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STRENGTH CHANGES OF SURROUNDING CLAY DUE TO SOIL-CEMENT COLUMN INSTALLATION

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SYNOPSIS: This paper discusses the reduction and subsequent recovery and increase of shear strength of clay in the vicinity of soil-cement column. Laboratory and field tests were conducted to investigate the effects on surrounding clay during and after soil-cement column installation in soft Ariake clay. Discussions were made on the mechanism of strength changes of clay by considering the thixotropic recovery, reconsolidation effect, penetration of cement slurry and diffusion of exchangeable cations. On the basis of field and laboratory observations, 10 days after column installation, the decreased shear strength of surrounding clay during mixing was recovered and 30 days later, shear strength of surrounding clay increased 30% by average. Therefore, it is recommended that the increase of shear strength of clay can be taken into consideration in the bearing capacity and stability analysis of the composite ground.

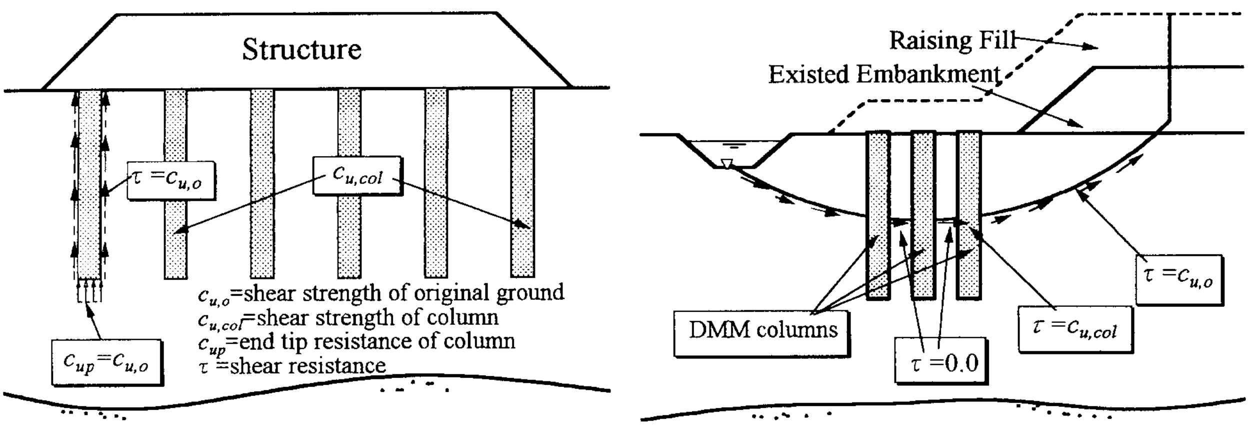
KEYWORDS: soil-cement column, sensitive clay, shear strength, thixotropy, laboratory and field tests, skin resistance, excess pore pressure, ion diffusion

1. INTRODUCTION

Deep mixing method (DMM) is one of the most commonly used deep soft ground improvement techniques in Japan. DMM column is manufactured by mixing the improvement chemical agents (slurry or powder of cement or lime) with the *in-situ* soft soil by the rotating mixing wings. The acquired strength of the column is due to the chemical reaction between cement (or lime) and soil particles (Hydration, pozzolanic cementation, ion exchange, flocculation, and carbonation). Hitherto, because of the high strength of hardened column and high improvement area ratio in practical engineering, most of the discussions and analyses on the strength increase of composite ground by this improvement technique were interpreted as due to the increase of the strength of the column itself[1)~6)] in the literature. Because the strength of the clay in the vicinity of DMM column will probably be decreased in sensitive clay during the mixing, one of the design considerations is that in stability analysis of column improved soft ground, the strength of surrounding clay is not considered[2)6)], and in bearing capacity analysis of DMM column improved composite ground, the strength of surrounding clay is taken as that of original ground[3)6)].

Estimates of the skin resistance capacity of floating type vertical columnar inclusions such as pile, sand compaction pile

(SCP), stone column and soil-cement (lime) column in clay ground were generally based on the undrained shear strength of clay *in-situ*, that is, a factor α was introduced to evaluate the skin resistance of piles and/or columns by use of $\tau = \alpha c_{u,o}$. The value of α was determined by considering the geotechnical condition and type of pile. In Ariake clay, the α value varies from 0.5~1.2 by laboratory and field tests[7] of different type of piles installed. In fact, during construction of displacement columnar inclusions such as SCP, stone column or pile, due to an expanding pressure existed on the surrounding clay, there will be a rapid increase in excess pore pressure in clay, which causes consolidation of surrounding clay. As a result, the increase of undrained shear strength of clay and, in addition, the increase of confining pressure and effective stress for the columnar inclusions can be expected. Randolph et al.[8)9)] (1979) based on the theory of expansion of cylindrical cavity investigated increase of strength of surrounding clay after pile driving. Asaoka et al.[10)] (1994) discussed strength increase due to SCP installation.



(a)Bearing capacity problem (b)Stability of embankment

Fig.1 Design approaches of composite ground improved with columns by deep mixing method (DMM)

In the case of the installation of soil-cement or lime column, from the aforementioned studies, it is possible that the strength of surrounding clay is affected. One of the effects is the strength reduction during mixing. Furthermore, after construction, the strength will increase attributed to the reconsolidation with the dissipation of excess pore pressure induced by installing of columns and/or recementation with the diffusion of cations from soil-cement or lime column[11)]. Because the study on this aspect is generally not covered in the literature, the convenient countermeasure in practical design is ignoring the strength of surrounding clay. However, the strength of the surrounding clay needs to be accounted for the design of composite ground improved by floating type columns. There are two aspects on the column improved composite ground; as shown in **Fig.1**, the first is on the shaft capacity of the column (**Fig.1(a)**α=1) and the other is on the stability by slip circle analysis (**Fig.1(b)**). The main objective of the present study is to investigate this problem to clarify the bearing capacity mechanism of floating type column and offer a more rational design approach for composite ground based on a field full scale test and two laboratory model tests.

2. TEST PROGRAMS

2.1 Laboratory model tests

2.1.1 Equipment for making the model column

The equipment used for making model columns with the diameter of 10cm or 15cm in laboratory is a small-sized soil-cement mixing device manufactured by Tenox Co. Ltd. with the same working principle as the common soil-cement (or lime) column installation machine. It includes three main parts: mixing device; slurry injection pump; controlling panel (as shown in **Fig.2**). For detailed specifications of the device please refer to the literature[12].

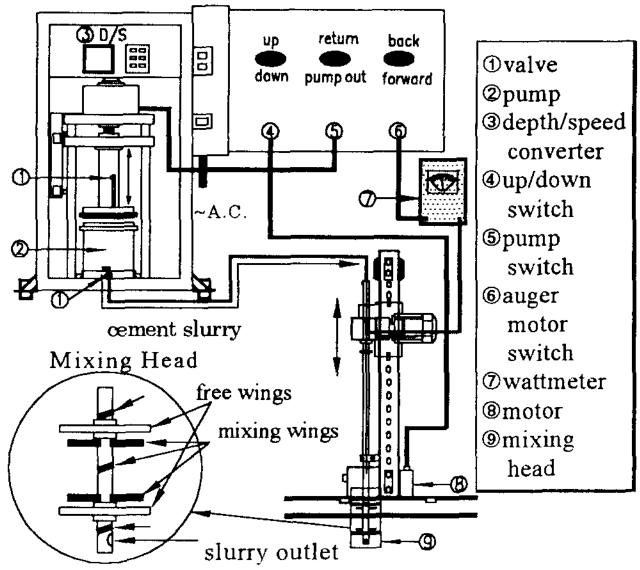
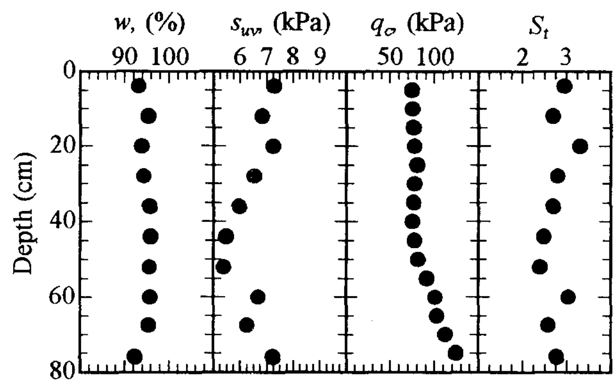


Fig.2 Equipment for making the model column

2.1.2 Test in soil mould

A completely remoulded Ariake clay (initial properties: specific gravity ρ_s=2.61, water content w_n=130~140%, liquid limit w_L=100%, plastic index I_p=57, initial electric-conductivity Ec=3.4~4.2ms/cm, pH=8.4~9.0, grain size distribution: clay 65%, silt 31%, fine sand 3.5%, and sand 0.5%) was set in a cylinder mould which has an inner diameter of 49cm and height of 70cm. Geotextiles were set at the bottom and top of the clay to let drainage and the clay was consolidated under a pressure of 10kPa for 15 days. Then, a miniature column (diameter of 10 cm and height of 20 cm) was manufactured by mixing the cement slurry (mixed amount of cement 300kg/cm^3, water-cement (W/C) ratio 80%) in middle of the reconstituted clay inside the mould by the model column manufacturing device. The feed rate was 0.25m/min during mixing down and the retrieval rate was 0.5m/min while mixing up and the rotating speed was 30rpm. After installation, the column was cured for 7 days. Then, tests were conducted to obtain the physical, chemical and mechanical properties of surrounding clay.



w-water content; s_{uv}-laboratory vane shear strength; q_c-cone resistance; S_t-sensitivity ratio

Fig.3 Soil properties after reconstitution in soil tank

2.1.3 Tests in soil tank

A concrete soil tank (with plan dimensions of 1.5m×1.5m and height of 1.0m) was filled with completely remoulded Ariake clay with the initial condition of the same clay as that in the mould test. Then, a 15kPa consolidation pressure was applied on the soil and consolidated for one year (the drainage condition was vertical drainage by putting the geotextiles at the top and bottom). Silicon grease was put on the tank wall to reduce the friction of the concrete wall during consolidation. After consolidation, the thickness of model ground was 82cm and the geotechnical properties are shown

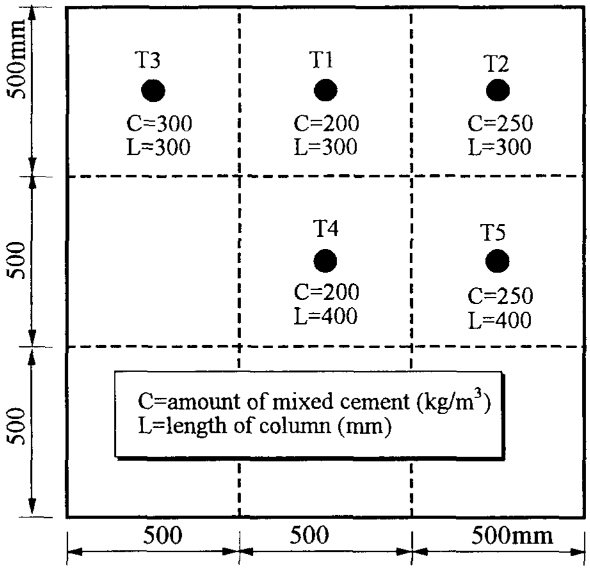
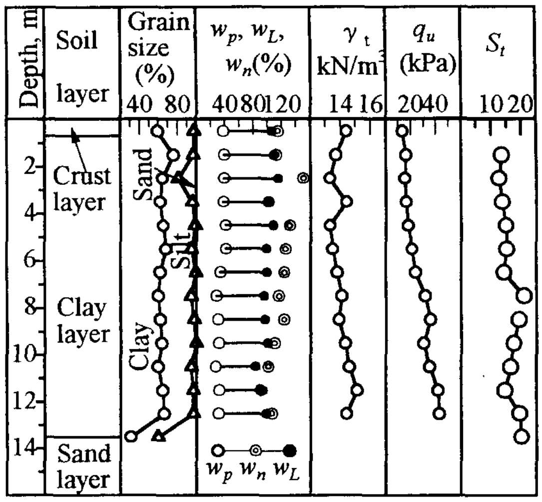


Fig.4 Model soil-cement columns installed in soil tank



w_p-plastic limit, w_L-liquid limit, w_n-water content, γ_t-unit weight, q_u-unconfined compressive strength, S_t-sensitivity ratio

Fig.5 Geotechnical profiles and soil properties *in-situ*

Table 1 Cation concentration, electric-conductivity (*Ec*) and pH in pore water of original ground in the field test (at Ashikari)

Depth (m)	Na^+ (meq/l)	K^+ (meq/l)	Ca^{2+} (meq/l)	Mg^{2+} (meq/l)	*Ec* (ms/cm)	pH
1~2	45.3	1.7	0.67	4.9	3.2	7.85
2~3	94.8	6.2	1.04	7.2	11.2	8.11
3~4	135.1	3.6	1.31	5.7	15.3	7.92
4~5	203.7	4.6	1.45	7.9	16.3	7.60
5~6	177.3	5.5	3.01	9.1	14.9	8.24
6~7	134.8	7.8	2.11	8.8	13.0	8.02
7~8	112.7	6.1	2.95	7.7	10.8	8.18

in **Fig.3**. The sensitivity after reconstitution is not high, ranging from 2 to 3.

A total of 5 columns (T1~T5) were installed by the same device as introduced above and the number, amount of mixed cement and height of columns are shown in **Fig.4**. The water-cement ratio of cement slurry was 80% and the diameters of columns were 10 cm. Afterwards, the columns were cured for 28 days.

2.2 Field construction test

In case of high sensitive soft clay ground such as Ariake clay, there exists a so-called thixotropic phenomenon of surrounding clay including strength reduction due to disturbance during mixing and strength recovery after construction. In laboratory test, because the clay specimen is a reconstituted clay by consolidating the remoulded soft clay, the sensitivity of the model ground is not high and the thixotropic phenomenon is not obvious. In the field test, much more attention is paid on the thixotropic phenomenon.

2.2.1 *In-situ* geotechnical profiles and soil properties

Figure 5 shows the soil profile and the soil properties of clay layers at Ashikari, Saga Prefecture, Japan. The top layer

(0.0 to 0.6m depth) is a crust layer of unsaturated clay formed by weathering with higher strength and low void ratio. The underlying from 1m to 13.5m depth, is the Ariake clay layer with grey colour typically having many shells and some silt content especially at 2 to 4m depth. Below the Ariake clay layer is a sand layer. The sensitivity of *in-situ* Ariake clay at Ashikari is very high, ranging from 10 to 25.

Table 1 tabulates the concentration of main cations (Na^+, K^+, Mg^{2+}, Ca^{2+}) dissolved in pore water, electric-conductivity (*Ec*), and pH of pore water *in-situ*. As seen in **Table 1**, the concentration of cations *in-situ* distributed with depth; in the upper (1 to 3m depth) and lower (5 to 8m depth) layers, is lower than that of 3 to 6m depth. The sodium ion concentration in Ashikari is much higher than that of the offshore place ($Na^+ \simeq 10$meq/L). This is because the construction site is near the estuary of Rokkaku river to Ariake sea, and the clay was strongly influenced by sea water during it sedimentation. The distribution of *Ec* with depth is similar to the cations and is approximately half compared to the sea water sample taken from the estuary. The pH is slight variation with depth and not much higher than that of the sea water sample from estuary. The chemical properties of the sea water sample from the estuary consisted of the followings: concentration of main cations Na^+=304.3meq/l, K^+=10.2meq/l, Mg^{2+}=26.2meq/l, Ca^{2+}=1.9~4.2meq/l; pH=7.7; electric-conductivity: *Ec*=25.4ms/cm.

2.2.2 Column construction machine

The soil-cement column mixing machine used in the field test construction was a newly developed low pressure slurry mixing machine called Slurry Double Mixing (SDM) method made by Shinwa Techno Company [13]. As shown in **Fig.6,** this machine is composed of driving and controlling part, slurry injection part and mixing head. The mixing head includes two mixing wings spacing 30cm and rotating in opposite direction. The injection pressure of slurry ranged from 50 to 150kPa. The characteristics of this machine are as follows: improvement depth=8m, diameter of mixing wing=0.6 to 1.4m, rotating speed=60 to 80rpm, maximum production rate 3.6m/min. In order to ensure the verticality of the column, vertical sensors are used. For detailed specifications of the machine refer to literature[13].

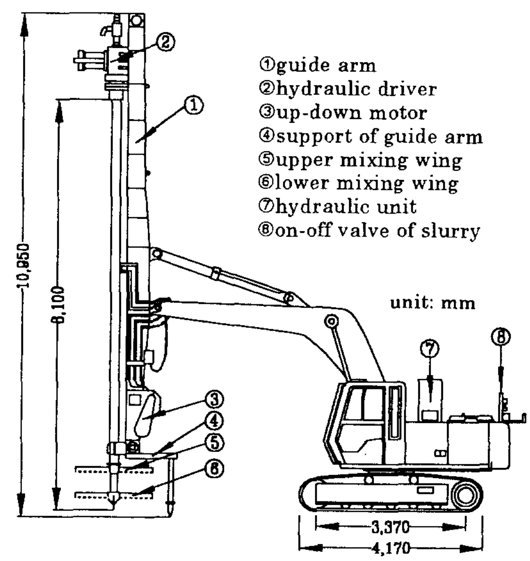


Fig.6 Slurry double mixing (SDM) machine

2.2.3 Construction test procedure

The field construction test was done in July, 1996. The soil-cement columns were installed at Ashikari inside the river bank near the estuary of Rokkaku river to Ariake sea. The soil-cement columns with a diameter of 1.14m were installed in a triangular pattern at 1.5m centre to centre space down to 7.0m depth. The improvement chemical agent was cement slurry with the water-cement ratio of 100% and cement amount of 140 kg/m^3. The cement is ordinary Portland cement, and its chemical ingredients are CaO=60.2%, MgO=1.5%, SiO_2=19.3, Al_2O_3=4.4%, Fe_2O_3=2.5%, SO_3=7.4%, and 4.7% of other ingredients. The water used in making the cement-slurry was from a creek near the site. The chemical properties of the creek water were Na^+=8.2meq/l, Ca^{2+}=2.9meq/l, Mg^{2+}=3.2meq/l, K^+=1.4meq/l, pH=7.4, *Ec*=1.5ms/ cm. A total of 12 columns had been subsequently installed in 310 minutes. The improvement ratio in area was 50%. The installation speed for column numbers 1 to 8 was 1 m/min and that of column numbers 9 to 12 was 0.71 m/min. The plan and section views of the test site are shown in **Fig. 7**, and the appearances of columns after hardening are shown in **Photo 1**.

The variations of pore water pressure and cone resistance were tested before, during and after column installation. Furthermore, samples were taken before and after column installation to investigate the changes of physical, chemical, and mechanical (laboratory vane shear and unconfined compressive strength) properties. Four piezometers had been set in the middle of three columns with 33cm to column surface and down to 1.5m, 3.5m, 5.5m and 7.0m depths, respectively and a dummy piezometer was set at a little far distance (about 2.8m, approximately 4.85 times of column radius, R_c). The cone resistance test and samples were taken in three different locations, as shown in **Fig.** 7(**a**), with a distance to the column surface of 6.5cm(A), 20cm(B) and 33cm(C), respectively.

3. RESULTS OF THE TEST STUDIES

3.1 Results from laboratory tests

In order to investigate the property changes of surrounding clay due to column installation, tests had been taken on the measurement of the changes of water content, pH, electric-conductivity, and cone resistance in surrounding clay after curing. The tested results are presented as follows:

Figure 8 gives the variations of water content (w_n), pH value, electric conductivity (Ec), and cone resistance ratio ($S_{Rq}(t) = q_{c,r}(t)/q_{c,o}$) of surrounding clay with a radial distance ratio from the centre of column after column installation. In **Fig. 8**, $q_{c,r}(t)$ is the cone resistance index in the range of r/R_c=1 to 3 after t days curing; $q_{c,o}$ is the cone resistance index out of the influential region. As shown in **Fig. 8**, after curing, within a distance of twice the column

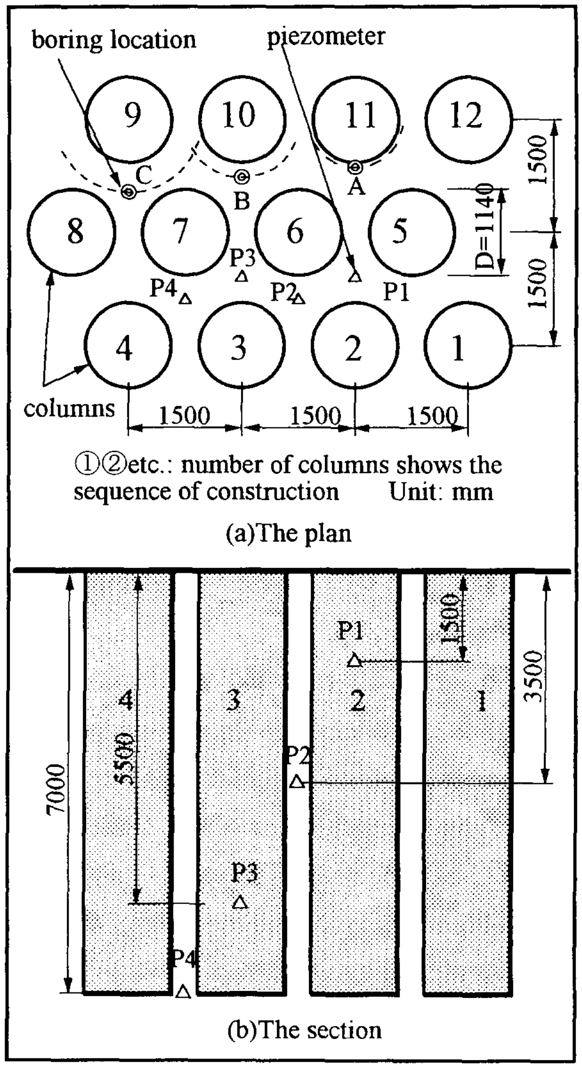


Fig.7 Layout of columns and instrumentation *in-situ*

Photo 1 Soil-cement columns after construction

radius in surrounding clay, the water content decreased about 5 to 15%, pH value was increased, *Ec* was increased, and $S_{Rq}(t)$ was increased. Furthermore, the nearer the measured points to the column surface, the greater the measured changes. Within a very small distance from the column surface (r/R_c=1.05), the measured values were almost same as that in column. Thus, it is indicated that the surrounding clay was strongly affected by columns.

3.2 Results from field test

3.2.1 Dissipation of excess pore pressures

Figure 9 shows the variation of excess pore pressure at

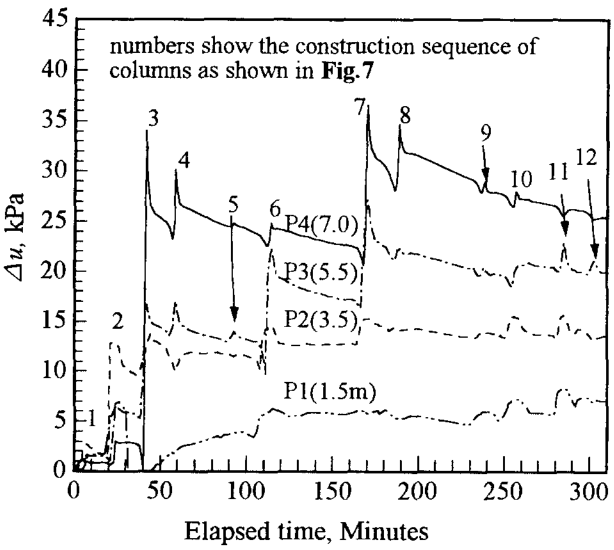


Fig.9 Excess pore pressures measured during column installation in the field test

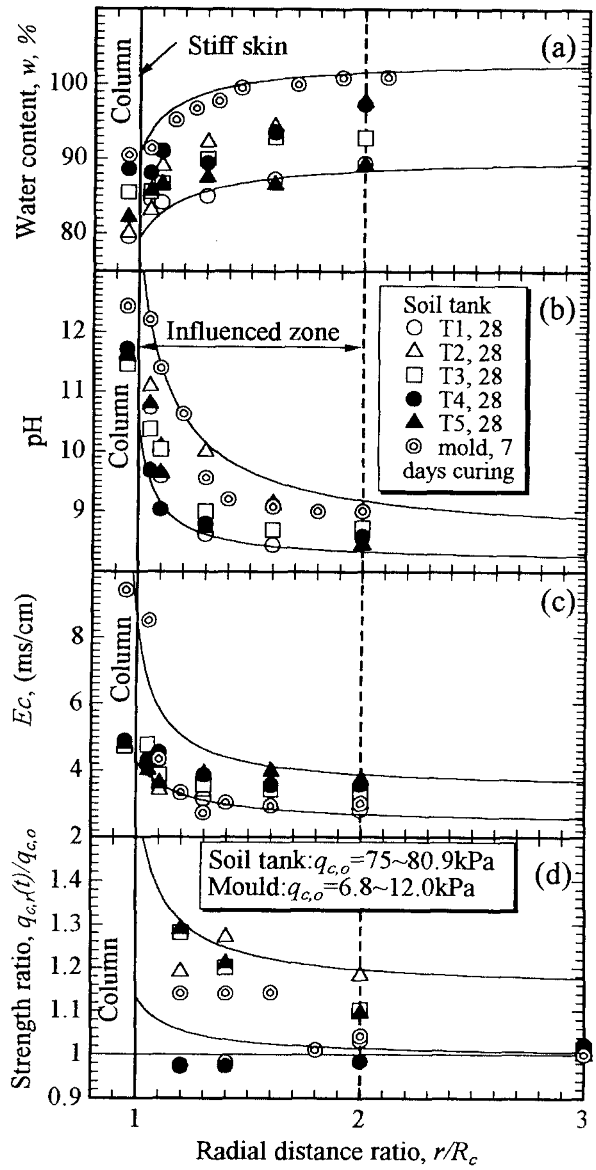


Fig.8 Variation of soil properties of clay after column installation in the laboratory tests

different depths during a total of 12 soil-cement column installation. During the cement column construction, the measured excess pore pressures were gradually increasing and reaching maximum when the mixing head arrived at the depth of piezometer. The maximum excess pore pressure during column installation was 36.6kPa at 7 m depth, and it was greater than the effective overburden stress as shown in **Fig. 10**. The dissipation of excess pore pressure in the initial 5 minutes was very fast amounting to 20 to 30% of maximum excess pore pressure and as shown in **Fig. 9**, in which spike points can be observed in the Δu-t (time) curve.

Figure 11 shows the dissipation of excess pore pressure after construction. The rate of dissipation varied with the depth being faster at the top and the bottom of the column than at the middle. After 8 days, only 10% of the maximum excess pore pressures remained and after 16 days, the excess pore pressure completely dissipated.

3.2.2 Variation of physical and chemical properties

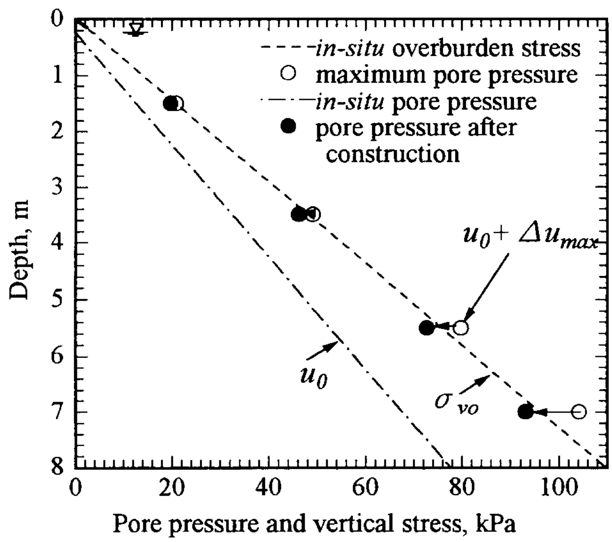


Fig.10 Variation of pore pressure and the relationship with *in-situ* stress during construction

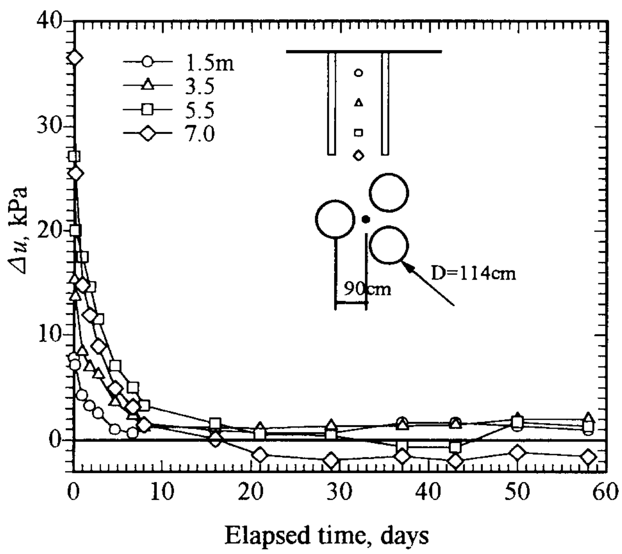


Fig.11 Dissipation of excess pore pressure after construction

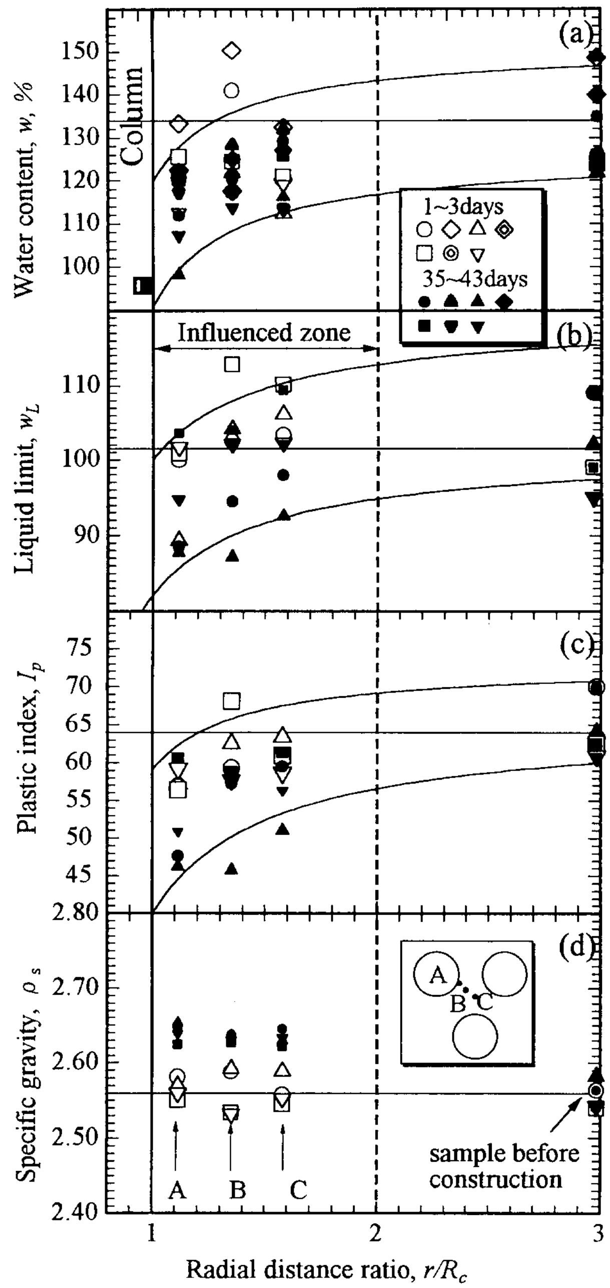


Fig.12 Variation of physical properties after construction in the field test

Figure 12 depicts the measured value of physical properties at 1 to 3 days and 35 to 43 days after construction. At 1 to 3 days after construction, there were no significant changes of physical properties. However, one month after construction, the water content decreased, ranging from 5 to 20% and specific gravity increased. The liquid limit and plastic index slightly decreased. Moreover, the variations of physical properties in radial direction reached their stabilized values at radial distance, $r \geq 2R_c$ (R_c-the radius of column).

Figure 13 indicates the variations of the ratios of concentrations of cations (Na^+, K^+, Mg^{2+}, Ca^{2+}) dissolved in pore water in the surrounding clay after and before column construction. As shown in the figure, from 1 to 3 days after column installation, there were no significant changes of cation concentration. However, one month later, divalent cations increased greatly, i.e. Ca^{2+} increased from 8 to 20 times, Mg^{2+} from 3 to 6 times; monovalent cations also increased (K^+

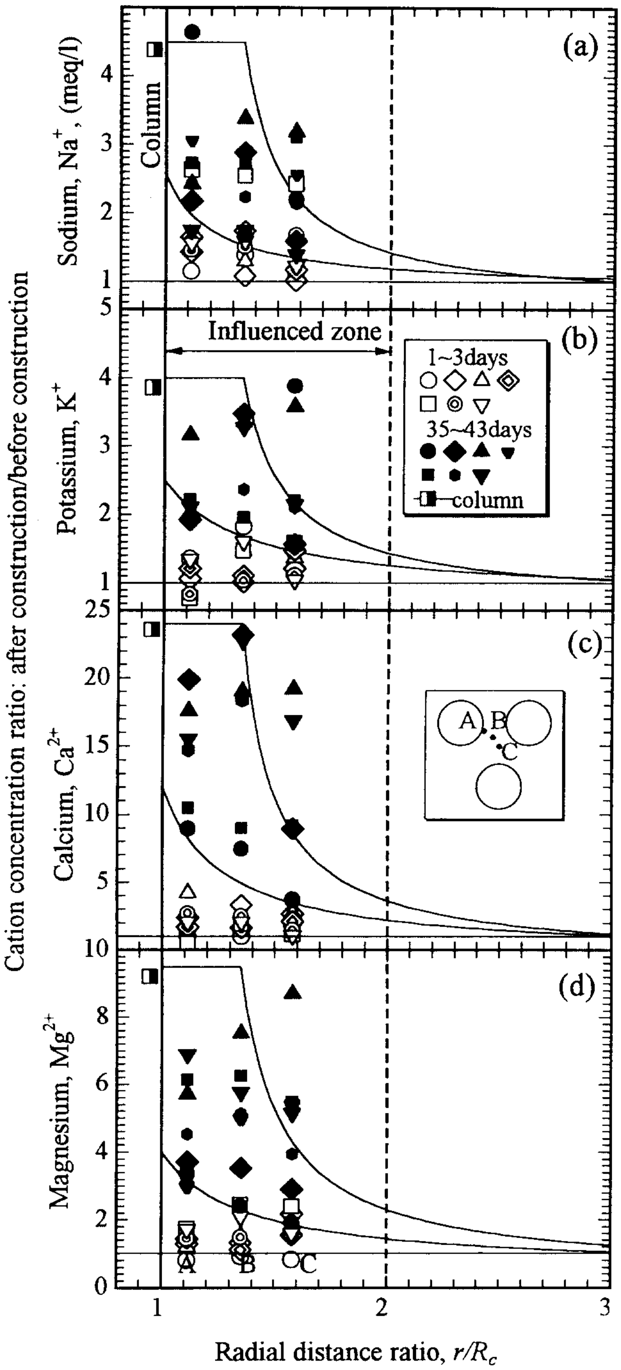


Fig.13 Ratio of cation concentration after and bebore construction in the field test

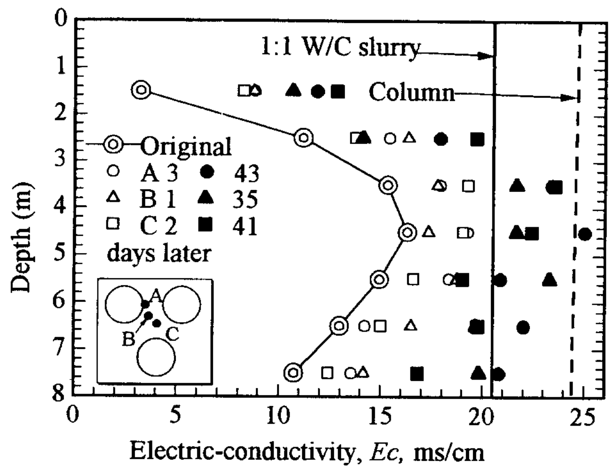


Fig.14 Varition of electric-conductivity after construction in the field test

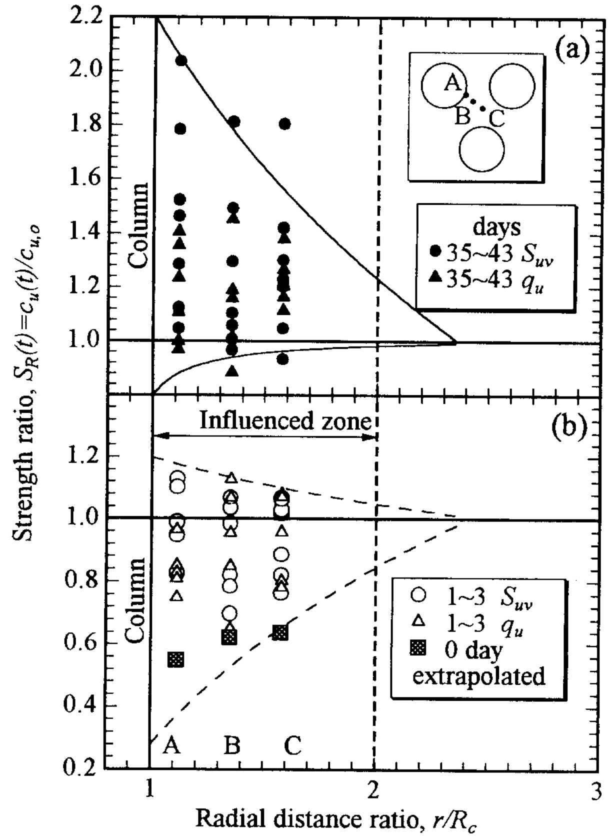


Fig.15 Variation of strength after construction in the field test

3 times, Na^+ from 2 to 3 times). **Figure 14** plots variation of electric-conductivity (*Ec*) in surrounding clay, column and 100% W/C slurry. After construction, *Ec* was increased with time such that one month later, *Ec* increased 3 to 5 times. The increase of electrolyte concentration could speed up the thixotropic hardening of clay[14)15)].

3.2.3 Shear strength reduction, and subsequent recovery and increase after construction

Figure 15 plots the tested shear strength ratio ($S_R(t)=c_u(t)/c_{u,o}$; where $c_u(t)$=shear strength after construction, $c_{u,o}$=shear strength of intact ground before construction) which varied with radius after construction. The strength had been tested mainly by laboratory vane shear test (S_{uv}), and the clay strength were also tested by the unconfined compressive test (q_u)

and *in-situ* cone resistance test (q_c) for checking. The reduction of strength immediately after column installation was about 40% from the extrapolated results and recovery of strength occurred immediately after column installation. The strength ratio at 1 to 3 days after construction ranged from 0.6 to 1.08. After 30 days, the strength ratio ranged from 0.95 to 2.04 with an average of 1.3. After 60 days, the strength ratio continuously increased, ranging from 0.99 to 2.17 with averaged value of 1.33. From this test result, it is confirmed that the strength of surrounding clay recovered and increased after it was disturbed during mixing.

4.STRENGTH CHANGES OF THE CLAY AND THE INFLUENCIAL FACTORS

In the light of the test results, it is shown that the changes of the physical, chemical and mechanical properties occured in surrounding clay after column installation. There exists a so called stiff skin (or hardened zone) very close to column with radial distance of $r \cong 1.05R_c$. It has been confirmed that there is no obvious interface between column and soil, or in other words, it is integrated between soil and column. This zone may be formed due to part of the cement-slurry forced out by the wings during mixing. Strength changes are also found in the region outside hardened zone until radial distance, $r \cong 2R_c$. Three related factors on the strength changes may be inferred as follows: the first is temporary reduction due to mixing and recovery of strength (thixotropic phenomenon); the second is the consolidation of the clay with the dissipation of excess pore pressures induced by the mixing and injection of slurry; the third is the effect of the diffusion of the improvement agents and/or ions. The details of the three factors are discussed in following sections.

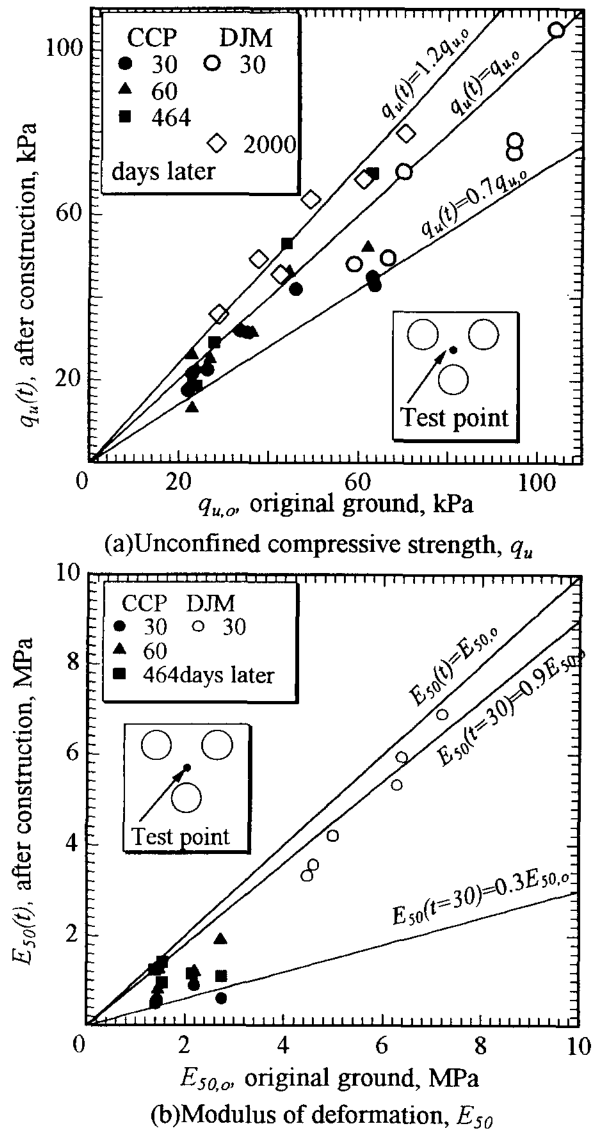


Fig.16 Recovery of clay strength properties in the vicinity of after CCP and DJM columns

4.1 Strength changes due to thixotropy

The degree of disturbance to surrounding clay is different from the mixing methods. **Figure 16** depicts the results from another field test near the test site of this study. As shown in the figure, the degrees of strength decrease and recovery induced by the CCP (Chemical Churning Pile-one of the high pressure mixing method) and DJM (Dry Jet Mixing) are much different. There is no obvious different from the results of unconfined compressive strength of CCP and DJM. After 60 days, the strength disturbed by the two methods recovered to that of original value. However, the recovery of the modulus of deformation E_{50} are much different.

For DJM method, 30 days after construction, E_{50} returned to the original value. For CCP, the corresponding E_{50} value was only 30% of the original value. Even one year later, E_{50} still did not return to the original value. Thus, there exists differences of strength parameter recovery by different mixing methods, which is very significant in the deformation analysis of composite ground.

4.2 Consolidation accompanied with dissipation of excess pore pressures

The dissipation of induced excess pore pressures in surrounding clay in a very short period immediately after column installation is plotted in **Fig. 9**. Massarch and Broms[16] (1977), and Massarch[17] (1978) had analyzed excess pore pressures induced and its rapid dissipation in the initial stage by use of the model of expansion of cylindrical cavity during pile driving and pointed out that during pile driving, hydraulic fracturing occurred in surrounding clay. Consequently, the excess pore pressure dissipated through the fracturing cracks. For soil-cement column installation, because there exists an expanding pressure due to the injection of slurry, the soil could be displaced laterally and large heave near the ground surface can occur[2)6)]. As shown in **Figs.9** and **10,** the measured maximum excess pore pressures are greater than overburden effective stresses. Hence, the effective stresses in the vicinity of column can be negative such that hydraulic fracturing can occur in a region adjacent to the column. Consequently, the surrounding clay will be consolidated with the dissipation of the induced excess pore pressures.

4.3 Effects of the penetration of improvement agents and diffusion of ions

On the one hand, immediately after the column installation, owing to the difference of ion concentration and pore pressure between the column and the surrounding clay, ions will diffuse to surrounding ground. Moreover, there are fracturing cracks in the vicinity of columns so that the cement slurry can conveniently move into the fracturing cracks under the expanding force during mixing.

The increase of electrolyte concentration will disrupt the existing clay particle-water-electrolyte equilibrium system. The excess cations will be attracted to the negatively charged clay surface to equalize the concentrations through the followings mechanisms[14)]: hydrogen bonding, cation hydration, attraction by osmosis, dipole orientation in an electric field, attraction by London dispersion forces. Subsequently, more cations are adsorbed into the diffuse double layer of clay particle and cations may react with the released alumina $[Al(OH)_6]^{3-}$ and/or silica $[SiO_4]^{4-}$ from clay minerals resulting in aggregation and improve engineering behaviour of the clay.

Rajasekaran and Rao(1997) had done a laboratory test to investigate the changes of geotechnical properties of surrounding clay by installing lime column and by point injection of lime. The test results showed that in the surrounding clay with a radial distance ranging from 2 to 4 times of column radius, the strength increased about 4 to 8 times of the original values and compressibility was reduced to about 1/3 to 1/4 of the original value.

5.INVESTIGATION ON THE STRENGTH CHANGES

5.1 Components of the strength changes

Figure 17 shows the variation of strength ratio of the surrounding clay after column installation. As indicated in the figure, the relationship between strength ratio and elapsed time can be expressed by the following expression:

$$S_R(t) = S_R(0) + \Delta S_R(1 - e^{-kt}) \qquad (1)$$

where:

$S_{R}(t)$: strength ratio at any t time after construction $=c_u(t)/c_{u,o}$

$c_u(t)$= shear strength at any t time after construction

$c_{u,o}$= original shear strength of intact clay before construction;

$S_R(0)$: strength ratio immediately (0 time) after column installation $=c_u(0)/c_{u,o}$;

ΔS_R: increment of strength ratio after very long period of time $\Delta S_R=(c_u(\infty)-c_u(0))/c_{u,o}=\Delta c_u/c_{u,o}$;

k = coefficient of strength recovering speed.

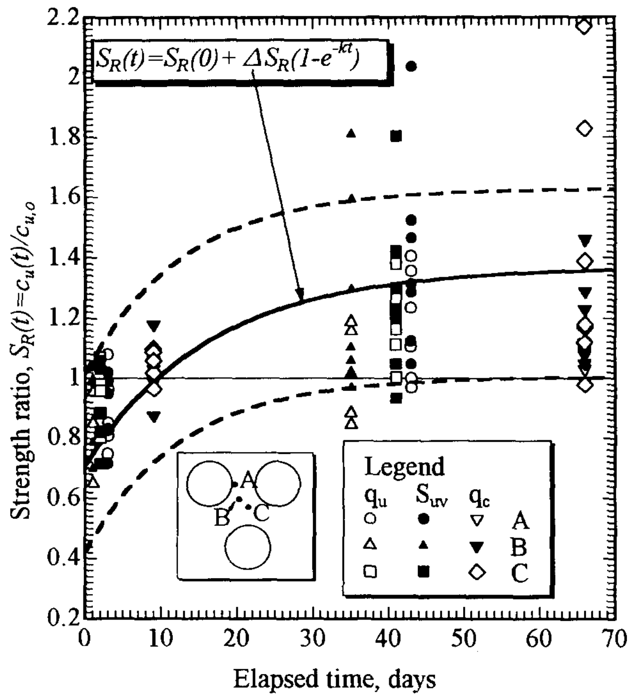


Fig.17 Variation of clay strength in the vicinity of SDM columns

The degree of disturbance due to column installation can be obtained as:

$$D_{dis} = \frac{c_{u,o} - c_u(0)}{c_{u,o}} \times 100\% = (1 - S_R(0)) \times 100\% \qquad (2)$$

Therefore, from the regression analysis, the degree of disturbance at locations, A, B, and C is 45.2%, 37.9%, and 36.3%, respectively. The increment of corresponding strength ratio after very long period of time is 0.851, 0.575, and 0.584, respectively. Therefore, the corresponding shear strength ratio ($S_R(\infty)=S_R(0)+\Delta S_R$) after very long period of time at A, B, and C is 1.39, 1.21, and 1.22, respectively.

The regained strength can be expressed as follows:

$$\Delta c_u = \Delta c_{u,recon} + \Delta c_{u,cement} \qquad (3)$$

where,

Δc_u: total increment of shear strength;

$\Delta c_{u,recon}$: strength increment due to the reconsolidation after dissipation of excess pore pressures.

$\Delta c_{u,recem}$: strength increment due to cation exchange and flocculation by the diffused cation as K^+, Na^+, Mg^{2+}, Ca^{2+} etc.

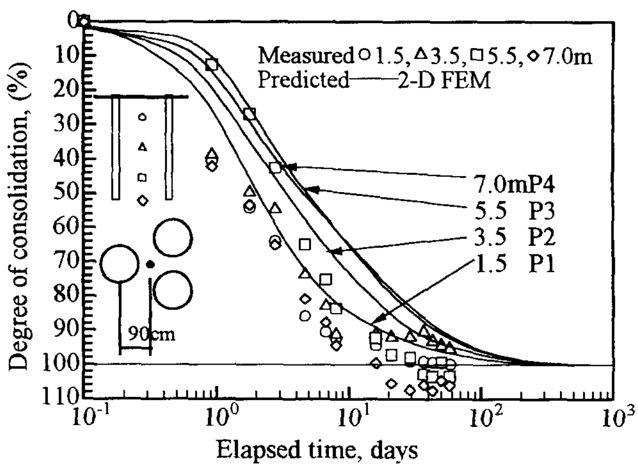


Fig.18 Measured and predicted degree of consolidation based on piezometer observation and FEM analysis in the field test

5.2 Water movement to column

Figure 18 indicates the measured and predicted degrees of consolidation with elapsed time due to dissipation of excess pore water pressures. The predicted values were calculated by 2-D FEM analysis under axi-symmetrical stress condition and in the analysis the column was taken as impermeable materials. Furthermore, the permeability of clay in the analysis were assumed by the following equation:

$$k_h = 2k_v = 8k_{v,Lab} \quad (4)$$

where

k_h = horizontal permeability of clay;

k_v = vertical permeability of clay;

$k_{v,Lab}$ = permeability parameter from laboratory standard consolidation test (as shown in **Table 2**).

Table 2 Permeability in FEM analysis

Depth (m)	Permeability, $k_{v,Lab}$, ($\times 10^{-4}$, m/day)
0~0.5	83.4
0.5~2.5	3.39
2.5~4.5	2.70
4.5~6.5	1.64
6.5~9.0	0.96
9.0~13.5	0.905
Note: k_h=2k_v=8$k_{v,Lab}$	

From **Fig.18**, the measured dissipation rates of excess pore pressure was much faster than the predicted values by FEM. This may be due to faster dissipation of excess pore pressure through the fracturing cracks induced by mixing as discussed in Section 4.(2).

In the later stages after the column installation (e.g. one day later), the pore water moved towards the column. **Figures 8(a)** and **12(a)** show the water content distribution after curing 7 and 28 days in laboratory test, and 1 to 3 days and 35 to 43 days in the field test. As shown in these figures, from 1 to 3 days after construction, water content begins to decrease in surrounding clay. After curing more than one week, water content decreased over 20% in the vicinity of column with a distance to 1.05R_c. At a distance of (1.05 to 2.0R_c), the water content decreased from 5 to 20%. Outside this region at distance of 2R_c, there are no significant water content changes. During the hydration and pozzolanic reaction in hardening of cement, great amount of water is required. Not only the water in column takes part in hydration but also water from surrounding clay is absorbed to the column. Furthermore, hydration and pozzolanic reaction generates heat which raised the temperature of column and surrounding clay[18]. High temperature can lead to drying, water content reduction and speed up the consolidation cementation[19].

5.3 Skin resistance capacity of column

From the results of investigation on the undrained shear strength of clay in the vicinity of soil-cement column, the presence of stiff skin and influenced zone around the column has been confirmed. In the stiff skin, the strength is almost the same as the column. In the influenced zone, the total strength ratio after 60 days ranged from 1.0 to 2.17 with average of 1.33. Therefore, in evaluating the skin resistance of floating type column, this behavior should be considered. **Figure 19** shows the measured bearing capacity of floating type of soil-cement column from field loading test[20] together with calculated bearing capacity from following equation[3)20)]:

$$q_{ac} = N_c c_{up} + \frac{\pi D L c_{uf}}{A_c} \quad (5)$$

where:

q_{ac} = calculated bearing capacity;

N_c = bearing capacity factor;

c_{up} = undrained shear strength of clay under column tip;

c_{uf} = undrained skin resistant cohesion, $c_{uf}=c_{u,o}$;

$c_{u,o}$= undrained shear strength *in-situ*;

D, A_c, L = diameter, design section area, and length of column.

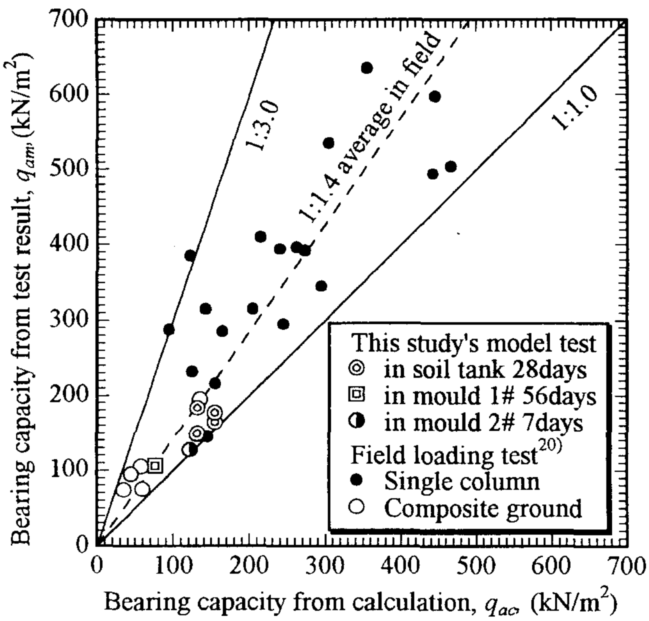


Fig.19 Comparison of measured and calculated bearing capacities of the soil-cement columns

As seen in this figure, all the tested bearing capacity is greater than the calculated values based on the undrained shear strength before column construction. From the laboratory model tests, q_{am}/q_{ac} (q_{am} measured bearing capacity from loading tests) varies from 1.05 to 1.37. The field test data in **Fig. 19** are from a total of 31 loading tests of which 23 tests were on the floating type columns installed by Teno-column installation machine at various ground conditions[20)]. The field test results show that the bearing capacity ratio is much greater than that in laboratory tests. Hence, it is confirmed that evaluation of shaft bearing capacity of floating type DMM column by use of the equation of $\tau_{max}=c_{uf}=\alpha c_{u,o}$ is enough to ensure safety.

6.CONCLUSIONS

In accordance with the experimental results and the subsequent discussions, the following conclusions can be drawn:

(1)The effects on surrounding clay by soil-cement column in high sensitive clay can be divided into two stages: at first, during construction there is strength reduction and recovery (thixotropy). Then, the strength gradually increased during consolidation (physical process) and cementation (chemical process).

(2)In the vicinity of the column, less than $1.05R_c$ a stiff skin is formed. Moreover, at distances ranging from 1.05 to $2.0R_c$, the shear strength is certainly greater than that of original intact ground influenced by the consolidation process and diffusion of improvement agents and/or ions.

(3)In the evaluation of shaft capacity, the conventional empirical equation $\tau_{max}=\alpha c_{u,o}$ from original undrained shear strength $c_{u,o}$ can be used. The coefficient of α is greater than unity, 10 days after construction. In the case of laboratory test, the α value ranged from 1.05 to 1.37, while in the case of field test, the corresponding value was from 1 to 3 with average of 1.42. Therefore, it is safe enough to set α=1 in calculation of shaft capacity of floating type DMM column.

(4)When soil-cement columns are used to improve the soft clay foundation of dike embankment, if the cement slurry mixing method is used, the reduced strength of the clay between columns can be recovered 10 days after column installation. Therefore, the strength of the clay between columns can be included in the stability analysis of embankment.

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