Effect of lateral stress on the liquefaction resistance of SCP-improved sandy soils

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**ABSTRACT:** When the sand compaction pile (SCP) method is implemented to improve loose deposits of sandy soils, its effect is evaluated generally in terms of increase in density which is beneficial for reducing the liquefaction potential of the deposits during earthquakes. Additional advantage can be expected to occur due to a concurrent increase in lateral stress. When the resistance to liquefaction is evaluated on the basis of SPT $N$-value or CPT $q_c$-value, the increased resistance to penetration due to the sand compaction has been interpreted conventionally as being associated mainly with the increase in density. Therefore, in order to properly evaluate the effectiveness of ground improvement in compacted soils, it is necessary to quantify the effect of lateral stresses on the penetration resistance and liquefaction strength. In this paper, charts incorporating the effect of various lateral stress ratios proposed earlier by the authors are used to quantitatively analyze the contributions of the increased lateral stress and increased penetration resistance on the resulting liquefaction resistance. Results indicate that if the increase in penetration resistance due to SCP implementation is larger, the effect of increased lateral stress becomes smaller and such trend is more pronounced in denser deposits.

1 **INTRODUCTION**

The effectiveness of compaction methods as ground improvement technique is usually evaluated through penetration resistances measured in-between sand piles before and after improvement in terms of $N$–values in standard penetration testing (SPT) or $q_c$–values in cone penetration testing (CPT). However, it is known that in addition to the increase in penetration resistance due to pile installation, there is also an increase in lateral stress in the ground. Most liquefaction potential evaluation procedures formulated for natural or reclaimed deposits make use of relations between liquefaction resistance and penetration resistance ($N$–value or $q_c$ value) assuming that the lateral stress $\sigma'_{h}$ in the sand deposit would be approximately equal to 0.5 times the vertical effective stress $\sigma'_v$, i.e., the $K_C$–value defined as $K_C = \sigma'_{h}/\sigma'_v$ is 0.5. However, for ground improved by compaction methods where $K_C > 0.5$, a different liquefaction resistance curve as illustrated in Fig. 1 can be defined which would include the effect of both increased penetration resistance and increased lateral stress. In the figure, $N_i$ and $q_{c1}$ are the SPT $N$–values and CPT $q_c$–values, respectively, normalized to an overburden pressure of 1 kg/cm², while $R$ is the liquefaction resistance of the ground.

In current design practice, however, only the effect of increased density on liquefaction resistance is incorporated. In order to fully assess the effectiveness of ground improvement, it is necessary to quantify both the effect of increased $K_C$–values and increased penetration resistance on the liquefaction resistance of remediated grounds. In this paper, both of these effects were analyzed based on recommended charts developed earlier by the authors (Harada et al., 2008), and the proportions of their contributions to the increased liquefaction resistance were quantified.
In Japan, the sand compaction pile (SCP) method is the most popular method of remediating liquefiable ground. As illustrated in Fig. 2, this method involves the installation of well-compacted sand piles of large diameters through the process of repeated driving down and extracting motion of a vibrating steel pipe. As the sand pile is compacted and enlarged, the adjacent ground is pushed laterally and compacted, resulting in increased density of the ground as well as increased lateral stress.

To illustrate such increase in density, typical SPT $N$-values obtained from sites improved by both vibratory SCP and non-vibratory (Nv) SCP procedures are shown in Fig. 3(a) while examples of CPT $q_c$ values from both vibratory and non-vibratory (Nv) SCP–improved ground are illustrated in Fig. 3(b). It is observed that penetration resistances obtained between the installed sand piles are increased as the piles pushed and displaced the adjacent sandy ground.
Moreover, results of cases where various instruments (e.g., pressuremeters and dilatometers) were used to measure the lateral stresses before and after implementation of both vibratory and non-vibratory SCP methods are presented in Fig. 4. In the figure, the relation between the lateral stress ratio, $K_C$, and improvement ratio, $a_s$, are plotted 1 month and 2 years after the SCP operation. Note that the data points corresponding to $a_s=0$ refer to the condition prior to the implementation of SCP method. It can be observed that substantial increase in $K_C$–values are observed after SCP implementation, with larger increase in $K_C$ values occurring at higher $a_s$.

**EFFECTS OF LATERAL STRESS ON PENETRATION RESISTANCE AND LIQUEFACTION RESISTANCE**

As mentioned earlier, conventional curves relating penetration resistance and liquefaction resistance are used when evaluating the effectiveness of the SCP method. However, it has been confirmed that any increase in penetration resistance and liquefaction resistance includes the effect of both increased density and increased $K_C$–value. Therefore, when evaluating liquefaction resistance using penetration resistance alone, a problem of “double count” exists and this needs to be eliminated to accurately reflect the effectiveness of improvement.

Toward this end, the authors have developed a methodology to quantify separately the effect of increased penetration resistance and increased $K_C$–value on liquefaction resistance (Harada et al.,...
The methodology adopted in the study is summarized below.

(1) The relations between liquefaction resistance $R$ and the penetration resistance ($N_1$-value or $q_{c1}$-value) commonly used both in Japan (e.g., JRA, 1996; AIJ, 2001) and in North America (Youd et al., 2001; Robertson et al., 1998) were employed as reference curves. These are based on a number of laboratory tests and performance data during past earthquakes and it was assumed with good reasons that all the relations between $R$ and $N_1$ or $q_{c1}$ are applicable for deposits consolidated under the $K_c=0.5$ condition. Note that neither the $R-N_1$ relation nor the $R-q_{c1}$ relation were addressed without reference to the relative density $D_r$.

(2) The effect of relative density $D_r$ is introduced in the reference curves relating liquefaction resistance and penetration resistance mentioned in step (1). This is done by expressing the penetration resistance in terms of $D_r^a$ either through the void ratio range ($e_{\text{max}} - e_{\text{min}}$) or mean grain size $D_{50}$, using the same formulations proposed by Cubrinovski and Ishihara (1999).

(3) The effects of $K_c$-conditions on liquefaction resistance $R$ and on penetration resistance $N_1$ or $q_{c1}$ need to be known. For this purpose, the results of laboratory chamber tests available in the literature (e.g., Harada et al., 2000; Huang and Hsu, 2005; Salgado et al., 1997) were compiled to derive the relations between different $K_c$-conditions and penetration resistances for soils with different $D_r$. Moreover, the effects of different $K_c$-conditions on liquefaction resistance $R$ as investigated by Ishihara and Takatsu (1979) using cyclic torsional test results were employed.

(4) In the final step, the influence of relative density on the above relations was eliminated and the relations between the liquefaction resistance $R$ and penetration resistance $N_1$ or $q_{c1}$ were derived for different $K_c$-conditions. Thus, the effect of $K_c$-values on proposed correlations between $R$ and $N_1$ or $q_{c1}$ both in Japan and North America were derived.

The details of the above procedure are presented in a paper by Harada et al. (2008) and summarized in the flowchart illustrated in Fig. 5. Based on the steps of evaluation performed as outlined above, the relation between penetration resistance and liquefaction resistance for the case of $K_c=0.5$ was modified to account for different $K_c$-values. From these relations, charts were derived as shown in Figs. 6 and 7 for penetration resistance in terms of $N_1$-values and $q_{c1}$-values, respectively.

Note that in the charts presented, an approximate relation ($N_1)_{90}=1.3(N_1)_{80}$ was incorporated to correct for the difference in energy transfer between Japanese and American SPT practice. Moreover, a 0.65 factor was used to take into account the fact that liquefaction resistance in Japanese codes is expressed in terms of the maximum acceleration, while the average value of acceleration during seismic shaking is used in American practice.
Based on the above discussion, the increase in liquefaction resistance of ground improved by the sand compaction pile method is due to two components i.e., the increase in penetration resistance and increase in $K_C$-values. From the charts presented, it is noted that for ground with low penetration resistance (loose deposit), the gradient due to increase in $K_C$ is much greater than the gradient coming from the penetration increase alone. This indicates that the effect of $K_C$ on $R$ is more significant than the effect of penetration resistance for a loose state of deposits. On the other hand, when the ground has high penetration resistance (dense deposit), the gradient of the liquefaction curve for $K_C=0.5$ is generally high, indicating that the effect of penetration resistance is much more significant than the effect of $K_C$. Thus, it can be said that with increasing $K_C$-value, the liquefaction resistance increases, but its effect becomes smaller at higher density.

4 COMPONENTS CONTRIBUTING TO INCREASE IN LIQUEFACTION RESISTANCE

Fig. 6. Recommended charts correlating SPT $N_1$-value and liquefaction resistance $R$ for various $K_C$-values based on: (a) Japanese code; and (b) American practice.

Fig. 7. Recommended charts correlating CPT $q_{c1}$-value and liquefaction resistance $R$ for various $K_C$-values based on: (a) Japanese code; and (b) American practice.
To expound on this in more detail, the contributions of increased $K_C$–value and increased penetration resistance on the resulting increase in liquefaction resistance were analyzed quantitatively. Both liquefaction curves based on Japanese and American practices were considered. For illustration purposes, the data for loose (pre–SCP $N_1$–value=5 or $q_{c1}=5$ MPa) and dense deposits (pre–SCP $N_1$–value=15 or $q_{c1}=10$ MPa) were evaluated for $K_C=0.5, 1.0$ and $1.5$.

The results are illustrated in Figs. 8 and 9 for $N_1$–values based on Japanese and American practice, respectively, while the corresponding results are given in Figs. 10 and 11 for $q_{c1}$–values, respectively. The left graphs in each figure correspond to low initial (pre–SCP) penetration resistances, while the right graphs are for higher penetration resistances. In the graphs, the vertical axes represent the increase in liquefaction resistance $\Delta R$, while the horizontal axes show the increase in penetration resistance, i.e. $\Delta N_1$ or $\Delta q_{c1}$. The numbers indicated in the charts correspond to the contribution of increased penetration or increased $K_C$–value (from 0.5 to 1.0, or from 1.0 to 1.5). For example, consider the left-most graph in Fig. 8(a), representing a loose deposit ($N_1=5$) prior to SCP implementation. After compaction, $N_1$–value rose to 10, and such increase in penetration resistance alone accounted for 54% of the total increase in liquefaction resistance while the remaining 46% was due to increase in $K_C$–value from 0.5 to 1.0. On the other hand, if the $K_C$–value is increased from 0.5 to 1.5 during SCP implementation, the contribution of the increase in $N_1$–value to the increase in $R$ is about 39%, while the contributions of the increase in $K_C$ from 0.5 to 1.0 and from 1.0 to 1.5 are 33% and 28%, respectively.

For all figures, it can be observed that the larger the increase in penetration resistance, the increase in liquefaction resistance becomes higher. However, compared to the contribution of increase in $K_C$–values, the contribution of increase in penetration resistance is relatively more significant, accounting for about 50–80% of the increase in liquefaction resistance in the case of high increase in penetration resistance (e.g., if $\Delta N_1=15$ or 20, or $\Delta q_{c1}=7.5$ or 10 MPa). Moreover, it is observed that such trend is stronger when the initial penetration resistance is high or if the penetration testing is done through CPT. Similar trends were observed for the charts correlating $R$ and $N_1$ or $q_{c1}$ based on North American practice.

5 CONCLUSIONS

In this paper, the effect of increased penetration resistance and increased lateral stresses on the liquefaction resistance of ground improved by the sand compaction pile (SCP) method was investigated through the design charts initially proposed by the authors that can account for various lateral stress ratios $K_C$. Based on the results of detailed investigation, the larger the increase in penetration resistance, the higher the increase in liquefaction resistance. If the increase in penetration resistance due to SCP is larger, the contribution of increased $K_C$ to the increased liquefaction
resistance becomes smaller, with the trend being more pronounced in initially denser deposits.

The proposed charts presented herein have been developed for clean sand deposits and an extension of this study to ground containing some amount of fines is planned in the future.

Fig. 9. Plots showing the contributions of increased $N_1$–value and $K_C$–value on the increase in $R$ for grounds with: (a) low and (b) high initial SPT $N_1$–values (based on American practice).

Fig. 10. Plots showing the contributions of increased $q_{c1}$–value and $K_C$–value on the increase in $R$ for grounds with: (a) low and (b) high initial CPT $q_{c1}$–values (based on Japanese practice).

Fig. 11. Plots showing the contributions of increased $q_{c1}$–value and $K_C$–value on the increase in $R$ for grounds with: (a) low and (b) high initial CPT $q_{c1}$–values (based on American practice).
REFERENCES


