MECHANISM OF WAVE-INDUCED LIQUEFACTION AND DENSIFICATION IN SEABED

KOUKI ZEN and HIROYUKI YAMAZAKI

ABSTRACT

The mechanism of the wave-induced liquefaction and densification in permeable seabed is examined with particular reference to the excess pore pressure. A concept of the ‘oscillatory’ excess pore pressure has been introduced and verified by model experiments using a newly developed apparatus. The following main conclusions are drawn from the study; 1) the oscillatory excess pore pressure is excited in the seabed where the wave-associated bottom pressure is propagated into the seabed with some damping and phase lag, 2) the magnitude of the change in the effective stress is identical with the oscillatory excess pore pressure, 3) the liquefaction and the densification occur alternately in the seabed, even in one period of the wave loading, and 4) the wave-induced liquefaction is quite different from the earthquake-induced liquefaction in the mechanism of the generation of excess pore pressures.

Key words: coast, liquefaction, model test, ocean soil, pore pressure, sand, water pressure, wave Propagation (IGC: E 7/E 8)

INTRODUCTION

The wave-induced instability of permeable seabed is one of the important subjects in considering the design of offshore structures such as breakwaters, platforms, pipelines and anchors. Recently, Yamamoto (1977), Madsen (1978), Finn et al. (1983) and Okusa (1985) have shown analytically that the ocean wave propagation induces oscillatory shear stresses in the seabed. If these stresses exceed the shear strength of the seabed, significant deformations and a failure may occur. Inoue (1975), Nago (1981) and Tsui and Helfrich (1983) have observed experimentally that the oscillatory surface pressures on deposits are propagated into the deposits with some attenuation. The field measurements done by Okusa et al. (1984) and Maeno and Hasegawa (1985) have revealed almost the same findings as those in the laboratory tests. The attenuation of pore pressures is considered a significant reason for the shear stress oscillation in the seabed.

Among these works, some have suggested that the liquefaction might be caused by the wave-associated stresses in the seabed. The wave-induced liquefaction is, however, not

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clearly addressed in terms of the 'excess' pore pressure which is one of the comprehensive parameters in evaluating the liquefaction potential. Therefore, it is essential to make clear the mechanism of the generation of excess pore pressures for a proper understanding of the wave-induced liquefaction.

The densification may be caused by the wave-associated effective stresses, if they are repeated in the porous seabed under drainage conditions. Bjerrum (1973) has suggested that the high relative density of the sea floor in the North Sea could be attributed to a large number of cyclic shear stresses generated by the differential wave loading of the seabed. It is, however, not clarified why the water pressures work as external forces on the soil skeleton, effective to densify the porous seabed. The mechanism of the wave-induced densification is another interest in this paper.

CONCEPT OF LIQUEFACTION AND DENSIFICATION

Ocean Waves and Seabed

When ocean waves propagate over a seabed, they impose a periodical water pressure on the surface of the seabed. Although the real nature of ocean waves is very complex, the wave-associated pressure on the surface of the seabed may be evaluated with the linear wave theory. The approximation with the linear wave theory can be considered reasonable, when the fundamental properties of a group of irregular waves are expressed by a superposition of the sinusoidal waves. This assumption has been used in most of the previous works (Henkel, 1970; Bjerrum, 1973 among others). According to the linear wave theory, the bottom pressure associated with waves, \( p(x, t) \), is expressed as follows:

\[
p(x, t) = p_0 \sin \left( \frac{2\pi}{T} x - \frac{2\pi}{T} t \right)
\]  

where the amplitude of the bottom pressure, \( p_0 \), is,

\[
p_0 = \frac{\gamma w H}{2} \frac{1}{\cosh(2\pi h/L)}
\]

\( \gamma w \); the unit weight of water, \( h \); the water depth, \( H \); the wave height, \( T \); the wave period, \( L \); the wave length, \( t \); time and \( x \); the coordinate axis in the direction of the progress of waves.

In this study, the oscillation of the bottom pressure and the motion of the seabed are assumed one dimensional. Also, the cyclic shear stresses on the surface of the sea floor induced by the bottom current are omitted. These assumptions may be acceptable if the wave length is large enough, compared with the thickness of the seabed, to ignore the horizontal water flow on and in the seabed as well as the horizontal deformation in the seabed. Then, Eq. (1) can be written as:

\[
p(t) = p_0 \sin \left( \frac{2\pi}{T} t \right)
\]  

Oscillatory Excess Pore Pressure and Effective Stress

The initial vertical total stress \( \sigma_v(x, 0) \), at arbitrary depth, \( x \), of the deposit is expressed by:

\[
\sigma_v(x, 0) = \sigma'_{v}(x, 0) + \mu(x, 0)
\]

where, \( \sigma'_{v}(x, 0) \) and \( \mu(x, 0) \) are the vertical effective stress and the hydrostatic pressure at the still water level, respectively. When the water level is subjected to a change from the initial still water level, it induces the total stress variation in the deposit. Provided that the pore pressure change from the initial state of the hydrostatic pressure, \( p(z, t) \), is not equal to the pressure change imposed on the surface of the deposit, \( p(0, t) \), the vertical total stress, \( \sigma_v(z, t) \), and the pore pressure, \( \mu(z, t) \), in the deposit are given by the following equations, respectively:

\[
\sigma_v(z, t) = \sigma_v(z, 0) + p(0, t)
\]

\[
\mu(z, t) = \mu(z, 0) + p(0, t)
\]

Both pressures are positive in the direction to which they increase from the initial hydrostatic pressure. Therefore, the vertical effective stress in the deposit, \( \sigma'_{v}(z, t) \), can be obtained by subtracting Eq. (5) from Eq. (4) and substituting Eq. (3):

\[
\sigma'_{v}(z, t) = \sigma'_{v}(z, 0) + \{p(0, t) - p(z, t)\}
\]

The effective stress change, \( \Delta\sigma'_{v}(z, t) \), is
excess pore pressure, $u_e(z,t)$, is defined as:

$$u_e(z,t) = - \{ p(0,t) - p(z,t) \} \quad (8)$$

The effective stress change, $\Delta \sigma'_v(z,t)$, is induced by this excess pore pressure, $u_e(z,t)$.

**Criteria for Liquefaction and Densification**

In order to apply the concept mentioned above to the seabed, the schematic drawings of the pore pressure and effective vertical stress distributions are illustrated in Fig. 2. The solid curves in Fig. 2(a) indicate the pore pressures beneath a wave trough and a wave crest. The excess pore pressure expressed by Eq. (8) is transient in nature, because the $p(0,t)$ and $p(z,t)$ are oscillatory and periodical in real ocean environment. Consequently, the effective vertical stress, $\sigma'_v(z,t)$, expressed by Eq. (6) varies periodically in accordance with the change of $(p(0,t) - p(z,t))$ in the seabed. If it attains zero or less at certain depths, the soil skeleton will become a liquefied state there. Thus, the criterion for the wave-induced liquefaction is easily derived from Eq. (6) by setting the vertical effective stress, $\sigma'_v(z,t)$, equal to zero or less:

$$\sigma'_v(z,0) \leq - \{ p(0,t) - p(z,t) \} = u_e(z,t) \quad (9)$$

The solid curves in Fig. 2(b) show the vertical effective stress distribution drawn by replacing the $\sigma'_v(z,0)$ in Eq. (6) with $\gamma'z$. Where, $\gamma'$ is the submerged unit weight of deposits.

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**Fig. 1. Pressure ratio and wave period (after; a) Tsui and Helfrich (1983), b) Yamamoto (1977), c) Inoue (1975), d) Maeno and Hasegawa (1985), e) Okusa et al. (1984)) (1 kgf/cm$^2$ = 98 kPa)**

Fig. 1 shows the relationship between the pore pressure ratio, $p(z,t)/p(0,t)$, and the wave period, $T$, at the non-dimensional depth of $z/l = 0.45$. Where, the $l$ is the thickness of permeable deposits. The $p(z,t)/p(0,t)$ is expressed by the $p_m/p_0$ in Fig. 1. Solid circles in Fig. 1 are data obtained from the experiment in this study. As can be seen in Fig. 1, the $p_m/p_0$ is strongly dependent of the wave period, $T$, and it becomes less than 1.0 against the wave period smaller than 15s. The aforementioned ‘quick’ variation of the water level is considered to be brought by the ocean waves with the wave period less than about 15s.

Since the pore pressure change, $p(z,t)$, is to be equal to the water pressure change on the surface of the deposit, $p(0,t)$, under the steady state condition, the difference can be thought to represent an ‘excess’ component of the pore pressure in the deposit. So, the
The lines numbered with ①, ② in Fig. 2(b) correspond to ones numbered ①, ② in Fig. 2(a), respectively. In Fig. 2(b), the liquefied zone shown by slant lines where the vertical effective stress becomes zero or less appears near the surface of the seabed, under the wave trough.

Whereas, if the vertical effective stress, \( \sigma'_e(z, t) \), attains greater values than the \( \sigma_v(z, 0) \) as shown by the line numbered ② in Fig. 2(b), the wave-induced vertical stress exerts downward forces on the soil skeleton to possibly densify the seabed. The criterion for the densification may be expressed by:

\[
\Delta \sigma'_e(z, t) = p(0, t) - p(z, t) > 0 \quad (10)
\]

The spatial difference between the \( p(0, t) \) and the \( p(z, t) \) induces downward or upward flows of water in the seabed. As the oscillatory excess pore pressure, \( u_e(z, t) \), becomes negative when the \( \{p(0, t) - p(z, t)\} \) satisfies Eq. (10), the stress increment which causes the densification of the seabed can be attributed to the downward seepage force exerted on the soil skeleton.

As mentioned above, the wave-induced liquefaction is governed by the following factors: the initial vertical effective stress at calm, \( \sigma'_v(z, 0) \) and the oscillatory excess pore pressure, \( \{p(0, t) - p(z, t)\} \). When these factors are obtained, the liquefaction potential will be easily evaluated. So, for convenience, the quantities \( \sigma'_v(z, 0) \), \( p(0, t) \) and \( p(z, t) \) are expressed as, \( \sigma'_v(z, 0) = \sigma'_v0 \), \( p(0, t) = p_b \) and \( p(z, t) = p_m \), hereafter.

**EXPERIMENTS**

**Test Apparatus**

A newly developed test apparatus is schematically shown in Fig. 3. The cylinder for constituting the model seabed is assembled by piling up the sixteen transparent acrylic cells. The height of a cell is 100 mm or 200 mm, the outer diameter is 225 mm and the inner diameter is 200 mm. The fabricated cylinder is bolted with the steel bars to the rigid top and base plates to assure the rigidity of the cylinder. In the process of fabrication, the O-rings are attached to the side of the cells and banded with the collars to prevent the water leakage.

The air pressure control unit, called a pneumatic sine loading unit in this paper, generates the static and cyclic air pressures and supplies them to a balloon tank. The air pressures are converted to the water pressures through the interface between the water and a rubber tube in the balloon tank.

The pore water pressure transducers (Maximum capacity 2 kgf/cm²(196 kPa)) are installed on the outlets of each cell as indicated by the symbols P0, P1, P3, P5, P7, P9, P11, P13 and PL in Fig. 3. The earth pressure gauge (Maximum capacity 2 kgf/cm²(196 kPa)) is also mounted on the base plate to measure the vertical stress at the bottom of the deposit. The analog signals from these transducers and gauge are converted to digital ones and electrically recorded onto the cassette data recorder. The markers indicated by the symbols M1 to M14 in Fig. 3 are buried every 0.1 m or 0.2 m interval in the sand deposit to measure the settlements and vertical strains at each depth. Crushed gravels of 5 mm in diameter are used for the markers.

**Preparation of Deposit**

The grain size accumulation curve and the
physical properties of Toyoura sand used for the tests are shown in Fig. 4 and Table 1, respectively. The air-dried Toyoura sand necessary to constitute the 50% of the initial relative density in each cell was weighted in advance and stored in the container. The sample was poured into the cell with a spoon dividing every three layers for one cell and a light tamping was added to the sample. The total thickness of the deposit constituted in the cylinder is 0.28 m or 1.90 m.

The carbon dioxide gas was circulated in the deposit in order to increase the degree of saturation. After the completion of the circulation, the hydro-static pressure of 1.0 kgf/cm² (98 kPa) was applied on the surface of the deposit on the hypothesis of the water depth of 10 m in real ocean. It was left for more than 10 hours as it was.

**Test Procedure**

Before starting the wave loading, the positions of markers were measured from outside of the transparent cylinder with the vernier calipers. Then, the new origins of the markers were dotted again on the sidewall of the transparent cylinder.

The wave loading conditions were chosen, as shown in Fig. 5, from the relationships between the significant wave period, $T_{1/3}$, and the significant wave height, $H_{1/3}$, observed in the fields by Hirose and Takahashi (1982). In Fig. 5, the $H_o/L_o$ denotes the wave steepness and the $2\theta_o$ indicates the double amplitude of the applied pressure on the surface of the deposit. The sinusoidal wave forms of the cyclic pressures were applied to the surface of the deposit. The so-called stage test, in which the loading conditions were changed after every 500 cycles of loading in each stage, was adopted. Test cases and the applied loading conditions are summarized in Table 2. Test No. 4 was performed under the condition that an overburden pressure of 0.0716 kgf/cm² (7 kPa) was added to the deposit surface with small steel balls whose diameter was 2.5 mm.

During the tests, the cyclic pressures and the vertical stress changes were measured at the appropriate number of waves. The displacements of the markers in the deposit were measured at the end of each loading stage. As four markers were placed in a level
plane, the values of the four are averaged.

After all loading stages have been completed, the sample was cut in each cell to take it out of the cylinder, then the relative density and the degree of saturation were measured by the cell. The degree of saturation of the sample in each cell was between 97% and 101%.

**TEST RESULTS**

**Oscillatory Pore Pressure**

Fig. 6 is an example of the wave-associated bottom pressure and the oscillatory pore pressures recorded at the depths corresponding to the transducers PO to P13 in Fig. 3. Fig. 6 clearly shows that the oscillatory pore pressures at the time, 2.25 s, say at the 270° of the phase, attenuate from $p_{m1} = -0.243$ kgf/cm² (−23.8 kPa) to $p_{m13} = -0.190$ kgf/cm² (−18.62 kPa) with the depths. In addition, it is found that their peak values shift to the right indicating greater phase lags at the lower depths. The attenuation is more clearly illustrated in Fig. 7 in which the oscillatory pore pressure, $p_m$, is normalized by the amplitude of the bottom pressure, $p_b$. The phase lags of the pore pressures, $\theta$, observed in each depth are also drawn in Fig. 8. As shown in Figs. 7 and 8, the bottom pressure is propagated into the deposit with certain damping and phase lag.

The relationship between the pore pressure ratio, $p_m/p_b$, and the wave period, $T$, has been previously presented in Fig. 1 together with
other investigators' results. In spite of the measurements in the different kinds of sand deposits, the results show almost the same relationships. This means that the wave period is one of the important factors to induce the damping and phase lag of the oscillatory pore pressure in the seabed. Other influential factors on the oscillatory pore pressure are presented elsewhere (Zen and Yamazaki, 1990).

Effective Stress Oscillation

The vertical stresses at the bottom of the deposit are measured with the earth pressure gauge mounted on the base plate shown in Fig. 3. The oscillatory pore pressures are measured at the bottom of the deposit and at the several depths in the deposit. The thickness of the deposit is set 0.28 m in order to decrease the effect of the side friction between the sample and the cylinder. Fig. 9 shows the cyclic pressure on the surface of the deposit, $p_b$, and the corresponding variation of the vertical effective stress at the bottom of the deposit, $\Delta \sigma'_{v}$. The $\Delta \sigma'_{v}$ illustrated by the solid line is obtained by subtracting the oscillatory pore pressure from the measured total stress changes. It is confirmed from Fig. 9 that the vertical effective stresses vary periodically in accordance with the pressure changes on the surface of the deposit. The effective stress change reveals a considerably large phase lag against the surface pressure change. The meanings of the dotted line and the open circles will be discussed in the latter chapter.

Settlement, Residual Strain and Relative Density

Fig. 10 shows the typical examples of the accumulative settlements in the deposit measured after each stage of the wave loading. The distribution curves indicate considerably larger settlements near the surface of the deposit than those at greater depths. Especially,
in the final stage (in Stage No. 7), the accumulative settlements attain about 45 mm at the surface. In addition, the curves have inflection points at certain depths, indicating drastic increases of the settlements at the shallower depths beyond those points.

The vertical residual strains are calculated from the measured settlements of the adjacent markers in the deposit and illustrated in Fig. 11. The residual strains, except those in Test No. 4, attain more than 8% at the vicinity of the surface of the deposit while they reach 3% at most for the depths below 0.8 m. The inflection points are also observed in the distribution curves of the residual strains. The small residual strains in Test No. 4 are attributed to the overburden pressure applied to the surface of the deposit (Zen et al., 1987).

Fig. 12 shows the relative density measured by the cell after the wave loading is completed. The increases of the relative density from the initial value of about 50% are clearly indicated in Fig. 12. Especially, it is quite surprising that the final relative density reaches no less than 80%. In Test No. 1 it attains about 100% at the vicinity of the surface of the deposit. Such increases of the relative density are considered to be led by the cyclic water pressures imposed on the surface of the deposits, since no other external forces are applied to the deposit, since no other external forces are applied to the deposit. Comparing the distribution curves in Fig. 12 with those in Figs. 10 and 11, it is found that their shapes are quite similar. In addition, the noticeably large increases are observed near the surface of the deposit. These remarkable increases could not be explained even though the compressibility of the deposit against the maximum vertical effective stress ever experienced has been taken into account. It is supposed that some peculiar phenomenon progressing the densification has happened at the depth near the surface of the deposit. This will be discussed in detail lately.
DISCUSSIONS

Verification of Wave-induced Liquefaction

Fig. 13 indicates the distribution of the wave–associated vertical effective stresses in
the deposit evaluated by using Eq. (10) from the measured cyclic pressures on the surface
of the deposit, \( p_b \), and the oscillatory pore pressures, \( p_m \). According to Eq. (9),
the liquefaction is evaluated to take place at the phases where the distribution curves of the
\( (p_b - p_m) \) come in touch with the initial vertical effective stress, \( \sigma'_v \). The \( \sigma'_v \)
is calculated from the measured dry density of the deposit and indicated by the dotted line in Fig. 13.
It is clearly shown that the liquefaction criterion is satisfied at the phases between 150\(^\circ\)
and 270\(^\circ\). These phases correspond to the wave trough and its vicinity. In Fig. 13,
the liquefaction zone appears to be developed at rather shallower depth and to progress to
deeper depth as the waves propagate. The maximum liquefaction depth of 0.13 m is
observed at the phase, \( \omega t = 210^\circ \)in this test case. This is one of the typical characteristics of the wave–induced liquefaction, unlike the earthquake–induced liquefaction.

Another interesting point in Fig. 13 is that the three different types of the phenomena
such as the liquefaction, the expansion and the densification are continuously induced in
the deposit even in one period of the wave loading. When a great number of storm
waves pass through, these three types of the phenomena may be cyclically excited in the
seabed.

The aforementioned discussions are based on the assumption that Eq. (7) holds true.
This assumption can be verified referring to Fig. 9. In Fig. 9, the dotted curve indicates the vertical effective stress change calculated from the \( p_b \) and the \( p_m \) by using Eq. (7).
As can be seen in Fig. 9, the dotted curve compares well with the solid curve at the range of the phases where the \( \Delta \sigma'_v \) denotes positive, say at the range of the phases between 0\(^\circ\) and 90\(^\circ\) and between 300\(^\circ\) and 360\(^\circ\).
This means that the effective stress oscillation is identical with the oscillatory excess pore pressure, although the sign is opposite. On the other hand, at the range of the phases between 120\(^\circ\) and 240\(^\circ\), the solid curve does not show a good agreement but is rather larger than the dotted line. The reason may be considered as the followings;

When the liquefaction occurs in the deposit up to the depth, \( d_t \), the soil particles above
the depth become a suspended state. Then, as the surface of the deposit is considered to
descend to a new surface by \( d_t \), the change of the total stress at the new surface of the
deposit, \( p_t \) is given by:

\[
p_t = p_b + \gamma' d_t
\]

(11)

Then, the vertical effective stress oscillation \( \Delta \sigma'_v \), can be derived by subtracting the \( p_m \)
from the \( p_t \):

\[
\Delta \sigma'_v = (p_b - p_m) + \gamma' d_t
\]

(12)

In order to evaluate the real effective stress oscillation from the oscillatory excess pore pressure, the \( (p_b - p_m) \) should be corrected by \( \gamma' d_t \). The \( \gamma' d_t \) is easily calculated from the
\( d_t \) at each phase given in Fig. 13. Correcting the typical points on the dotted curve in Fig.
9, the open circles are obtained. It is clearly that the corrected plots lie on the solid curve
which represents the measured vertical effective stress oscillation, \( \Delta \sigma'_v \). This means that
Eq. (7) is applicable to any phase of a wave cycle.

Fig. 14 shows the liquefaction depth, \( d_t(p_m) \),
assessed from Eq. (6) by using the measured oscillatory excess pore pressures and the initial vertical effective stresses. In these cases, the thickness of the deposit, \( l = 1.90 \text{ m} \), is used. The maximum liquefaction depths of 0.3 m, 0.5 m and 0.75 m are observed respectively in Test Nos. 1, 2 and 3. No liquefied zone is, however, observed in the Test No. 4 in which the steel balls are placed on the surface of the deposit. The liquefaction tends to be restricted to happen when the vertical effective stress at calm is comparatively larger than the excess pore pressure which will be excited in the deposit. This is deduced from Eq. (9) as well.

Liquefaction and Behaviour of Model Structure

One of our interests is what happen to the structures if the oscillatory water pressures are imposed on the sea floor. A heavy-weight model structure the dimension of which is 190 mm high, 69.4 mm wide and 69.4 mm long and the weight of which is 2502 gf (24.5 N), is placed on the surface of the deposit. As a light-weight model structure, a plastic ball to which a rigid bar is attached is buried at the depth of 0.02 m in the deposit. The behaviour of structures is recorded by taking photos from outside of the transparent cylinder. Photo 1 is the example of the behaviour of the heavy model and the light model. Fig. 15 shows the movement of the models read and drawn from such photos as Photo 1. As can be seen in Photo 1 and Fig. 15, the heavy model is submerging and tilting by degrees in accordance with the number of waves. It has touched to the wall of the cylinder at the number of waves, about 400 cycles. The movement of the heavy model structure was more larger than the settlement of the deposit itself. Whereas, the light model is coming out of the deposit. The light model has finally floated out of the deposit at the number of waves, 49 cycles and 90 cycles. It is very interesting to note that the behaviour of the models is quite similar to that of the structures observed on or in the liquefied ground during earthquakes.

Fig. 16 is drawn to provide a rational explanation of these phenomena. The wave-associated vertical effective stresses, \( (p_0 - p_m) \) shown in Fig. 16 are measured in the same tests with those shown in Fig. 15. The test results in Fig. 16 (a) denote that the \( (p_0 - p_m) \) touches the dotted line of the initial vertical effective stress, \( \sigma'_v \), at the phase of 180°. This means that the liquefaction occurs
Photo. 1. Behaviour of model structure; (a) Heavy model, (b) Light model

Fig. 16. Vertical effective stress changes;
(a) Beneath a light model structure, (b) Beneath a heavy model structure
(1 kgf/cm$^2$ = 98 kPa)
above the depth of about 0.1 m in the deposit. Since the light model is buried in the liquefied zone in Fig. 16 (a), the gradual floatation of it can be attributed to the liquefaction induced repeatedly in the deposit. On the other hand, the distribution curves of the \((\rho_0 - \rho_m)\) in Fig. 16 (b) do not touch the dotted line, say the liquefaction is not necessarily induced in the deposit. Taking account of the consequent reduction of the shear strength due to the decrease of the vertical effective stress, however, the settlement of the heavy model can be considered to have been mobilized by the reduction or loss of the bearing capacity of the deposit.

**Effect of Liquefaction on Densification**

As mentioned in the previous section, notable increases of the relative density are observed near the surface of the deposit. These increases can be understood by taking into account the drastic change of the compressibility of the deposit due to the liquefaction. Fig. 17 shows a schematic drawing of the stress-strain relationship under one dimensional condition. The initial vertical effective stress lies on the point \(O_1\) in Fig. 17. When the stress oscillation with an amplitude of the \(\Delta \sigma'\) is generated in the deposit, the vertical strain, \(\varepsilon_{v1}\), is produced during the first cycle of loading shown by the curve \(O_1P_1Q_1O_2\). With the same manner as the first loading, the vertical strain, \(\varepsilon_{v2}\), is developed during the second loading cycle. The \(\varepsilon_{v1}\) is larger than the \(\varepsilon_{v2}\) because the stress-strain curve starting from the point \(O_1\) proceeds along the virgin curve \(O_1P_1\). If the amplitude of the vertical effective stress oscillation is increased from the \(\Delta \sigma'\) to the \(\Delta \sigma''\) so as to induce the liquefaction in the deposit, the stress and the strain at the moment of the liquefaction are represented by the point \(Q_2\) in Fig. 17. As the soil particles are in a suspended state there, the reconstitution of the soil skeleton will take place in the process of the line \(Q_2Q_3\). The loosely reconstituted deposit reveals a large vertical strain, \(\varepsilon_{v3}\), because the recompression is done again along the new virgin curve \(Q_3P_4\). Such a pass as the curve \(O_3P_3Q_3Q_3'O_4\) is repeated until the density of the deposit comes to a steady state. Therefore, the recompression after the liquefaction is considered to result in the large vertical strain and relative density of the deposit.

Fig. 18 shows the comparisons of the liquefaction depths among the \(d_1(p_m)\), \(d_1(S_t)\) and \(d_1(D_r)\). Where, the \(d_1(p_m)\), \(d_1(S_t)\) and \(d_1(D_r)\) denote the liquefaction depths respectively shown in Fig. 14, evaluated from the inflection points in Fig. 10 and from those in Fig. 11. As can be seen from Fig. 18, the liquefaction depths obtained by three ways are correlated very well with each others. The reason that the changes of the relative density in Test No. 4 has become so smaller
Table 3. Wave-induced and earthquake-induced liquefaction

<table>
<thead>
<tr>
<th>External forces</th>
<th>Earthquakes</th>
<th>Ocean Waves</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loading</td>
<td>Cyclic shear stress</td>
<td>Oscillatory water pressure</td>
</tr>
<tr>
<td>Input</td>
<td>Base</td>
<td>Surface</td>
</tr>
<tr>
<td>Period</td>
<td>Order of 1s</td>
<td>Order of 10s</td>
</tr>
<tr>
<td>Duration</td>
<td>Several minutes</td>
<td>A couple of days</td>
</tr>
<tr>
<td>Drainage condition</td>
<td>Undrained</td>
<td>Partially drained</td>
</tr>
<tr>
<td>Excess pore pressure</td>
<td>Positive, Gradual increase</td>
<td>Positive and negative, Oscillatory</td>
</tr>
<tr>
<td>Effective stress</td>
<td>Decrease</td>
<td>Decrease and increase alternately</td>
</tr>
<tr>
<td>Liquefaction phenomena</td>
<td>Occur at once</td>
<td>Occur transiently and repeatedly</td>
</tr>
</tbody>
</table>

than other test cases, can be attributed to the fact that the liquefaction was not induced in the deposit on account of the applied overburden pressure. Therefore, it may be concluded that the densification of the deposit has been remarkably progressed where the liquefaction was induced in the deposit. Bjerrum (1973) has suggested that the densification of the seabed could be caused by the cyclic shear stresses in the seabed. It is, however, found that the densification is possible to be caused only by the one dimensional vertical oscillation of the water pressure on the sea floor.

Wave-induced and Earthquake-induced Liquefaction

The differences of the characteristics between the wave-induced liquefaction and the earthquake-induced liquefaction are summarized in Table 3 and illustrated in Fig. 19. These are mainly led by the difference of the excitation or generation of the excess pore pressures in the two types of the liquefaction. The typical characteristics of the wave-induced liquefaction are that it occurs transiently and appears periodically so many times during a storm wave, being attended with the possible densification. The wave induced liquefaction, which reveals almost the same mechanism as the earthquake-induced liquefaction, is also reported to take place in real ocean environment (Lee and Foch, 1975; Rahman et al., 1978; Umehara et al., 1979; Ishihara and Yamazaki, 1984 and Zen et al., 1986). It is another type of the wave-induced liquefaction to be noted.

CONCLUSIONS

The mechanism of the wave-induced liquefaction and densification in seabed is examined in terms of the excess pore pressure. The following conclusions may be drawn:

1. The ocean waves excite the 'oscillatory' excess pore pressure in the seabed where the wave-associated bottom pressure is propagated into the seabed with some damping and phase lag.

2. The oscillatory excess pore pressure at arbitrary time and depth is expressed by the difference between the wave-associated bottom pressure and the oscillatory pore pressure in the seabed.

3. The magnitude of the change in the vertical effective stress is identical with the oscillatory excess pore pressure in the seabed.

4. The liquefaction in the seabed occurs where the wave-associated vertical effective stress becomes equal to the vertical effective stress at calm. Whereas, the densification is possible to take place where the wave-associated vertical effective stress exceeds the vertical effective stress at calm.

5. Even in one period of the wave loading, both the liquefaction and the densification occur alternately in the seabed.

6. The model experiments demonstrated

Fig. 19. Characteristics of wave-induced and earthquake-induced excess pore pressures
that the heavy-weight structures may submerge into the seabed, while the light-weight structures may float out of the seabed. The behaviour of models is similar to that of structures observed on or in the liquefied ground during earthquakes. It is found, however, that the wave-induced liquefaction is quite different from the earthquake-induced liquefaction in the mechanism of the generation of excess pore pressures.

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NOTATIONS

\( d_h \) = liquefaction depth  
\( H \) = wave height  
\( h \) = water depth  
\( L \) = wave length  
\( t \) = thickness of permeable deposit  
\( N_w \) = number of waves  
\( p \) = wave-associated water pressure  
\( p_o \) = wave-associated water pressure on the surface of seabed  
\( p_s \) = wave-associated water pressure at the bottom of liquefied deposit  
\( p_{os} \) = oscillatory pore pressure in seabed  
\( p_{a} \) = amplitude of \( p_o \)  
\( p_{s} \) = amplitude of applied dynamic pressure  
\( q_s \) = overburden pressure  
\( S_s \) = settlement in deposit  
\( T \) = wave period  
\( u_e \) = excess pore pressure  
\( e_v \) = vertical strain  
\( \theta \) = phase lag  
\( \sigma_v \) = vertical total stress  
\( \sigma_{ve} \) = vertical effective stress  
\( \sigma_{ve}^* \) = vertical effective stress at calm  
\( \Delta \sigma_{ve} \) = vertical effective stress increment

REFERENCES


