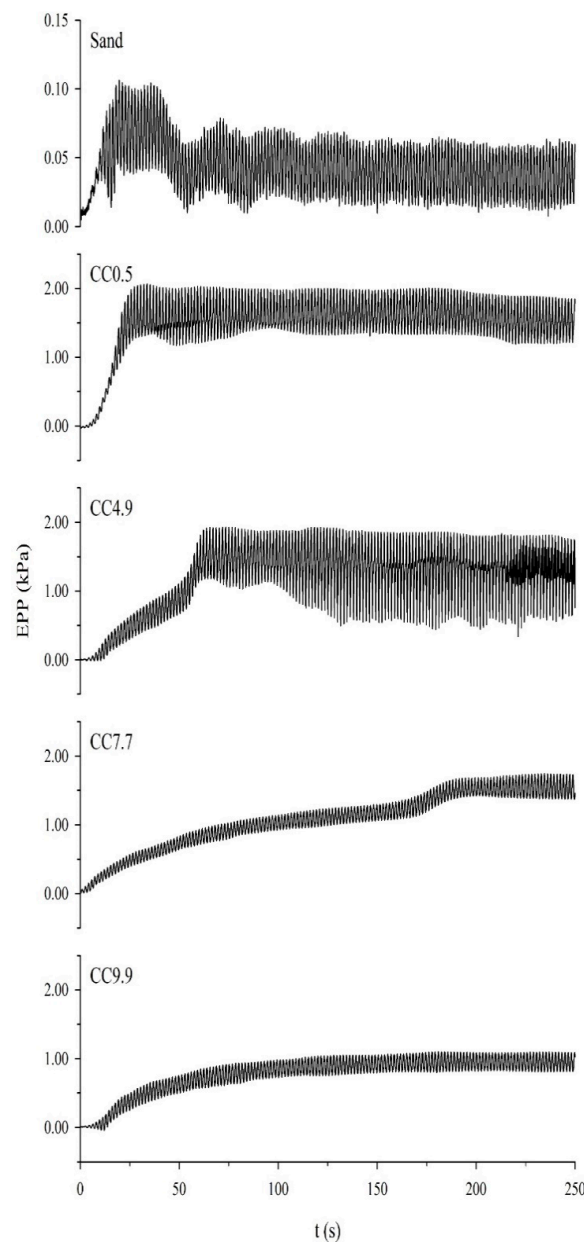


**Figure 4.** Excess pore water pressure (EPP) in mixed bed at  $z = 0.23$  m (the wave height is 0.14 m and the wave period is 1.6 s).

The measurements show that seabed is not liquefied in the sand-alone case, where  $P_{ACC}$  is much smaller than  $\sigma_0$ . On the other hand, in the well mixed soil bed containing certain amount of CC, it is found that the EPP accumulated obviously. As can be seen from Figure 4, the lower clay content, the faster EPP accumulated. When CC is small, 0.5–4.9, the amplitude of pore pressure is large and the change of EPP is irregular. When CC is large, 7.7–9.9, the amplitude of pore pressure is smaller and the EPP is regular. This is because the liquefaction of low CC cases is more intense, resulting in severe deformation of the seabed. The change of seabed elevation leads to the variation of pressure. In addition, the time required for EPP accumulation to reach its maximum increases with the increase of CC. This is due to the increase of CC changes the permeability and plasticity of soil. In summary, with the increase of CC, the accumulation of EPP slowed down. As a result, the value of final EPP accumulation become smaller and amplitude of EPP as well.

With the buildup of EPP, an upward-directed pressure gradient was generated. Because of the low permeability of the seabed, pore water cannot be squeezed from the seabed. When the seabed was liquefied and in a fluid state, the pores between particles gradually became larger, the drainage and wave-induced compaction occurred. Due to the limitation of experimental facilities, wave-making time is limited. Therefore, wave-induced compaction process occurred only in low CC cases, CC0.5 and CC1.0. The compaction phenomena for the cases with larger CC that need comparatively longer wave acting time was not observed. EPP oscillated sinusoidally because of the wave action. In order to obtain a clearer tendency of EPP, the curve of EPP was smoothed by averaging the value over a wave period. Figure 5 shows period-averaged EPP in low CC cases at depths  $z = 0.09, 0.16$  and  $0.23$  m. Period-averaged EPP is defined by Equation (6). It is seen that the observed EPP response in Figure 6 is fully consistent with the conceptual description of the liquefaction and compaction process in sand-bed by Sumer et al. [28] and Miyamoto et al. [29]. The process of liquefaction and compaction is classified into four stages, Pressure builds up, Liquefaction, Dissipation starts and Consolidation completed in Figure 6. Dissipation is shown clearly in Figure 5. It can be concluded that, with the decrease of CC, duration of Liquefaction decreases, time point of dissipation starts advances, and duration of Dissipation decreases.

$$\bar{P} = \frac{1}{T} \int_t^{t+T} P dt \quad (6)$$

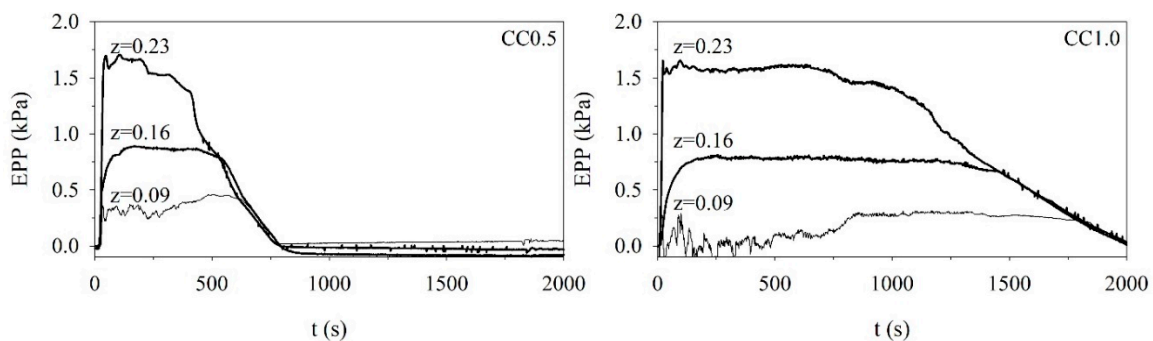


**Figure 5.** Period-averaged EPP in low clay contents (CC) bed (the wave height is 0.14 m and the wave period is 1.6 s).

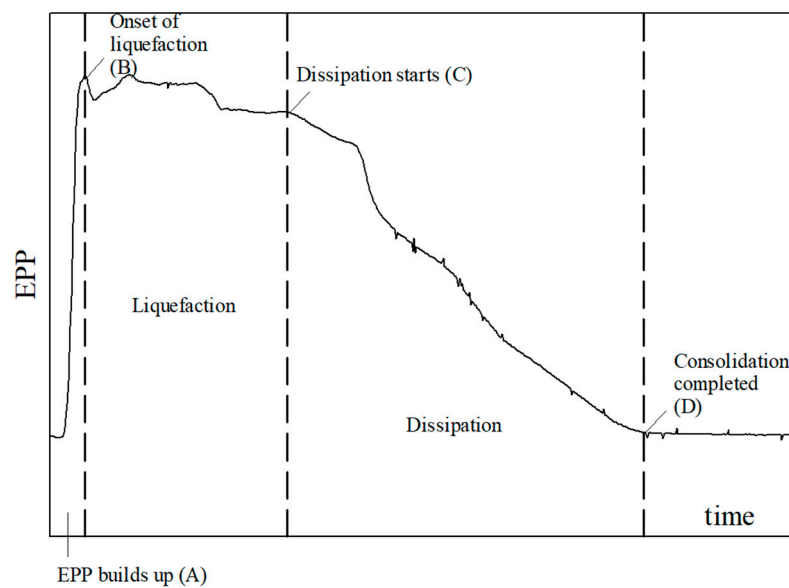


**Figure 6.** Schematic variation of period-averaged pore-water pressure at depth  $z$ .

Figure 7 shows screenshots of experiment video of CC0.5. Screenshots are taken every 20 s. Here are representative pictures. It can be seen that a thin suspended sand layer is above the bed surface and there is no obvious deformation of bed at 20 s, while EPP reached maximum at  $z = 0.09$  m around 25 s. The seabed began to fluctuate and suspended sand layer became thick at 40 s. The trough of the seabed corresponds to the wave crest and vice versa. EPP reached maximum at  $z = 0.23$  m around 50 s as shown in Figure 5. Sediment suspension on the bed surface was divided into two layers: thick layer (red line) and thin layer (yellow line) at 60 s. The shape of sediment suspension changed from mushroom cloud to strip from 80 s to 800 s. EPP dissipated around 405 s at  $z = 0.23$  m, 545 s at  $z = 0.16$  m and 635 s at  $z = 0.09$  m as shown in Figure 5. EPP Dissipation finished at around 800 s. It is difficult to see the change of seabed fluctuation on the screenshot. From the video, it can be seen that the fluctuation amplitude of seabed decreases gradually at 440 s and stops at 580 s. Sand ripples begin to form in 700 s. Sediment suspension dissipated when EPP dissipation completed.

Figure 8 shows the number of waves,  $N_w$ , for period-averaged EPP to reach  $P_{ACC}$  plotted against the CC at different depth. It can be seen that  $N_w$  increases with the CC when CC is more than 2.5 regardless of the depth, and  $N_w$  increases with the depth regardless of the CC except CC14.2. It is due to the decrease of permeability of mixed soil bed with the increase of CC, and more waves are needed to transmit wave energy to deeper soil layer under the same wave condition. When CC is 14.2%, wave energy cannot be transmitted to  $z = 0.23$  m, thus,  $N_w$  at  $z = 0.16$  m is bigger than  $N_w$  at  $z = 0.23$  m.

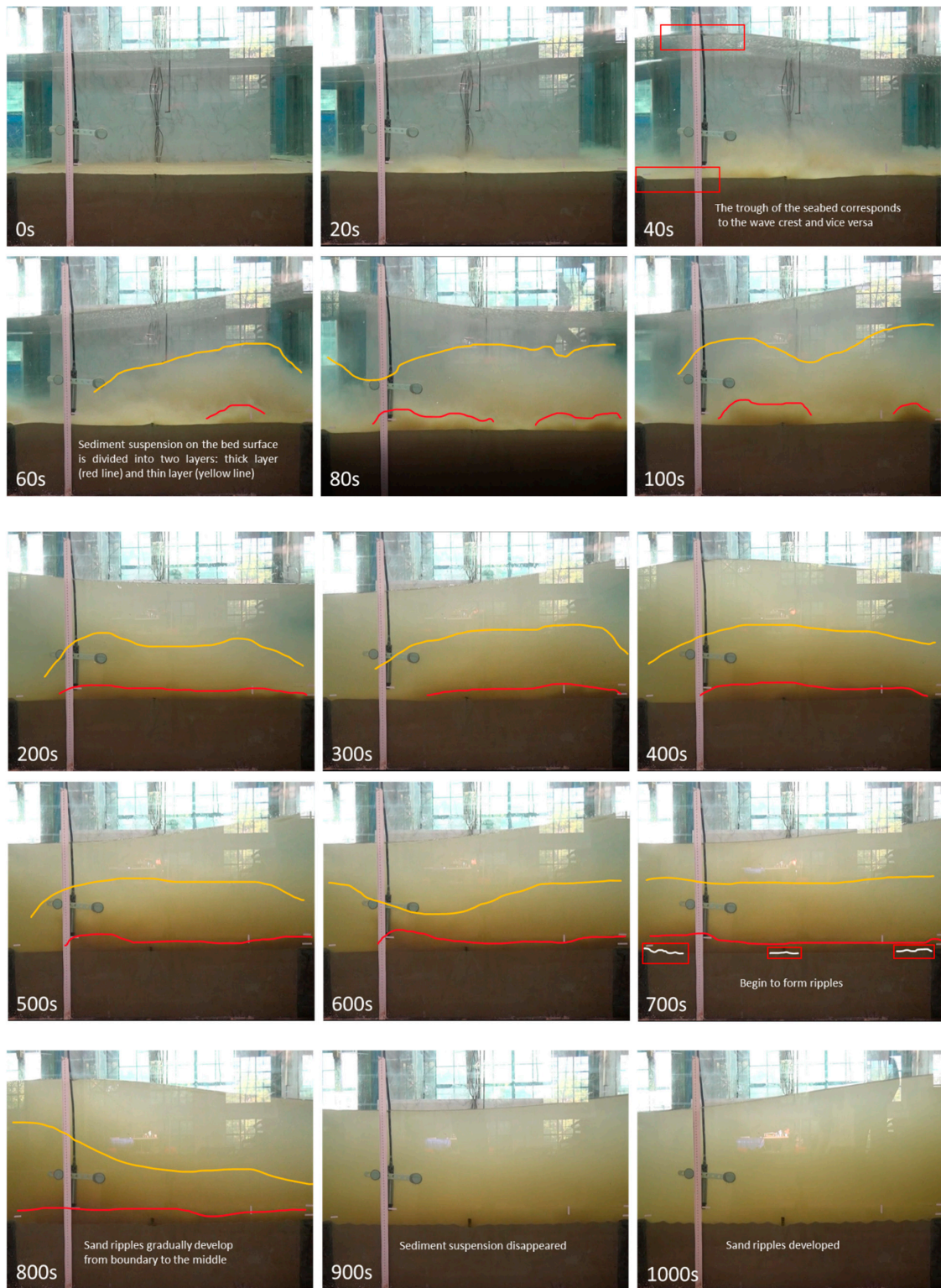


**Figure 7.** Liquefaction and consolidation of CC0.5 seabed.

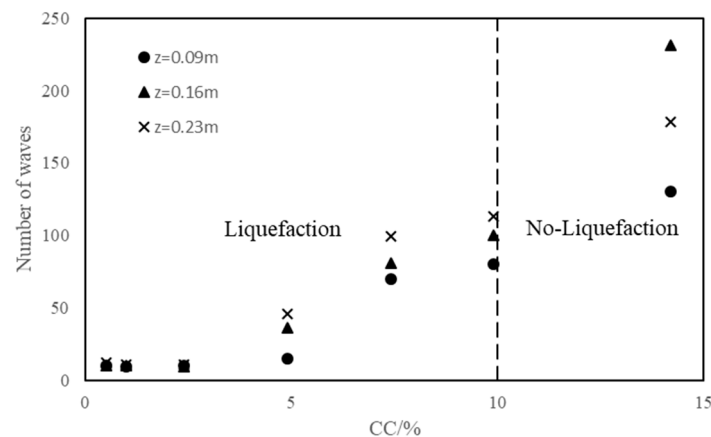


**Figure 8.** Number of waves (cycles) for period-averaged EPP to reach  $P_{ACC}$  at different depth.

### 3.2. Mechanism of Clayey Seabed Liquefaction

As discussed previously, sand-clay mixture with CC less than approximately 10% is susceptible to liquefaction, the liquefaction susceptibility decreasing with increasing CC. The reason for the influence of CC on liquefaction of clayey seabed is linked to cohesive strength of clayey soil, seabed permeability and seabed compressibility.

Up to now, it is hard to directly measure the cohesive strength between particles in mixed clayey bed. Kirca et al. [20] measured undrained shear strength of pure clay. The cohesive strength of pure clay is rather small, 1.75 kPa, and the value will be smaller for mixture with low CC. Therefore, cohesion is not expected to play a major role in wave-induced liquefaction of sand-clay mixture, unless the CC is much higher than the critical value for the sand grains to be completely encapsulated by the clay matrices.

Seabed permeability is an important factor affecting seabed liquefaction [30]. It was studied in the past in conjunction with laboratory simulations of seismic-induced liquefaction by Prakash [31]. His results show that the smaller the permeability, the more susceptible the soil is to liquefaction. Seabed cannot liquefy with large permeability because all EPP in the soil would dissipate as rapidly as they develop. Gratchev et al. [7] studied permeability through observing the microfabric of artificial mixtures by using SEM. Mixtures were divided into four microfabric according to CC, non-plastic, low plasticity clayey sand, medium plasticity clayey sand and high plasticity clayey sand. It pointed out that CC reduced the number of contact points between the sand particles and clay bridge in low plasticity clayey sand provided a structure that was easier to rearrange under cyclic shear strains. Pore of high plasticity clay sand were filled with clay, so the permeability was very low and pore water cannot flow. Kirca et al. [20] also explained the liquefaction of silt-clay mixture with the theory of Gratchev et al. [7]. Wiebicke et al. [32] studied granular fabric under shearing by x-ray computed tomography. However, aforementioned theory cannot explain the liquefaction of very low CC cases. The coefficient of permeability for sand and low CC mixture are almost same, as shown in Table 1.

Figure 9 shows microfabric of seabed obtained by SEM (Hitachi SU8100). It can be seen that clay aggregations generally gathered at the sand particle contact points when CC is low (CC0.5, CC1.0, CC2.5 and CC4.9), and clay produced a matrix that made microfabric of seabed more compact when CC is high (CC7.7, CC9.9 and CC 14.2). With the increase of CC, clay aggregations and matrix increase, and porosity between particles decreases. Gratchev et al. [7] suggested that the aggregation of clay at contact points of sand produced significant pore spaces and the contact of sand grains were easily destroyed during cyclic loading. The destruction of grains connection was the trigger for seismic-induced liquefaction. Different from seismic load, wave load comes from the fluctuation of sea water above the seabed. Figure 10 shows the mechanism of liquefaction. Seabed is compressed and seepage force is downward when wave is at the crest. The pore water pressure increases greatly due to the compression of soil skeleton and seepage effect. Seabed would rebound and seepage force

is upward when wave is at the trough. The pore water pressure dissipates through drainage and rebound of soil skeleton. Due to the decrease of permeability and plastic deformation of the seabed after compression of the soil bed, neither drainage nor rebound of soil skeleton can be balanced with the compression and seepage effect of the seabed at wave crest, so the pore water pressure accumulates. As described by Li [33], excess pore water pressure is related to the change of soil volume. When the compression coefficient of soil skeleton is 0, any seepage action will not produce excess pore water pressure. So, the compressibility of seabed has a great influence on liquefaction. Compression modulus of seabed,  $E_s$ , was measured by compression test and shown in Table 2. It can be seen that with the increase of CC,  $E_s$  decreases and compressibility of seabed increases. Figure 11 shows the compression progress of clayey seabed. It is obviously that there will be clay aggregations at the sand grains contact points (from [7] and Figure 9) when CC is low. The contact friction force between particles that maintained the shape of soil skeleton becomes smaller due to the aggregations, so grains are easy to slip and rotate when seabed subjected to wave load. In a word, clay content improves the compressibility of clay-sand seabed and thus increases the possibility of seabed liquefaction.

In summary, the cohesive strength has little influence on the liquefaction of clayey seabed, while the permeability and compressibility play important roles in the liquefaction. As shown in the experimental results, CC does not only affect the permeability of seabed, but also change the structure of seabed, making it easier to compress. Under the influence of permeability and compressibility, the pore water pressure develops easily when the wave load acts on the seabed. In this experiment, the seabed with permeability of 0.000363–0.000613 cm/s and compression modulus of 10.53–7.60 MPa will liquefy under laboratory conditions.

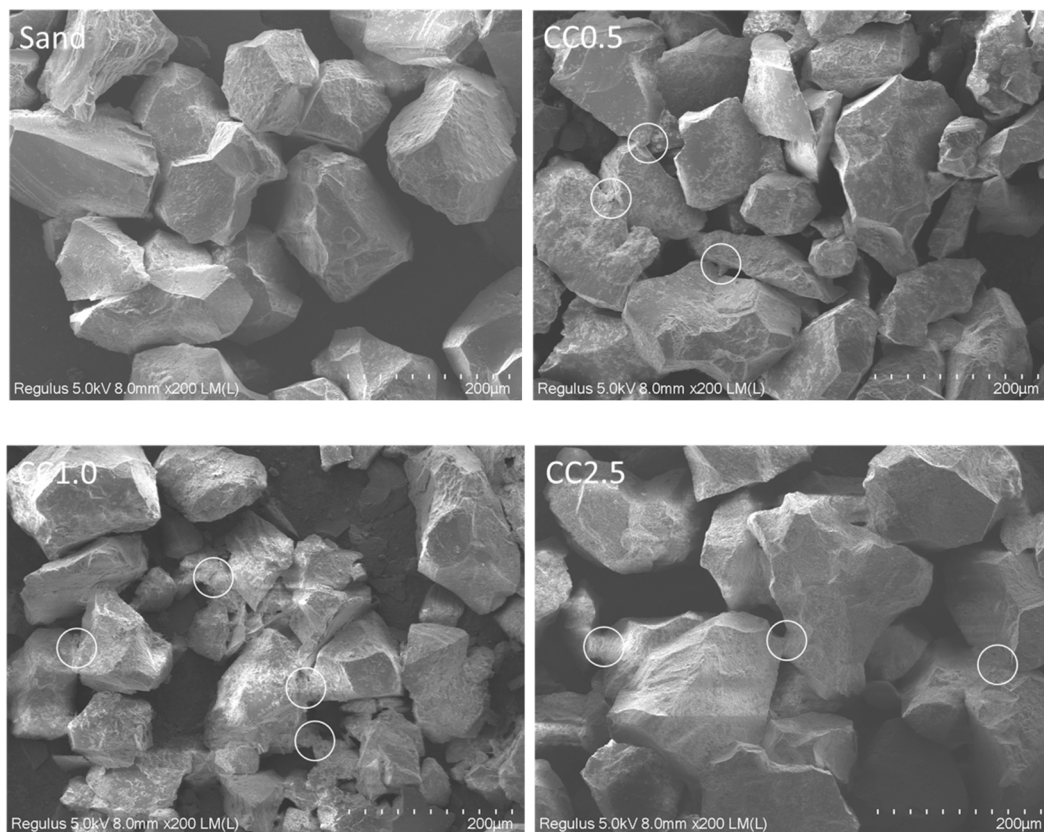


Figure 9. Cont.

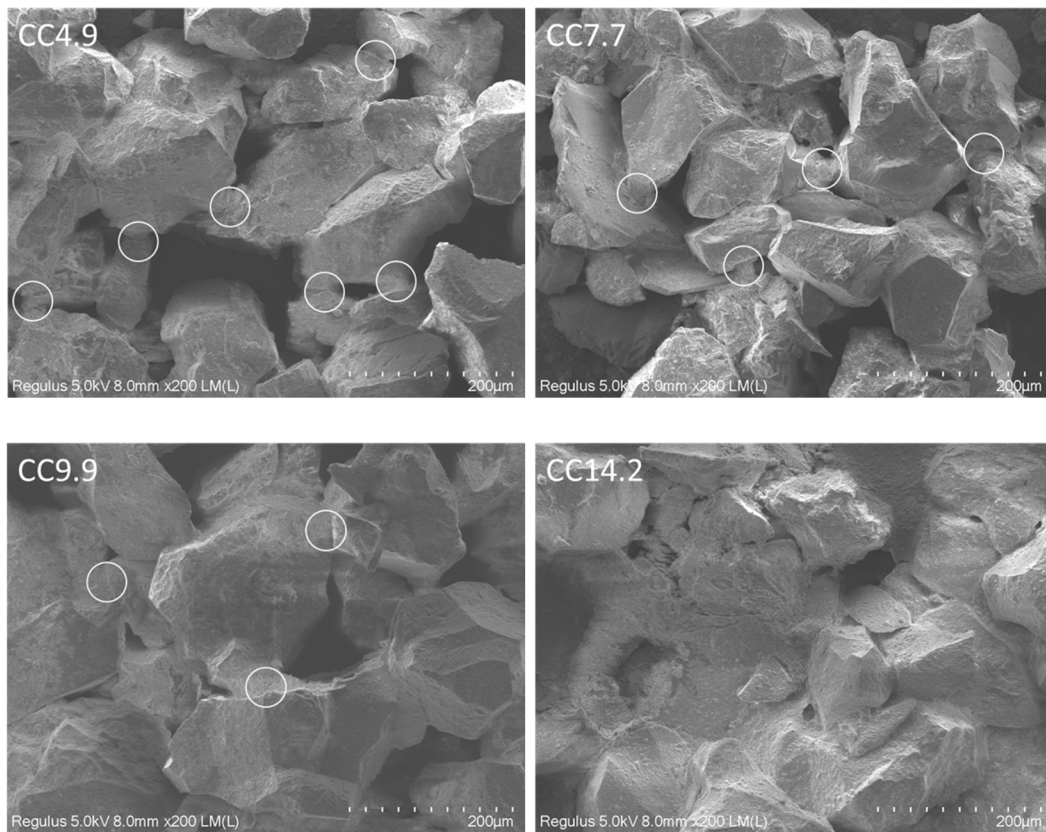


**Figure 9.** The microfabric of seabed with different CC (aggregation in white circle).

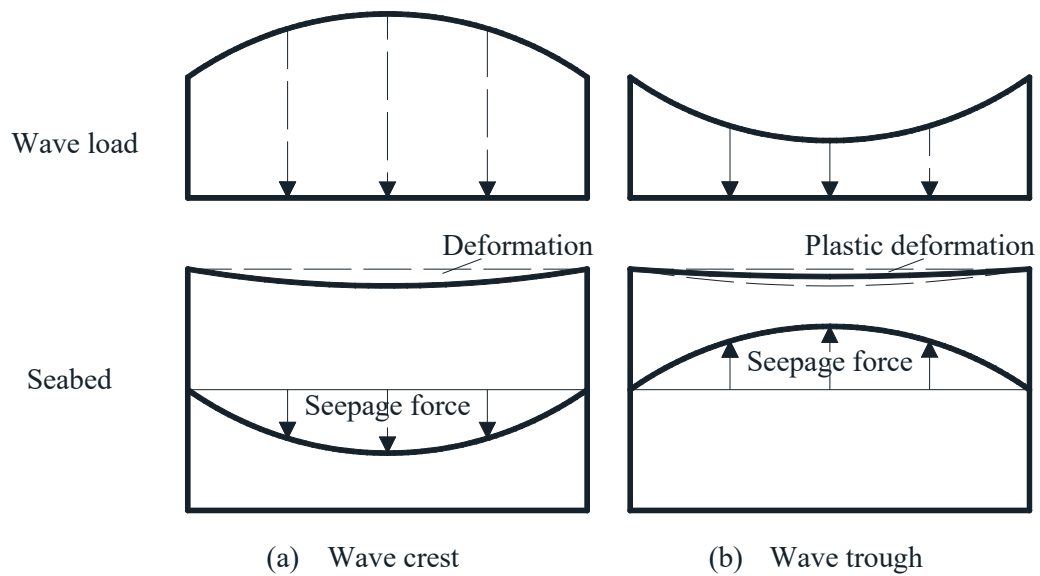


**Figure 10.** The Schematic diagram of liquefaction mechanism (a) is the seabed response at wave crest and (b) is the seabed response at wave trough.

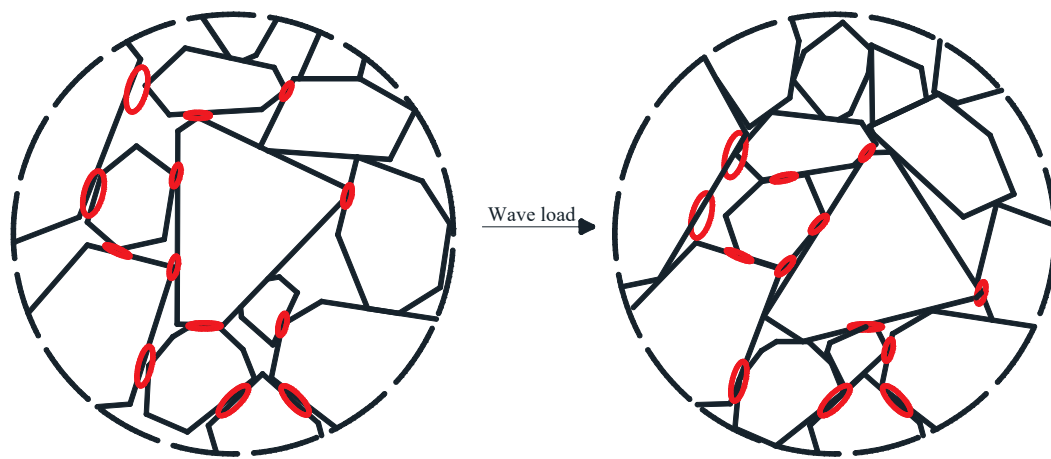


**Figure 11.** The Compression of clayey soil.

As shown above, CC plays an important role in wave-induced liquefaction. Sand may become liquefaction-prone by adding clay and sand-clay composite do not liquefy when CC is more than 10%. The reason for these phenomena is that CC affects both the permeability and compressibility of clayey seabed. Compared with previous studies [7,20–23], present experiments revealed that a small CC can also have a significant impact on seabed liquefaction, and the permeability and modulus of compressibility can provide a simultaneous analysis for engineering liquefaction evaluation. Lastly, the mechanism of clayey seabed liquefaction still needs to be further studied. The reason of pore water pressure accumulation under wave action is clear. But the reason why the duration of seabed liquefaction with different CC is different need to be explored, and the mechanism of pore pressure dissipation need to be investigated.

#### 4. Conclusions

In this work, liquefaction of the clayey seabed was explored by means of wave flume and soil mechanics tests. The primary conclusions are as follows:

- (1) With the introduction of CC, fine sand will have more susceptibility to liquefy until the clay content exceed a certain value so that the mixed bed will not be liquefied. In this study, the value is 10%. the lower limit critical value was not observed in these experiments that even the seabed with 0.5% CC could liquefy under laboratory wave conditions.
- (2) When CC is small, 0.5–4.9, the amplitude of pore pressure is large and the change of EPP is irregular. When CC is large, 7.7–9.9, the amplitude of pore pressure is smaller and the EPP is regular. Accumulation and amplitude of EPP both decreased with the increase of CC when the clay content is more than 5%.
- (3) Wave-induced compaction occurred only in CC0.5 and CC1.0. With the increase of CC, duration of liquefaction increases, time point of compaction-start delays, and duration of dissipation increases.
- (4) The relationship between CC and the wave-induced liquefaction of clayey seabed was established. It was found that CC significantly affects the microfabric of seabed. Clay aggregations and matrix not only influences the permeability but also the compressibility of the seabed. Under the influence of permeability and compressibility, the seabed is in different states with different condition, from non-liquefiable (pure sand) to liquefiable (CC0.5–7.7), and to non-liquefiable (CC9.9–14.2) again.
- (5) On the basis of the obtained results, an attempt to explain the mechanism of wave-induced liquefaction of clayey seabed was made.

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