LIQUEFACTION CHARACTERISTICS OF SATURATED SAND DEPOSITS UNDER NONUNIFORM VERTICAL STRESSES

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ABSTRACT

A saturated sand deposit placed below a structure is subjected to nonuniform vertical stresses as shown in Fig. 4. It is known that there is an interesting experimental fact that a saturated sand deposit near a structure on the same horizontal plane liquefies more quickly than it below a structure. In order to clarify the liquefaction characteristics and the amount of liquefaction-induced settlement of a saturated sand deposit under nonuniform vertical stresses, a series of shaking table tests were performed on a simplified model (1), prepared in three Kjellman's type shear boxes. This model consisted of three saturated sand layers for which the effective vertical stresses \( \sigma_{vo} \) were 10.4 kPa, 30.0 kPa and 49.6 kPa respectively. Similar tests were also carried out on four models (models (2) to (5)) consisting of two sand layers having different \( RVS \) values \( (=\sigma_{vo}/\omega_{vo})h \), in which \( \sigma_{vo} \) and \( \omega_{vo} \) represent lower and higher vertical stresses for the two respective sand layers.

Since the values of \( RVS \) and \( k_h \) affect the permeability of and the transmission of pore water pressure between the sand layers, we found significant differences in liquefaction occurrence for such model sand layers.

It was also found that the liquefaction-induced settlement of the model sand layer increases substantially in the range of smaller vertical stresses and remains at an almost constant value as vertical stresses increase.

Key words: liquefaction, model test, pore pressure, sand, settlement, special shear test, vertical load (IGC: D7/E8)

INTRODUCTION

It has been reported that structures constructed as "fills" such as embankments, earth dams and other civil engineering structures have been damaged by the liquefaction of saturated sandy deposits directly below these structures during earthquakes (Fact-finding committee on the Niigata earthquake damage of JSCE, 1966; Seed et al., 1975; Sasaki et al., 1984).

Several experimental studies have been made on the liquefaction characteristics of saturated sand deposits where the vertical stresses differ, such as those below and near fill structures or other civil engineering structures. All tests were carried out using a sand box fixed on a shaking table. The model structures were classified into two types: One, a rigid model structure resting on a relatively loose saturated sand layer (Yoshimi and Tokimatsu, 1975, 1977, 1978; Ishihara and Matsumoto, 1975). The other, a model sand embankment founded on a relatively loose saturated sand layer (Koga and Matsuo, 1990; Abe and Kusano, 1991). The test methods and results are outlined below.

Yoshimi and Tokimatsu (1975, 1977, 1978) measured the amount of settlement of a rigid model structure placed on a sand layer which had a relative density of \( D_r = 60\% \) and pore water pressures \( u \) induced in the sand layer during vibration. The sand layer prepared in the box was 130 cm in length, 19.7 cm in width and 30 cm in depth. Two types of rigid model structures measuring 20 cm in length and 19.7 cm in width and with a load intensity \( q = 2.0 \) and 6.0 kPa were used. Free vibration with a maximum acceleration amplitude of 100 gal was applied to the sand layer.

Figure 1 shows the distribution of the maximum pore water pressure ratio \( (u/\sigma_{vo})_{\text{max}} \) induced in the sand layer during vibration (Yoshimi and Tokimatsu, 1975). As can be seen in Fig. 1, the \( (u/\sigma_{vo})_{\text{max}} \) value for the sand layer near the rigid structure (P2, P3, P5 and P6) is roughly equal to 1.0 indicating that liquefaction occurred at these points. The \( (u/\sigma_{vo})_{\text{max}} \) of the sand layer below the rigid structure (P1 and P4), however, is equal to 0.4 indicating that liquefaction did not occur at these points. Yoshimi

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and Tokimatsu proposed that due to the higher effective vertical stress induced in the sand layer directly below the rigid structure, compared with those induced in the sand layer near the structure, the sand layer would show relatively little effect of liquefaction.

Ishihara and Matsumoto (1975) studied the dynamic bearing capacity of the saturated sand layer ($D_r=25\%$) on which the model foundation was placed. In their experiments, the pore water pressures at five points (Fig. 2(a)) were measured. A sand layer was prepared in a sand box 4.0 m in length, 1.0 m in height and 1.0 m in width. Four model foundations measuring 0.5 m x 0.5 m with $q=7.1$, 8.7, 10.5 and 12.9 kPa respectively were used. The acceleration of vibration was increased gradually. The authors investigated the relationship between the pore water pressure $u$ induced in the sand layer and the value of $q$, at the time when the acceleration of vibration became large enough to cause the settlement of the model foundation (Fig. 2(b)). The acceleration amplitude was about 40 gal. As can be seen, these test results are similar to those obtained by Yoshimi and Tokimatsu. Points K2 and K3 in the sand layer near the structure liquefied, while Point K1 in the sand layer below the structure did not liquefy. It was also observed that the value of $u/\sigma''_o$ of the sand layer below the structure decreased as the value of $q$ increased. It was assumed that this was also due to the factors previously outlined by Yoshimi and Tokimatsu (1975).

It has been stated that the liquefaction resistance of saturated sand decreases slightly with an increase in the confining pressure (Tatsuoka et al., 1981; Yunoki et al., 1982). These results were obtained from liquefaction tests using a cyclic triaxial test apparatus. As shown in Fig. 8, the same results were obtained in our experiments.

It seems therefore difficult to accept Yoshimi and Tokimatsu's conclusions that the sand layer below the structure does not liquefy due to an increase in the effective vertical stresses induced in the sand layer by the weight of the structure. Koga and Matsuo (1990) carried out vibration tests on six model embankments founded on a sand layer with a
In order to clarify the liquefaction characteristics and the amount of liquefaction-induced settlement of the saturated sand deposit below and near a structure (Fig. 4), we performed a series of shaking table tests based upon the simplified model (1) prepared by using three Kjellman's type shear boxes. This model consists of three individual soil elements for which the vertical stresses were 10.4 kPa, 30.0 kPa and 49.6 kPa (Fig. 5). The adjacent elements were connected by rigid pipes in order to allow the free flow of pore water. The sand layers of $\sigma'_{vo} = 10.4$ kPa and 30.0 kPa corresponded to soil elements near the structure, and the sand layer of $\sigma'_{vo} = 49.6$ kPa corresponded to a soil element below the structure.

We also wanted to clarify the reason why a sand layer near a structure easily liquefies and the sand layer below the structure does not liquefy. Similar tests were performed on models (2) ~ (5) consisting of only two sand layers having different vertical stress ratios $RVS$. $RVS$ is defined by Eq. (1) as,

$$RVS = (\sigma'_{vo})/ (\sigma'_{vo})_h$$

in which $(\sigma'_{vo})$ and $(\sigma'_{vo})_h$ represent the lower and higher vertical stresses, respectively.

On the basis of these test results, the difference in the increase of pore water pressures induced in the models' saturated sand layers during vibration in conjunction with the $RVS$-values and the magnitude of $k_s$, can be discussed.

TEST APPARATUS

The details of the shear box were described in previous studies (Ôhara and Yamamoto, 1988, 1989, 1992), and therefore shall only be briefly described here. For this study, we used three Kjellman’s type simple shear boxes (Fig. 5). In order to allow the free flow of pore water, the adjacent sand layers were connected using rigid pipes (I.D. = 8 mm) (Fig. 5 (1)-(9)) which were filled with pore...
water. In order to prevent the lateral expansion of the sand layer and to induce a simple shear deformation, six 1 cm polyvinyl chloride rings (I.D. = 30.2 cm, O.D. = 35.0 cm) (Fig. 5(3)) were stacked around the sand layer, which was previously covered with a 1.0 mm rubber membrane (Fig. 5(6)).

PREPARATION OF MODEL SAND LAYER

The three saturated Toyoura sand layers for model (1) were prepared by first carefully pouring wet sand into a water-filled rubber membrane. The sand layers were then lightly compacted with a tamper to obtain sand layers with a D60 = 55%. The sand layer was 30 cm in diameter and about 6 cm in height.

The physical properties of the Toyoura sand are as follows. $G_s = 2.646$, $D_{max} = 0.85$ mm, $D_{min} = 0.16$ mm, $U_c = 1.7$, $e_{max} = 0.944$, $e_{min} = 0.610$.

A set of steel plate weights and/or lead ring weights was then placed on the surface of each of the sand layers in order to apply the required vertical stresses $\sigma_{oz}$. The magnitudes of $\sigma_{oz}$ were 10.4 kPa, 30.0 kPa and 49.6 kPa, respectively, at the bottom of the sand layer. Although a small initial shear stress might be induced in the sand layer of $\sigma_{oz} = 30.0$ kPa, the effect on the liquefaction characteristic of model (1) was neglected.

Models (2) ~ (5) consisting of two sand layers were also prepared using the same method. The values of $\sigma_{oz}$ and $RVS$ of all model sand layers are summarized in Table 1.

TEST PROCEDURE

Horizontal sinusoidal vibration with a period of $T = 1/3$ sec and the required constant acceleration amplitude was applied to the model under undrained conditions. This was achieved by opening valves (7) and (9). The seismic coefficient $k_s$ used in these tests was 0.09, 0.13 and 0.17. Thus, in these tests the magnitude of stress ratio $\tau / \sigma_{oz}$ was applied to the model sand layers during vibration was nearly equal to $k_s$. Here $\tau$ represents the amplitude of cyclic shear stress acting at the bottom of the sand layer.

During vibration, the pore water pressures at the bottom of the sand layers, the amplitude of shear deformation, the change in height of the sand layers and the acceleration of shaking table were all measured. The transducers for these measurements were pore water pressure transducers (capacity: 980 kPa), displacement transducers (capacity: 10 mm) and an accelerometer (capacity: 5 g), respectively.

After liquefaction occurred, vibration was stopped. The drainage valve (9) was then opened, and the amount of settlement of the sand layers induced by the drainage consolidation was measured.

Liquefaction tests were also performed on the three as independent sand layers. This was achieved by closing valves (7) and (9). These sand layers will hereafter be referred to as element sand layers.

One of the significant disadvantages of these model tests is that they are unable to simulate the phenomena observed in-situ such as subsidence of foundation (loss of bearing capacity) of the sand deposit and flow of pore water from a free field to the subsoil below a structure during liquefaction. The permeability in a horizontal direction of the sand layer is also much higher and the transmission of pore water pressure due to the high hydraulic gradient induced between two sand layers occurs more easily in the model compared with that of in-situ sand deposits.

LIQUEFACTION CHARACTERISTICS OF MODEL SAND LAYER

Typical test records of model (1) which were carried out at $k_s = 0.17$ and 0.09 are shown in Figs. 6(a) and (b), respectively. Fig. 6(c) shows typical test record of the element sand layers at $k_s = 0.09$. Fig. 6(a) shows that the three sand layers of model (1) liquefied almost simultaneously, and is similar to the test record shown in Fig. 6(c). Fig. 6(b) represents the result for the sand layer of $\sigma_{oz} = 10.4$ kPa liquefied, although both sand layers of $\sigma_{oz} = 30.0$ kPa and 49.6 kPa did not liquefy. It was determined that liquefaction occurred at a time when the pore water pressure induced in the sand layer was equal to the value of $\sigma_{oz}$. In Figs. 6(a) ~ (c), the recording lines show the acceleration of the shaking table, the shear strain of or volumetric strain of the sand layers and the pore water pressures induced in the sand layers where $\sigma_{oz} = 10.4$ kPa, 30.0 kPa and 49.6 kPa. In Figs. 6(b) and 6(c), $\varepsilon_s$ is calculated to be the amount of change in the height of the sand layer induced during the vibration divided by its initial height and denotes the volumetric strain of the sand layer.

Figure 7 indicates the relationship between the stress ratio $\tau / \sigma_{oz}$ and the number of cycles required to cause liquefaction $n_L$ for model (1). Symbol $\uparrow$ shown on the right-hand side of Fig. 7 indicates that the sand layer did not liquefy. For comparison, the test results of the element sand layers are also shown in this figure.

As shown in Fig. 7, the $\tau / \sigma_{oz} - \log n_L$ relationship for the element sand layers is shown by a curve irrespective of the magnitude of $\sigma_{oz}$. The liquefaction resistances for $n_L = 5$, 10 and 20 of the element sand layers decrease very slightly with increasing $\sigma_{oz}$, as can be seen in Fig. 8. Fig. 8 was prepared on the basis of the results of the element

<table>
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<th>Model</th>
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LIQUEFACTION CHARACTERISTICS OF SAND DEPOSITS

Fig. 6(a). Typical test record of model (1) ($k_a=0.17$)

Fig. 6(b). Typical test record of model (1) ($k_a=0.09$)

Fig. 6(c). Typical test record of element sand layers ($k_a=0.09$)
sand layers shown in Fig. 7 and those not shown in Fig. 7. This result agrees well with the results obtained from other liquefaction tests, which were carried out under a confining pressure of $\sigma_3 \geq 49.0$ kPa using the cyclic triaxial test apparatus (Tatsuoka et al., 1981; Yunoki et al., 1982).

As previously stated, in the case of $\tau/\sigma_{io}^\prime=0.17$ (Fig. 7), the three sand layers of model (1) liquefied simultaneously at almost the same $n_t=12$. This is equal to the $n_t$ for the element sand layers at $\tau/\sigma_{io}^\prime=0.17$. In the case of $\tau/\sigma_{io}^\prime=0.13$, however, the sand layer of $\tau/\sigma_{io}^\prime=10.4$ kPa liquefied at $n_t=18$, while both sand layers of $\sigma_{io}^\prime=30.0$ kPa and 49.6 kPa did not liquefy. This is smaller than the $n_t$ for the element sand layer of $\sigma_{io}^\prime=10.4$ kPa. It was demonstrated therefore that the liquefaction characteristics of the three sand layers of model (1) depend on the magnitude of $\tau/\sigma_{io}^\prime$, that is, $k_s$. This agrees well with the results for the sand layer below the model structure with high vertical stress did not liquefy, although the sand layer near the model structure with low vertical stress did liquefy (Yoshimi and Tokimatsu, 1975; Ishihara and Matsumoto, 1975).

Figures 9(a) and (b) show the time histories for the pore water pressures $u$ and the volumetric strains $\varepsilon_v$ induced in three sand layers of model (1) obtained from the test results shown in Figs. 6(a) and (b), respectively. Figure 9(c) shows similar curves for the element sand layers obtained from the test result shown in Fig. 6(c).

Figure 9(a) shows that in the case of $k_s=0.17$, since $n_t$ is small ($n_t=12$), all three sand layers liquefied at almost the same time. Furthermore, the pore water pressures induced in the three sand layers increased at approximately the same rate until the sand layers began to liquefy. The reason for this is that the three sand layers are connected with each other by rigid pipes filled with pore water. Similarly in the case of $k_s=0.09$ (Fig. 9(c)) the three element sand layers liquefied almost simultaneously at $n_t=76$. The pore water pressures induced in the three element sand layers however increased individually.

It was observed (Fig. 9(b)) that in the case of $k_s=0.09$ where $n_t$ is small, the pore water pressures induced in the three sand layers of model (1) increased at approximately the same rate until the sand layer of $\sigma_{io}^\prime=10.4$ kPa began to liquefy. As mentioned above, the sand layer of $\sigma_{io}^\prime=10.4$ kPa only liquefies at $n_t=18$, and this value of $n_t$ is smaller than the $n_t$ for the element sand layers. The reason for this is that the pore water pressure induced in the sand layer of $\sigma_{io}^\prime=10.4$ kPa is affected by the pore water pressures induced in the sand layers of $\sigma_{io}^\prime=30.0$ kPa and 49.6 kPa.

When the sand layer of $\sigma_{io}^\prime=10.4$ kPa liquefied, its particle structure loosened and the sand layer began to expand due to the pore water flowing from the sand layer of $\sigma_{io}^\prime=30.0$ kPa. Thereafter, the pore water pressures of the sand layers of $\sigma_{io}^\prime=30.0$ kPa and 49.6 kPa gradually decreased due to the flow of pore water to the sand layer of $\sigma_{io}^\prime=10.4$ kPa. When the sand layer of $\sigma_{io}^\prime=10.4$ kPa liquefied, the pore water pressure ratio $u/\sigma_{io}^\prime$ of the sand layer of $\sigma_{io}^\prime=30.0$ kPa was 10.4/30.0 = 0.35 and of $\sigma_{io}^\prime=49.6$ kPa, it was 10.4/49.6 = 0.21. This phenomenon observed in the sand layer of $\sigma_{io}^\prime=10.4$ kPa is different from “heaving”. The direction of flow of the pore water from the sand layer of higher vertical stress to that of lower vertical stress is different than the phenomenon observed in the field during liquefaction.

In our previous paper (Ohara and Yamamoto, 1988) it was shown that when $u/\sigma_{io}^\prime$ of the sand layer reaches $\phi$
of the sand layers of $\sigma'_w = 30.0$ kPa and 49.6 kPa (shown by the dotted line).

After the completion of vibration the pore water pressures of the two sand layers of $\sigma'_w = 30.0$ kPa and 49.6 kPa decreased until they come into equilibrium with the sand layer of $\sigma'_w = 10.4$ kPa.

EFFECT OF RVS ON LIQUEFACTION CHARACTERISTICS OF MODEL SAND LAYERS

In the preceding section, it was shown that a sand layer of lower vertical stress ($(\sigma'_w)$) in model (1), that is, near the structure always liquefies. A sand layer of a higher vertical stress ($(\sigma'_w)_h$), below the structure does or does not liquefy depending on the magnitude of $K_s$. It was then proposed that whether the liquefaction of the sand layer of $(\sigma'_w)_h$ occurs or not, depends on the value of RVS (Eq. (1)).

In order to confirm this last hypothesis, liquefaction tests were performed in addition on four models (models (2) to (5)) consisting of two sand layers for which the values for RVS differ. The RVS values for these models are shown in Table 1. The RVS value of model (1) is, for example, 10.4/30.0 = 0.35 (Table 1).

Figure 10 shows an example of the time histories of $u$ and $\varepsilon_v$ for model (2) with RVS = 0.52. This data was obtained from a test carried out at $K_s = 0.13$. It may be noted in this figure, that both sand layers liquefied at almost the same time $t = 9.3$ sec ($n_l = 28$).

The relationship between $U'/\sigma'_w$ and $n_l$ for each sand layer in models (2) to (5) is shown in Fig. 11. The curves for model (1) and the element sand layers (Fig. 7) are also presented in this figure. In the case of models (2) and (5) for which the RVS-values are over 0.5, the liquefaction characteristics of both sand layers are roughly equal to those of the element sand layers. In the case of models (3) and (4) however, for which the RVS-values are below 0.5, the sand layer of $(\sigma'_w)_h$ liquefies more easily than the element sand layers, whereas the sand layer of $(\sigma'_w)_h$ does not liquefy.

Figure 12 shows the relationship between the maxi-
mum pore water pressure ratio \( \frac{u}{\sigma_{o}'} \) max induced in the sand layer of \( \sigma_{o}' \)) and the \( RVS \)-value obtained from the tests on models (1) ~ (5). All data are of those obtained for \( k_s \leq 0.13 \). Test results obtained by Yoshimi and Tokimatsu (1975), and Ishihara and Matsumoto (1975) are also rearranged in this figure, referring to Figs. 1 and 2.

It may be noted, from Fig. 12 that, although the type of sand, the density of the sand layer, the test equipment and the test conditions for these three tests were completely different, the relationship between \( \frac{u}{\sigma_{o}'} \) max and \( RVS \) is represented by a unique curve. It may also be noted in Fig. 12 that when the \( RVS \)-values are below 0.5, the values of \( \frac{u}{\sigma_{o}'} \) max for all sand layers do not reach 1.0, that is, they do not liquefy.

On the basis of these results, it can be concluded that the liquefaction characteristics of the saturated sand layers for which vertical stresses are quite different is affected by the difference in the permeability of and the transmission of pore water pressure between the sand layers induced by the values of \( RVS \) and \( k_s \).

It was found in the case of \( k_s \leq 0.13 \) that the pore water pressures induced in the sand layers increased in proportion to the number of shear cycles \( n \) until \( u/\sigma_{o}' \) reaches 0.5 ~ 0.6. The \( a \)-value (Eq. (2)) was therefore defined as an index indicating the rate of increase of pore water pressures, and the relationship between \( a \)-value and \( k_s \) for the model sand layers, was determined to be:

\[
a = \frac{u}{\sigma_{o}'} / \left( n / n_c \right)
\]  

(2)

The results are represented in Fig. 13. It can be seen in this figure, that the \( a \)-values of all sand layers of \( \sigma_{o}' = 10.4 \) kPa of model (1) are over 0.65 and are considerably higher than those of all element sand layers (Fig. 13). The increase of the pore water pressures induced in the sand layers of \( \sigma_{o}' \) are influenced by the pore water pressures induced in the sand layers of \( \sigma_{o}' \) as mentioned above.

LIQUEFACTION-INDUCED SETTLEMENTS OF MODEL SAND LAYER

The amount of settlement of the three individual sand layers of model (1) caused by the drainage consolidation after liquefaction was measured. This settlement of the sand layers differs from that caused by loss of bearing capacity observed in the field during liquefaction was measured. The relationship between the volumetric strain \( e_v \) and \( k_s \) for the three sand layers is shown in Fig. 14(a). The same relationship for the element sand layers is shown in Fig. 14(b). A considerable scatter in the values for \( e_v \) for each \( k_s \) can be seen in both figures. This is due to the difference of the shaking time during liquefaction (\( t_{LS} \)). The \( t_{LS} \) for model (1) and the element sand layers are in the range of 4.5 ~ 9.7 sec and 4.3 ~ 9.5 sec, respectively. It was demonstrated in our previous study (Ö-
Hara and Yamamoto, 1992) that the value of $\varepsilon_v$ increases with increasing $\varepsilon_s$. Therefore, the relationship between $\varepsilon_v$ and $k_h$ for the sand layer of each $\sigma_{w0}$ should be obtained from the shadowed portion.

It can be seen from Figs. 14(a) and (b) that the amount of $\varepsilon_v$ for each sand layer increases with increasing $k_h$. When the value of $k_h$ is maintained constant, the volumetric strain for each sand layer increases as the vertical stress increases. This is because the inertia forces acting on the sand layers during vibration become larger as the vertical stress increases. The sand layers were also densified by the drainage after liquefaction.

It can be observed in Fig. 14(a) that the $\varepsilon_v$ induced in the sand layers of $\sigma_{w0}=30.0$ kPa and 49.6 kPa of model (1) which did not liquefy at $k_h$ in the range of 0.09 ~ 0.13, is 0.5% to 1.7%. This value is significantly smaller than the $\varepsilon_v$ in the range of $2.3 ~ 4.6\%$ for $k_h=0.09$ to $3.1 ~ 5.3\%$ for $k_h=0.13$ induced in the element sand layers of the same vertical stresses when liquefaction occurred (Fig. 14(b)).

Figure 15 shows the relationship between the liquefaction-induced volumetric strain $\varepsilon_v$ for model (1) and the vertical stress $\sigma_{w0}$ at $k_h=0.09$, 0.13 and 0.17 obtained from the results shown in Fig. 14(a). It can be seen from this figure that, in the case of $k_h=0.17$ the liquefaction-induced volumetric strain $\varepsilon_v$ of the sand layers increases rapidly up to $\sigma_{w0}=30.0$ kPa and remains at a constant value of 5% with any further increase in vertical stress.

**CONCLUSIONS**

The characteristics of liquefaction and liquefaction-induced settlement for saturated sand deposits where the vertical stresses are partially different such as those below or near embankments and structures were studied. Model (1) consisting of three sand layers was prepared in three Kjellman’s type shear boxes, and the liquefaction tests were performed using a shaking table. The vertical stresses $\sigma_{w0}$ applied to each sand layer were 10.4 kPa, 30.0 kPa and 49.6 kPa. For comparison, the same tests were carried out on four different models (models (2) ~ (5)) consisting of two sand layers for which the combination of vertical stresses differed.

The conclusions obtained from these tests can be summarized as follows: The liquefaction characteristics of the model sand layers depend on the ratio of vertical stresses $RVS$ as well as the seismic coefficient $k_h$. In the case of $k_h>0.17$, irrespective of the $RVS$-values and in the case of $k_h<0.17$ and $RVS>0.5$, all sand layers for these models have the same liquefaction characteristics as those of the element sand layers. On the contrary, in the case of $k_h<0.17$ and $RVS<0.5$, the sand layer of lower vertical stress liquefied considerably faster than the element sand layers due to the effect of pore water pressure induced in the sand layers of higher vertical stress, although the sand layers of higher vertical stress did not
liquefy.

Since the number of shear stress cycles necessary to cause liquefaction becomes larger at a small $k_s$, unless the pore water pressure ratio $u/\sigma'_w$ for the sand layers of $\sigma'_w = 30.0 \text{ kPa}$ and $49.6 \text{ kPa}$ reach about 0.5 when the sand layer of $\sigma'_w = 10.4 \text{ kPa}$ liquefies, the pore water pressures of these sand layers decreases gradually due to extensive expansion of the latter sand layer produced after its liquefaction. The former sand layers do not liquefy.

The liquefaction-induced settlements of model (1) at $k_s=0.17$ are, such that the amount of volumetric strain increases extensively up to $\sigma'_w = 30.0 \text{ kPa}$ and remains constant at 5% with any increase in vertical stress.

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