An Experiment of Consolidation Behavior for Partly and Fully Penetrated SCP Ground

Jung, Jong-Bum[*1] Moriwaki, Takeo[*2]
Lee, Kang-Il[*3] Kang, Kwon-Su[*4]
Park, Byong-Kee[*5]

요 지

본 연구에서 미관통과 관통 샌드컴팩션파일(SCP) 개량지반에 대하여 일차원 압밀거동을 조사하기 위하여 실내모형실험을 실시하였다. 모델점토지반에 간극수압계, 토압계, 다이얼게이지등을 설치하여 SCP 개량지반의 압밀침하량과 응력분담비등을 측정하였다. 실험 결과, 미관통 SCP 지반의 압밀침하량은 관통 SCP 지반의 압밀침하량 보다 더 크게, 응력분담비는 더 적게 나타났다. 또한 SCP 개량지반의 응력분담비는 압밀시간, 지반심도, 상대밀도, 치환율과 관통률에 따라 변화됨을 알 수 있었다.

Abstract

A series of model tests was conducted to investigate the one-dimensional consolidation behavior of an improved ground where sand compaction piles(SCP) were either fully or partly installed in the model clay ground. In order to check the one-dimensional consolidation settlement and stress concentration ratios, earth pressure, pore pressure transducers and dial gauges were installed in the model clay ground. The test results revealed that the consolidation settlement of the partly penetrated SCP ground was larger than that of the fully penetrated SCP ground, and the stress concentration ratios (m) of the fully penetrated SCP ground were higher than these of the partly penetrated SCP ground. The stress concentration ratio was decreasing with the increase in the penetration depth of SCP.

Keywords : Model tests, Consolidation, Sand compaction piles, Partly penetrated SCP, Stress concentration, Penetration ratio, Relative density

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[*1] Member, Doctor of Engineering, Dept. of Civil and Environmental Eng., Hiroshima University, Japan
[*2] Associate Professor, Dept. of Civil and Environmental Eng., Hiroshima University, Japan
[*3] Member, Assistant Professor, Dept. of Civil Eng., Daejin University
[*4] Member, Assistant Professor, Dept. of Civil Eng., Seonam University
[*5] Member, Professor, Dept. of Civil Eng., Chonnam National University

1. Introduction

The method of the sand compaction piles has been mainly developed in Japan, since a patent for SCP method was applied in 1956. The principles of the SCP method are based on the research papers by Murayama and Tanimoto published in 1957 and 1960, as the hammering method was developed in 1957(Ministry of construction 1957). A composite foundation consisting of rows of compacted large diameter sand columns were driven into a soft clay ground in coastal district of Japan, as the foundation of a structure(Murayama 1957, 1962).

The SCP method was further used to improve soft clays since the 1960's(Ogawa 1963, Ibaraki 1965). The subsequent presentation of Murayama's paper in 1962 established the method of SCP application to clayey soil. This method was further justified through a number of researches and construction projects.

Moreover, the installation method of SCP on the sea was developed in 1967 in order to extend the applicability of this method. As the period of soil stabilization is considerably shortened by this method, compared to vertical drains, its usage has become one of the mainstreams of soil stabilization technique for offshore and onshore structures in Japan since the 1970's. The automatically controlled SCP driving system was invented in 1981 accommodating the vibration effect on soil properties. In the western world, stone column method was used to improve the soft ground since the 1970's(Baumann 1974, Hughes 1975).

Most of the researches on the SCP method have been focused on mainly 5 points: (1) bearing capacity and stability(Matsuo 1967, Kimura et al 1985), (2) consolidation settlement and deformation(Aboshi et al 1979), (3) design methods(Ichimoto 1981), (4) construction problems(Yoshikuni et al 1988) and (5) partly penetrated SCP problems(Kitazume et al 1995).

At present, the design method of the sand compaction piles is usually divided into two methods. One is to check the stability of improved ground in which a slip surface is assumed to pass through the replacement area composed of sand column and clay. Stability analysis is carried out by the circular arc method. A replacement area ratio and a stress concentration ratio on the slip surface are needed for the stability analysis. The other method is to check the consolidation settlement of the composite ground by considering the diameter, spacing and arrangement of the sand compaction piles and the stress concentration ratio of the composite ground. In this research, to clarify the consolidation behavior of an improved ground, one-dimensional consolidation tests were carried out on specimens of Hiroshima clay where partly and fully penetrated SCP were installed.

2. Materials

The clay used was dredged marine Hiroshima clay and the sand used was Toyoura sand, of which index properties are presented in Table 1. The mechanical properties of Hiroshima clay are shown in Table 2, which were obtained from oedometer tests and consolidated-undrained triaxial compression tests(CU or $\overline{\text{CU}}$) in the laboratory. The clay was remoulded at a water content of about twice the liquid limit and the slurry was then deaired in a vacuum container at a pressure above 700mm/Hg for 6 hours. The dry sand was poured into acrylic pipes having 36 and 46mm as inner diameter at the relative density of 70, 80%, and saturated by supplying deaired water from the bottom of the pipe. The sand piles were then frozen in a refrigerator at a temperature of -40° C.

Table 1. Physical properties of Hiroshima clay and Toyoura sand

Hiroshima clay				Toyoura sand		
Specific gravity, G_s	Liquid limit, $W_L(\%)$	Plastic limit, $W_P(\%)$	Plasticity index, $I_P(\%)$	Specific Gravity, G_s	Max. dry unit weight, $\gamma_{dmax}(kN/m^3)$	Min. dry unit weight, $\gamma_{dmax}(kN/m^3)$
2.617	110.0	43.2	66.8	2.640	16.147	13.067

Table 2. Mechanical properties of Hiroshima clay

Rate of strength increase c/△p	Coefficient of consolidation $C_v(cm^2/min)$	Compression index C_c	Swelling index C_s
0.357	55.2	0.758	0.105

3. Apparatus

In order to measure pore water pressure in the model clay ground, miniature pore water pressure transducers(PPTs, Druck PDCR81. Max.=686kPa), were installed at various locations as shown in Fig. 1. A deaired and saturated porous stone was fitted in front of each transducer for the purpose of minimizing the response time of the transducer to pore water pressure changes. It also helped to protect the silicone diaphragm of the transducer. The earth pressures in the model ground were measured by earth pressure transducers(ϕ =1cm) of 490kPa maximum capacity(Sankei Eng. P310-5). The pore pressure and earth pressure transducers were powered by a 5 volts DC supply. The transducers were installed at various locations as shown in Fig. 2. The load cell and three dial gauges were set to measure the consolidation pressure and the consolidation settlement of the specimen, respectively.

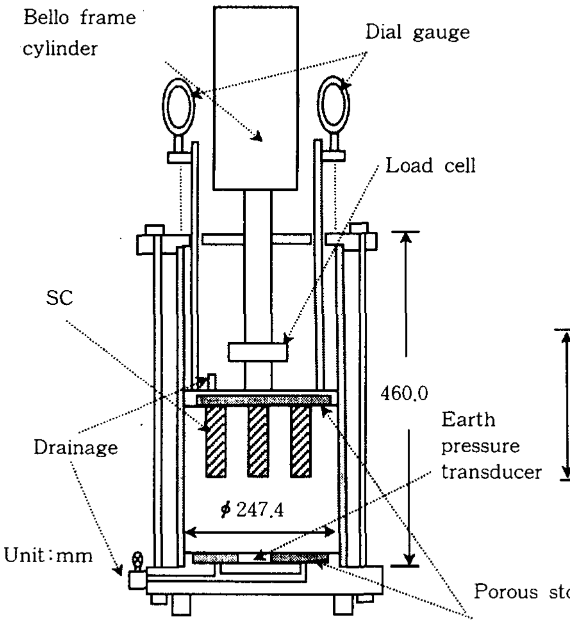


Fig. 1 Schematic diagram of apparatus

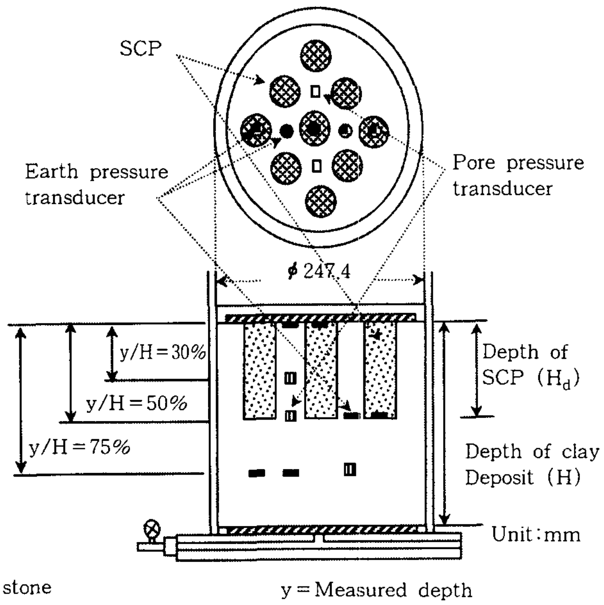


Fig. 2 Schematic of SCP ground and sensors

4. Test Procedures

The test procedures were as follows. After the self-weight consolidation for 2 days, the clay specimen was consolidated at a pressure of 49kPa in the double drainage condition. At least 2 weeks were necessary to reach 90% of the degree of consolidation. The thickness of the clay ground became approximately 18cm after primary consolidation. Holes of 36mm or 46mm diameter were carefully drilled in the consolidated clay with a hand drill and the earth pressure transducers were installed between the SCPs whereas pore pressure transducers were installed in the clay ground as shown in Fig. 2. Then, the frozen SCPs with earth pressure transducers in the center of SCP were inserted into the clay deposit in square pattern with a center to center distance of 24mm, which corresponds to the replacement area ratio(a_s) of 28.8% and 40% for the SCP diameter of 36mm and 46mm, respectively. Preconsolidation pressure of 49kPa was loaded for four days to remove the influence of stress release. When the preconsolidation process was completed, the specimen was further consolidated to a pressure 98kPa under the single(upper of the specimen) drainage condition.

In order to compare the effect of penetration ratio, relative density of SCP and replacement area ratio, ten model tests were conducted. Table 3 gives the program of these tests.

Table 3. Program of tests

Test code	SCP diameter, ϕ(cm)	Replacement area ratio, a_s (%)	Relative density of SCP, D_r (%)	Penetration ratio, H_d/H (%)	Total depth of ground, H (cm)
TC-1	0	0	0	0	17.20
TC-2	3.6	28.8	80±1.0	50	16.85
TC-3				75	17.10
TC-4				100	17.57
TC-5	3.6	28.8	70±1.0	50	16.90
TC-6				75	17.25
TC-7				100	17.85
TC-8	4.6	40.0	70±1.0	50	17.01
TC-9				75	17.34
TC-10				100	17.95

5. Test Results and Discussions

5.1 One-dimensional Settlement

Fig. 3 shows the relationship between the settlement at the top surface of the model clay ground and the elapsed time. From this figure it is found that the consolidation settlement of the ground improved by the SCPs is less than that of the non-treated clay ground. Likewise the consolidation settlement of the partly penetrated SCPs ground is larger than that of the fully penetrated SCPs ground. Consolidation settlement of the ground with a high replacement area ratio is less than that of the ground with low replacement area ratio. The difference in relative density of the SCPs between $D_r = 70\%$ and 80% hardly influences the consolidation behavior.

The relationship of the final settlements between the improved ground with the penetration ratio(H_d/H) and the non-treated ground is driven from the following equation. The amount of the final settlement of the partly penetrated SCPs ground is linearly proportional to the penetration ratio.

$$S_f = \{1-(1-\beta)\cdot H_d/H\}\cdot S_{f_0} \qquad (1)$$

$$\beta = \frac{1}{1+(m-1)\cdot a_s} = s_f/s_{f_0} \qquad (2)$$

where, S_f is final settlement of the improved ground with the penetration ratio of H_d/H, S_{fo} is final settlement of non-treated ground, β is the coefficient of settlement reduction and m is the stress concentration ratio. S_{fo} can be calculated by the compression index($C_c = 0.758$) by considering the initial depth for each tests. The stress concentration ratio(m) can be calculated by substituting the measured value of S_f and the calculated value of S_{f_0} in Equation (1) and (2).

Table 4. Calculated stress concentration ratio from final settlement

Replacement area ratio, a_s (%)	Relative density, D_r (%)	Penetration ratio, H_d/H (%)	Final settlement by 3t method, S (cm)	Calculated stress concentration ratio *, m	Measured stress concentration ratio, m
28.8	80	100	0.421	7.67	4.694
		75	0.521	5.73	3.924
		50	0.769	3.08	3.530
28.8	70	100	0.444	7.14	4.559
		75	0.563	5.11	3.699
		50	0.864	2.47	3.328
40.0	70	100	0.294	8.96	4.058
		75	0.417	5.87	3.465
		50	0.653	3.20	3.116

* Stress concentration ratio was calculated from Equation (2).

Table 4 shows the stress concentration ratio calculated from the final settlement of model tests. It can be seen from Table 4 that the obtained stress concentration ratio($m=7\sim9$) in the fully penetrated SCPs ground is larger than the value($m=3\sim5$) used in present design practice.

Comparing the stress concentration ratio of the partly penetrated ground with that of the fully penetrated ground, the stress concentration ratio decreases with the decrease of the penetration ratio. Namely, the values of $m=5\sim6$ for the $H_d/H=75\%$ and $m=2\sim4$ for the $H_d/H=50\%$ correspond to about 1/1.5 and 1/3 for the value of the fully penetrated SCPs ground, respectively.

From Table 4, it is found that the stress concentration ratio increases with relative density and replacement area ratio of the SCPs in both partly and fully penetrated SCPs ground.

Fig. 4 shows the relationship between the stress concentration ratio and the penetration ratio of the SCPs. The obtained stress concentration ratio varies linearly with the penetration ratio within the modeled range. As mentioned above, the stress concentration ratio used in the present design method is around $3\sim5$. However, for the fully penetrated SCPs ground the obtained stress concentration ratio is $m=7\sim9$ and is bigger than that of the present design method. This indicates that the present design method for the prediction of the final settlement underestimates(m) value for the fully penetrated SCP ground.

From these results, it can be concluded that the consolidation settlement strongly depends upon the penetration ratio. It should be noted that the consolidation behavior is explained by the degree of consolidation and depth.

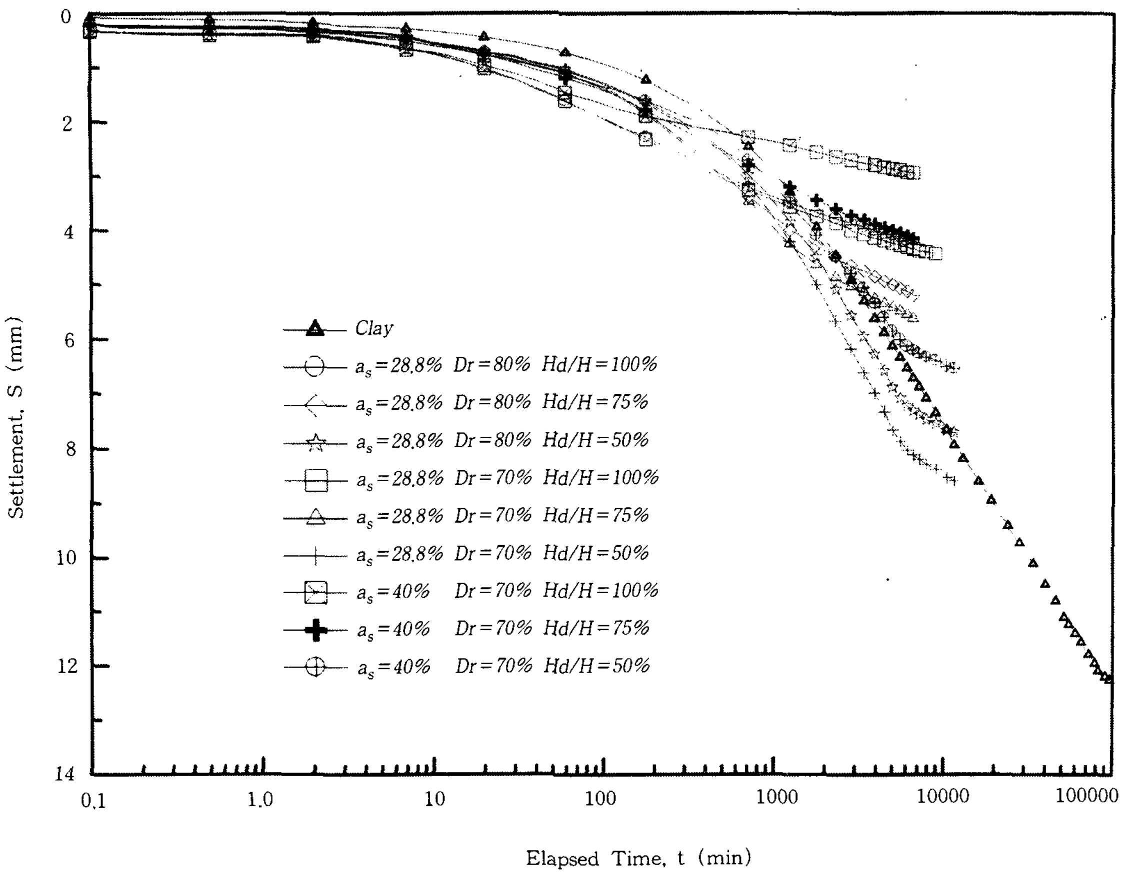


Fig. 3 Settlement-time curves of SCP and clay ground

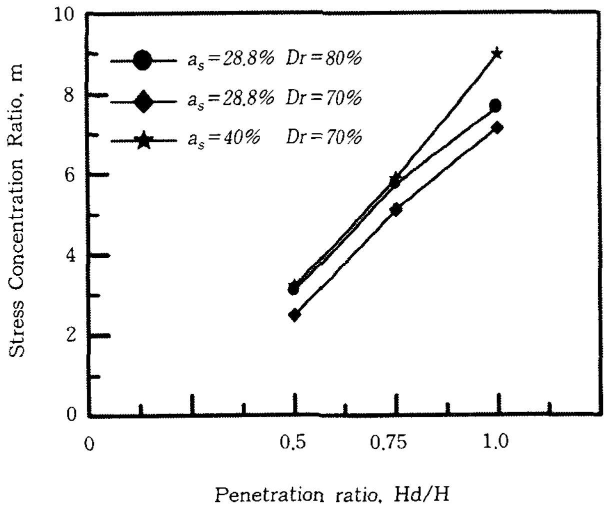
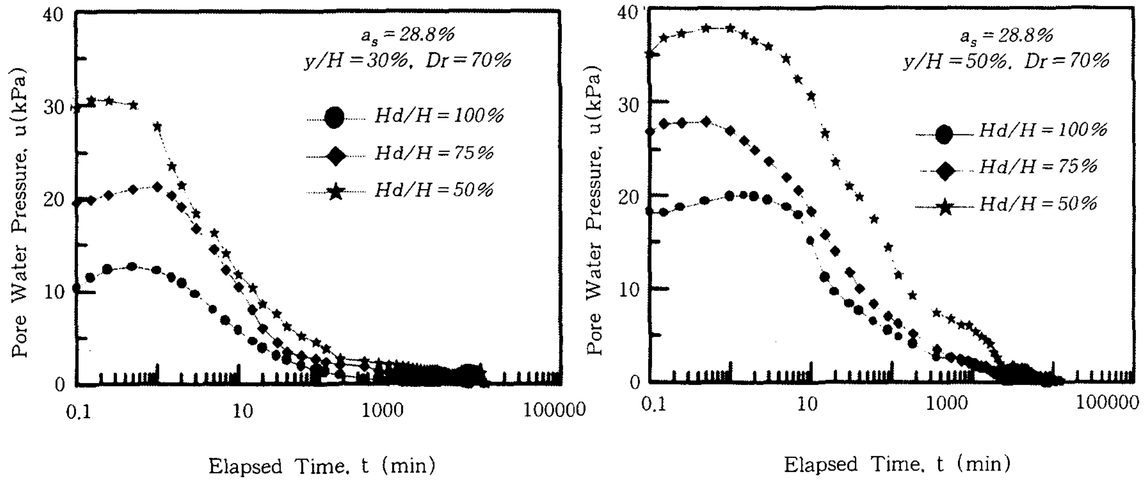


Fig. 4 Comparison of calculated stress concentration and penetration ratio

5.2 Dissipation of Pore Water Pressure

Fig. 5 (a) and (b) show the dissipation of pore water pressure for the replacement area ratio as = 28.8% and relative density of the SCPs D_r = 70% at the depth y/H = 30% and 50%, respectively. Fig. 6 shows the relationship between normalized pore water pressure and time at depth y/H = 50%. Pore water pressure was normalized by increased consolidation pressure, $\Delta\sigma_c$ = 49kPa in the fully and partly penetrated SCPs ground.

From Fig. 6(a), it is found that the relationship of the pore water pressure in the case of D_r = 80% is slightly faster than that in the case of D_r = 70%. Although the SCPs were penetrated to the depth where the pore water pressure was measured, the pore water pressure becomes faster with an increase of the penetration ratio of the SCPs. It is also found from Fig. 6(b) that in the case of a_s = 40% the pore water pressure at the start of consolidation has grown into the value equivalent to the clay stress calculated by using the stress concentration ratio shown in Table 4. However, in the case of a_s = 28.8% it has grown into about 1.2 to 1.4 times of the clay stress calculated by the same way. This implies that the stress calculation ratios obtained from the final settlement for the improved ground with a low replacement area ratio are larger than the actual values.

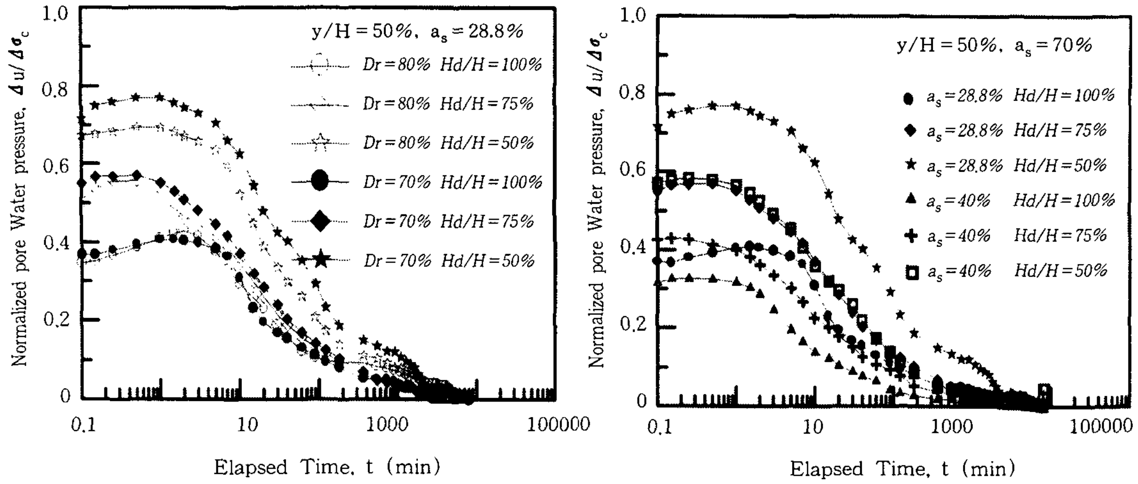


(a) Depth(y/H = 30%)

(6) Depth(y/H = 50%)

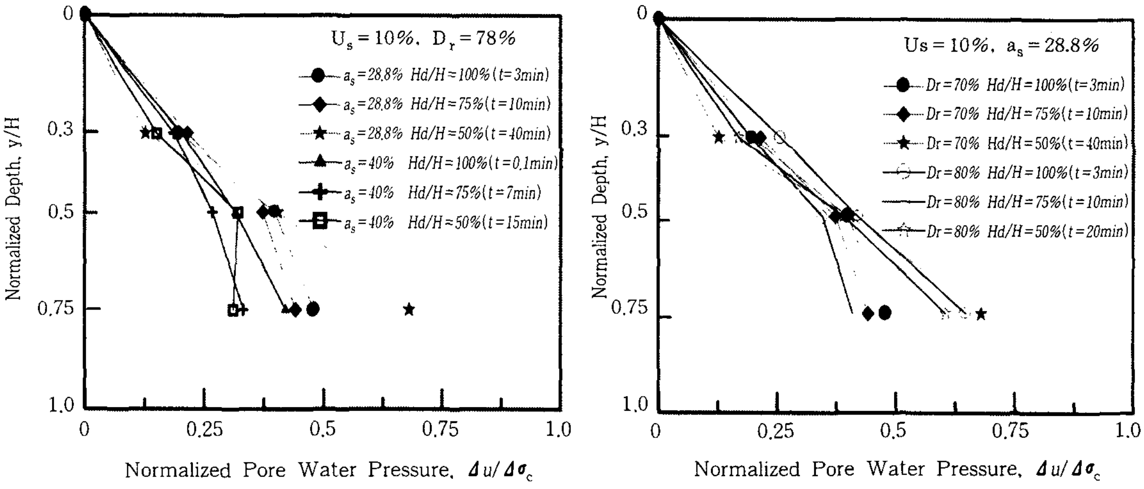
Fig. 5 Relationship between pore water pressure versus time(as = 28.8%, Dr = 70%)

Furthermore, Fig. 7 and Fig. 8 show the relationship between the normalized depth and the normalized pore water pressure at the degree of consolidation of U = 10% and 70%, respectively. From Fig. 7, it can be found that the degree of consolidation defined by the pore water pressure is roughly estimated as U = 50~70% and is larger than that determined by the amount of the settlement, U = 10%. The reason of this result is that the settlements caused by the secondary



(a) Replacement area ratio (a_s =28.8%)　　(b) Replacement area ratio (Dr =70%)

Fig. 6 Relationship between normalized pore water pressure versus time(y/H = 50%)



(a) Relative density (D_r =70%)　　(b) Replacement area ratio (a_s =28.8%)

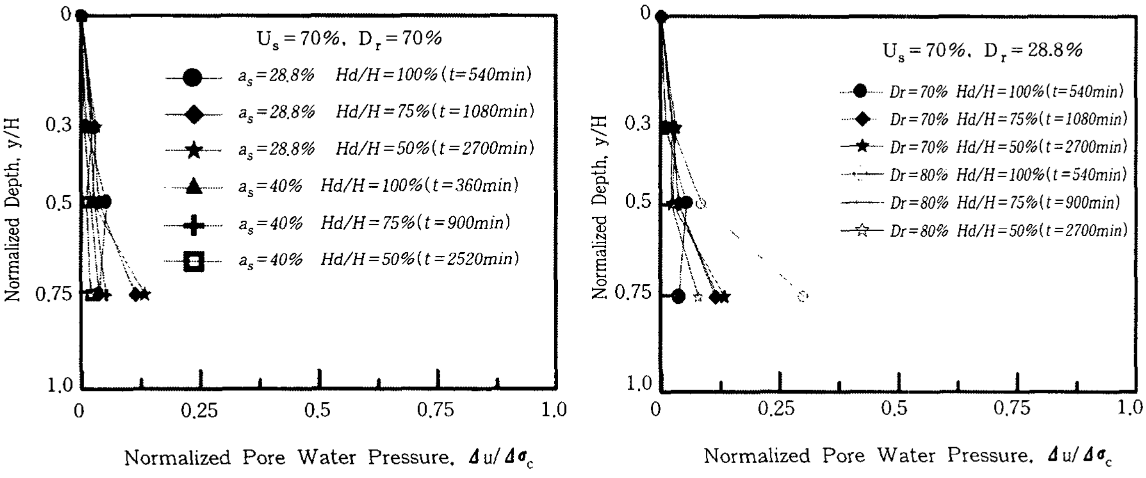
Fig. 7 Distribution of normalized pore pressure versus normalized depth(U = 10%)

consolidation and the redistribution of the vertical stress from the SCPs to the around clay are included in the final settlement determined by the 3t method(Japanese Geotechnical Society 1990).

It is also found that the dissipation rate of the pore water pressure increases in the layer where the SCPs are penetrated and decreases in the layer below the bottom of the SCPs with the decreases of the penetration ratio of the SCPs. The influences of the replacement ratio and the relative density of the SCPs on the isochrone of the pore water pressure cannot be found from Fig.

7 and 8. Especially, the pore water pressure in Fig. 8, where the degree of consolidation is U = 70%, is almost dissipated in all test cases in spite of the differences of the replacement area ratio and the relative density of the SCPs in each test cases.

From these results, it can be concluded that the behavior of pore pressure changed with the penetration ratio and depth. The dissipation of the pore pressure of the fully penetrated SCP ground is faster than that of the partly penetrated SCP ground. At the partly penetrated SCP ground, the area below the SCPs is strongly influenced by the dissipation of pore water pressure in the improved ground. The dissipation of pore water pressure in clay ground below the bottom of the SCPs has to be considered in the calculation of consolidation settlement in the design and construction. The pore water pressures with a high replacement area ratio showed generally faster dissipation than a low replacement area ratio at the same degree of consolidation. These show that the dissipation of pore pressure is strongly dependent on the penetration ratio and the replacement area ratio of improved ground.



(a) Relative density (D_r = 70%)

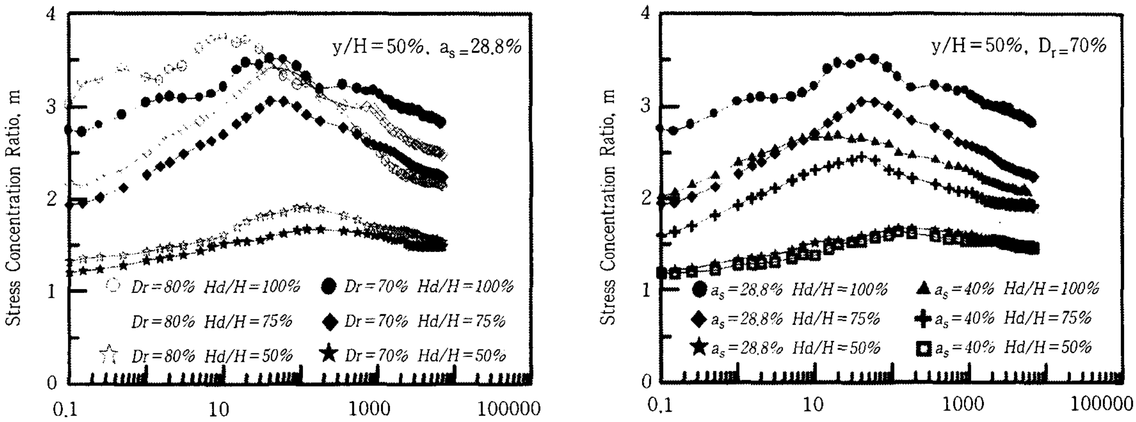
(b) Relative density (a_s = 28.8%)

Fig. 8 Distribution of normalized pore pressure versus normalized depth(U = 70%)

5.3 Stress Concentration Ratio

Fig. 9(a) and (b) show a change of the stress concentration ratio with the elapsed time at the depth $y/H = 50\%$ for the improved ground with $a_s = 28.8\%$ and $D_r = 70\%$. The stress concentration ratio in the initial part of consolidation is low in the improved ground because the total stress of clay ground was high due to the higher pore water pressure at the early stage of consolidation. As shown in these figures, the stress concentration ratio increases in the early stage of consolidation and has a peak during the process of consolidation, and then decreases afterward. It is believed that the concentrated stress on the SCPs during the early stage of consolidation is distributed to the clay ground during the process of consolidation. The final stress concentration ratio for the fully penetrated SCP ground($m=2\sim3$) is larger than that of the partly penetrated SCP ground($m=1.5\sim2$) at the middle of the model ground. The final stress concentration ratio of the fully penetrated SCP ground was $m=3$ and $m=2.5$ in the relative density of $D_r=70\%$ and 80%, respectively. This indicates that the distribution of the stress is larger with the high relative density. The stress concentration ratios in the partly penetrated SCP ground($H_d/H=50\%$) of as$=28.8\%$ and $a_s=40\%$ showed the same behavior($m=1.2\sim1.5$) during the consolidation process at the middle of the model ground. However, in the fully and highly penetrated SCP ground($H_d/H=100\%$ and $H_d/H=75\%$), the stress concentration ratio was high with the increase of replacement area ratio.

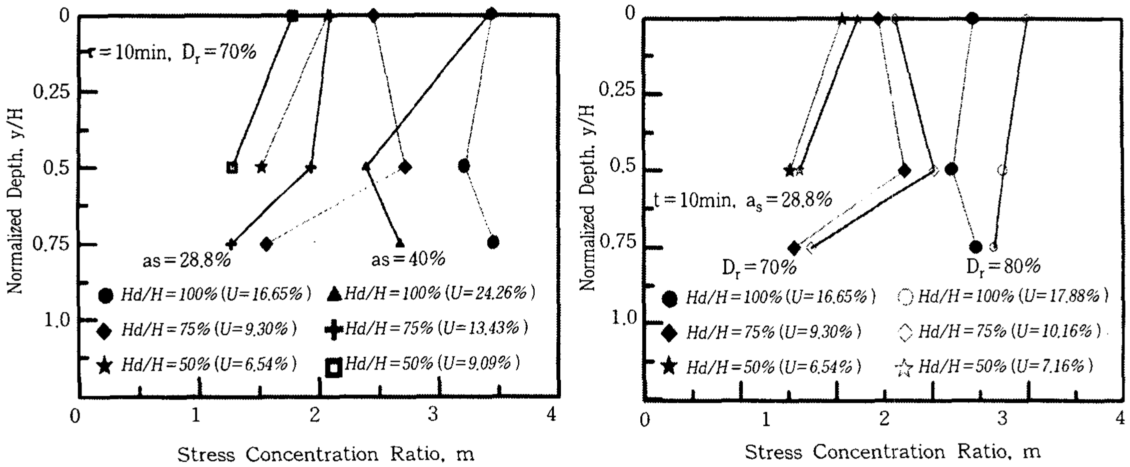
Fig. 10 and Fig. 11 show the relationship between the normalized depth and the stress concentration ratio in order to compare the effect of the replacement area ratio and relative density with the consolidation. The elapsed time of 10min is the time when the stress concentration ratio shows almost peak. The elapsed time of 900min is the time that the pore water pressure almost dissipated during the process of consolidation of the SCPs ground. From Fig. 10, the stress concentration ratio shows the same form and varies with the depth in spite of the difference of the relative density and the replacement area ratio. The stress concentration ratio is larger with the high relative density and the low replacement area ratio. However, in the elapsed time of 900min, the stress concentration ratio decreases with the increase of the relative density. This shows that the vertical stress in the SCPs is obviously distributed at the high relative density through the process of consolidation as shown in Fig. 11. However, the trend of the stress concentration ratio does not change during the process of consolidation. As shown in those figures, not only the stress concentration ratio varies with depth, but also the penetration ratio and the replacement area ratio.



(a) $D_r=70\%$ and $D_r=80\%$ on replacement area ratio($a_s=28.8\%$)

(b) $a_s=28.8\%$ and $a_s=40\%$ on relative density($D_r=70\%$)

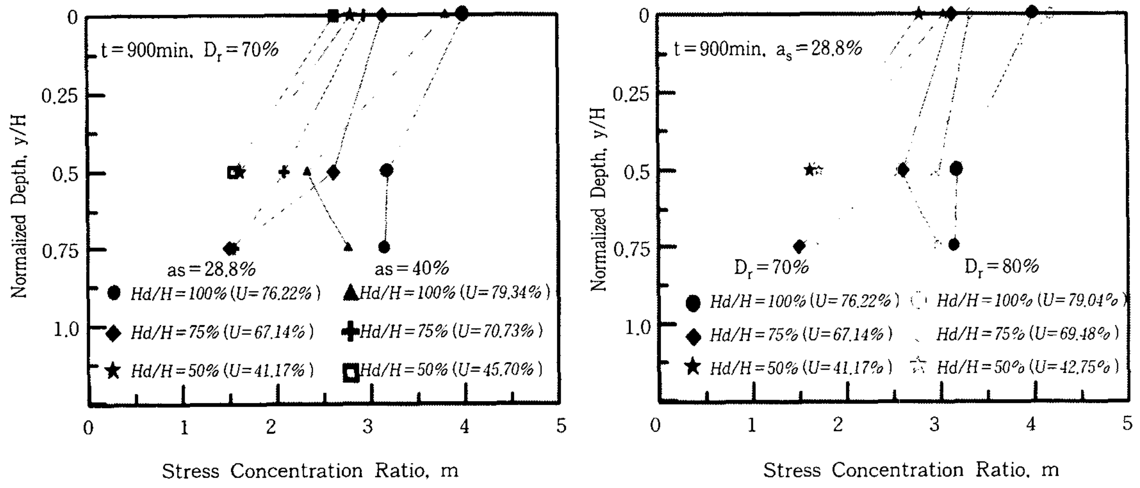
Fig. 9 Distribution of stress concentration ratio versus time of replacement area ratio and relative density



(a) Relative density(D_r = 70%)

(b) Replacement area ratio(a_s = 28.8%)

Fig. 10 Relationship between normalized depth versus stress concentration ratio(t = 10min)



(a) Relative density(D_r = 70%)

(b) Replacement area ratio(a_s = 28.8%)

Fig. 11 Relationship between normalized depth versus stress concentration ratio(t = 900min)

6. Concluding Remarks

The consolidation behavior of the partly and fully penetrated SCP ground was studied by the 1-G model tests. The main results of this paper can be summarized as follows:

1. The consolidation settlement of the improved ground was found to increase with the decrease of the penetration ratio of the SCPs.

2. The process of consolidation for the fully penetrated SCP ground moves rapidly than the partly penetrated SCP ground and non-treated ground.
3. The stress concentration ratio obtained from the measured stresses of the SCP and clay varied during the process of consolidation. The stress concentration ratio measured at the ground surface increased during the process of consolidation and reached $m=4.5$ for $H_d/H=100\%$, $m=3.5$ for $H_d/H=75\%$ and $m=3.0$ for $H_d/H=50\%$. On the other hand, the stress concentration ratio measured at the bottom of the SCPs increased, had a peak and then decreased during the process of consolidation. Its final value was $m=2.5$ for $H_d/H=100\%$, $m=2.0$ for $H_d/H=75\%$ and $m=1.5$ for $H_d/H=50\%$.
4. The measured stress concentration ratio varied with the depth and was larger in the upper layer than in the lower layer. This trend increased with a decrease of the penetration ratio of the SCPs.
5. The measured stress concentration ratio of the partly penetrated SCPs ground was larger in the case of the high relative density of the SCPs than in the case of the low one. However, the measured stress concentration ratio of the fully penetrated SCPs ground had the same trend as that of the partly penetrated SCPs ground in the early stage of consolidation. On the other hand, this value becomes to be larger in the case of the low relative density of the SCPs than in the case of the high one.
6. The influence of the replacement area ratio on the stress concentration ratio appeared when the penetration ratio of the SCPs was larger than $H_d/H=75\%$, furthermore the stress concentration ratio increased with a decrease of the replacement area ratio.
7. The stress concentration ratios calculated from the amount of the final settlement were as $m=7\sim9$ for $H_d/H=100\%$, m $=5\sim6$ for $H_d/H=75\%$ and $m=2\sim4$ for $H_d/H=50\%$. These values are larger than those obtained from the measured stress concentration ratio.
8. The dissipation of the pore water pressure at the depth of the bottom of the SCPs was delayed by the effect of the non-treated layer below the bottom of the SCPs. This trend increased with a decrease of the penetration ratio of the SCPs.

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