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A STUDY ON THE APPLICATION OF QUICK-LIME CONSOLIDATED BRIQUETTE PILES IN LOOSE SANDY SOILS

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ABSTRACT

During earthquakes, saturated loose sandy soils often liquefy, causing serious damage to buildings and underground structures. Various construction methods have been employed to stabilize these soils against liquefaction, the most popular being those which increase their density. Vibration and impact methods are commonly employed, but these are often a problem in urban areas. We studied the usage of a composite soil improvement material called Quick-Lime Consolidated Briquette. This material is placed to form cylinders, resulting in static consolidation of the surrounding soil due to absorption of soil water and consequent swelling. In our study, static consolidation increased density, and the swollen material became aggregate-like soon after placement, thus quickly stabilizing the surrounding soil. This report examines the preventive effect of quick-lime consolidated briquette piles against liquefaction as well as a method of determining optimum pile diameter and pitch.

Key words: compaction, earthquake, liquefaction, sandy soil, soft ground, soil stabilization (**IGC**: K5/K6/D9)

INTRODUCTION

In addition to subsurface compaction, several internal compaction techniques are conventionally used to stabilize deep layers of sandy soil. Of these techniques, vibro-composure and sand compaction, which utilize vibration and impact to reduce the void ratio of natural sandy soils and improve their relative density, are widely employed. However, this vibration and impact may disturb the living environment in urban areas.

Quick-lime Consolidated Briquette[1] (abbreviated QCB) is a composite soil stabilizer which reduces the water content of sandy ground by absorbing soil water, and causes the soil to swell and harden immediately after absorption. Soil voids are also reduced by the swelling pressure (the static consolidation effect) and the underground soil becomes a swollen aggregate-like material of relatively high density. Therefore, QCB is expected to in-

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crease the stability of sandy ground and thus prevent liquefaction. The effect of QCB is similar to that of vibro-composure and sand compaction.

The bearing capacity and settlement of sandy soil are closely related to its relative density. However, it is difficult to carry out a field study in which actual soil is tested because of problems involved with sampling and testing. Therefore, a standard penetration test which yields the N-value (*i.e.* the number of times a hammer hits the ground) and the relative density of the *in-situ* soil is usually employed to calculate the soil settlement and bearing capacity. The bearing capacity of sandy soil is generally obtained by introducing into a given formula the bearing capacity factor obtained after the internal friction angle of sand is calculated, using the N-value or relative density. The bearing capacity based on a maximum settlement of 2.5 cm is also calculated from the N-value. The bearing capacity can also be inferred from the formula for compressed volume, which incorporates the N-value, or by correlation between the elastic coefficient E_s ($=1/m_v$, m_v: coefficient of volume compressibility) and the N-value obtained in the test.

To improve the bearing capacity and the settlement characteristics of sand, it is necessary to determine the required increase in relative density or N-value, *i.e.* the increased density or the decreased void ratio caused by compaction. Thus it is important to study the relationship between the N-value and the void ratio. The relationship between the density of sand and its internal friction angle represents the relation between the initial relative density in a direct shear test or a triaxial compression test and the internal friction angle obtained from the shear resistance. However, in design, it is more effective to utilize the $N \sim e$ relation in which the N-value and the void ratio (e) respectively represent the internal friction angle and the relative density (D_r). If the $N \sim e$ relation is sufficiently accurate, it is utilized as a design parameter in improving loose sandy soil.

Since internal compaction cannot be monitored underground, information obtained in parallel tests is utilized to control the compaction. This procedure is very important in practical basic engineering. The improvement work should be determined based on the results of soil sample tests and previous soundings, and for this reason the accuracy of the soundings is important. Where the soil structure is complicated, it is essential that valid information be obtained which can be utilized in design to maximize the improvement and to achieve the appropriate treatment.

Based on the increase in N-values observed in indoor and *in-situ* tests, we propose a method in which the diameter and pile pitch of QCB piles are determined to maximize the accuracy of the improvement design, taking into account the sand grain size and the influence of the upper soil pressure. The results of tests performed to evaluate this method are described hereafter.

RELATION BETWEEN OVERBURDEN PRESSURE AND THE $N \sim e$ RELATION

The relative density has been inferred from the N-value in a standard penetration test by Terzaghi and Peck.[2] The N-value is considered to be constant even if the grain size, grain size distribution, or overburden pressure varies. In recent studies by Gibbs and Holtz,[3] some practical methods of inferring relative density have also been proposed. According to these methods, the N-value is significantly affected by the effective overburden pressure (P), one of the influential factors along with the relative density of the sand. One example is indicated in Fig. 1(A). The relationship between the N-value and the relative density proposed by Terzaghi and Peck is taken to be a case in which $P \fallingdotseq 14\ \mathrm{tf/m^2}$. Even if the degree of compaction is constant, the N-value increases corresponding to the increase in overburden pressure or the depth from the surface. Therefore, the degree of compaction determined from the N-value should be corrected by using the $N \sim P \sim D_r$ relation. Before the relationship between the N-value and the void ratio of the sand is obtained, the grain size and U_c should be obtained from the above method

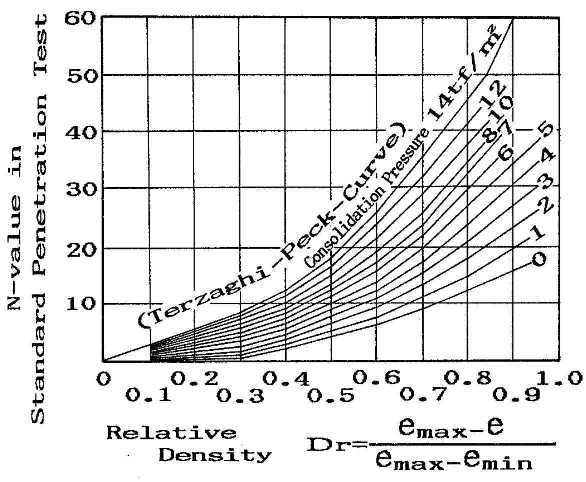

Fig. 1(A). $N\sim P\sim D_{r1}$ Relation

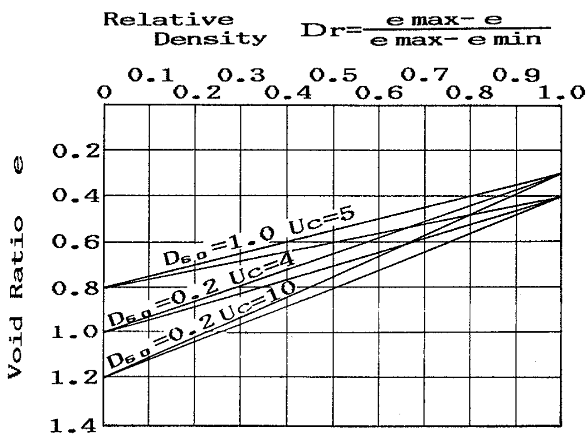

Fig. 1(B). $D_r\sim e$ Relation

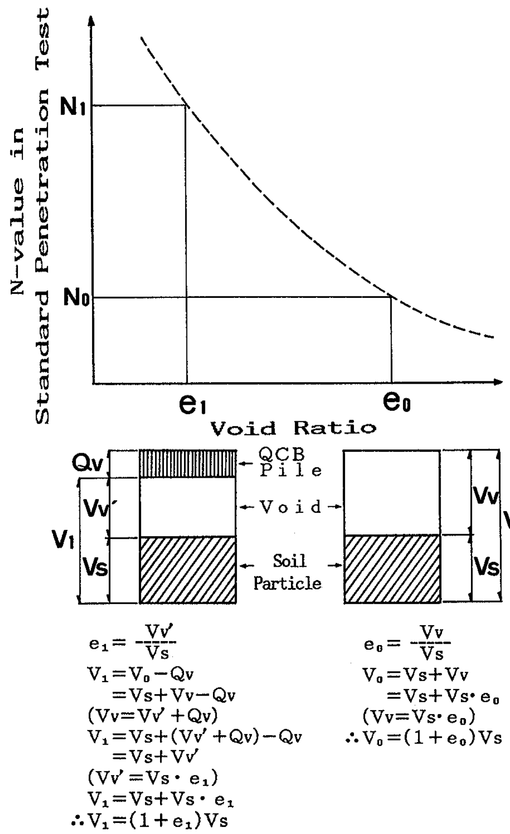

Fig. 2. $N\sim e$ Curve

for inferring $e_{\max}$ and $e_{\min}$. The $e\sim D_r$ relation can then be obtained from the equation $e=e_{\max}-D_r(e_{\max}-e_{\min})$ (Fig. 1(B)). These two relations, *i.e.* the $N\sim P\sim D_r$ and the $e\sim D_r$ relations, are utilized to relate the N-value to e. At present, this seems to be the best way to obtain valid information for improvement design.

In general, the relation between the N-value and the void ratio of sandy soils varies with the grain size distribution and the effective overburden pressure. However, it is generally represented by the $N\sim e$ curve shown in Fig. 2. As the N-value increases, the void ratio decreases. Based on the known $N\sim e$ relation of the soil, the void ratio of untreated soil (e_0) is reduced to the required void ratio (e_1) in order to increase N_0 (the N-value for untreated soil) to N_1 (the required N-value). N_1 and e_1 are sometimes determined in terms of static bearing capacity. However, in most cases, they are determined in terms of dynamic bearing capacity, *i.e.* to prevent liquefaction.

Based on the relation between D_r and the internal friction angle (ϕ), the $N\sim e$ curve shown in Fig. 2 was obtained by substituting N and e for ϕ and D_r. In design, as shown in Fig. 1(B), $e_{\max}$ and $e_{\min}$ are inferred by using D_{60} and U_c, which are obtained by grain size analysis. The $e\sim D_r$ relation is then obtained from these inferred values. The relation between D_r and the N-value obtained by a standard penetration test under confinement by overburden pressure is also utilized.

Consequently, the bearing capacity and settlement of untreated soil should be examined by utilizing the N-value of untreated soil. If soil improvement is necessary, the N-value of the improved soil (N_1) has to be calculated. The relative density of treated soil (D_{r_1}) and e_1 can be obtained from Figs. 1(A) and 1(B) after N_1 has been determined.

THE RELATIONSHIP BETWEEN THE DECREASE IN VOID RATIO, PILE DIAMETER, AND PILE PITCH

The types of placements of QCB piles are shown in Fig. 3.

Regular square placement:

$$V_0 = X^2 = V_s(1+e_0) \tag{1}$$

Volume after improvement:

$$V_1 = V_s(1+e_1) = V_0 - Q_v \tag{2}$$

From Eqs. (1) and (2),

$$\frac{V_1}{V_0} = \frac{(1+e_1)}{(1+e_0)} \tag{3}$$

$$\therefore Q_v = \frac{e_0 - e_1}{1+e_0} \times X^2 = \frac{\Delta e}{1+e_0} \times X^2 \tag{4}$$

Each letter represents an element as follows:

A: area stabilized by a pile (m^2)
e_0: void ratio before treatment
e_1: void ratio after treatment
Q: the volume of a QCB pile (m^3)
Q_v: the volume of a swollen QCB pile (m^3)
V_0: the volume of sandy soil (m^3)
V_s: the volume of sand particles in soil (m^3)
X: pile pitch (m)

The swelling rate of QCB (FQ) is given by

$$FQ = \frac{Q_v}{A+1} = \frac{Q_v}{X^2 \times 1} = \frac{\Delta e}{1+e_0} \tag{5}$$

FQ: the swelling rate of QCB in unit soil

The relation between A at unit depth, X, and Q_v is obtained from Eq. (5) as follows:

For regular square placement:

$$A = X^2 = Q_v / FQ$$

For equilateral triangular placement:

$$A = \sqrt{3} \cdot X^2 / 2 = Q_v / FQ$$

The relation between FQ and X is represented by the following equations:

For regular square placement:

$$FQ = Q_v / X^2$$

For equilateral triangular placement:

$$FQ = Q_v / (0.866 \cdot X^2)$$

⟨Information⟩

Initial diameter of a QCB pile: d_0

Diameter of a swollen QCB pile:

$$d_n \fallingdotseq (1.3 \sim 1.5) d_0$$

$$\therefore d_n \fallingdotseq 1.3 \cdot d_0$$

(A safety factor of 1.3 is used here.)

Initial volume of a QCB pile:

$$Q = \pi \cdot d_0^2 / 4$$

Volume of a swollen QCB pile:

$$Q_v = \pi \cdot (d_n)^2 / 4$$

$$\therefore d_n = \sqrt{4 \cdot Q_v / \pi} = 1.13\sqrt{Q_v}$$

and

$$d_0 = 0.869\sqrt{Q_v}$$

The relationship between Q and Q_v:

$$Q_v \fallingdotseq 1.7 \cdot Q$$

The relations between the decrease in void ratio, the diameter of a QCB pile, and pile

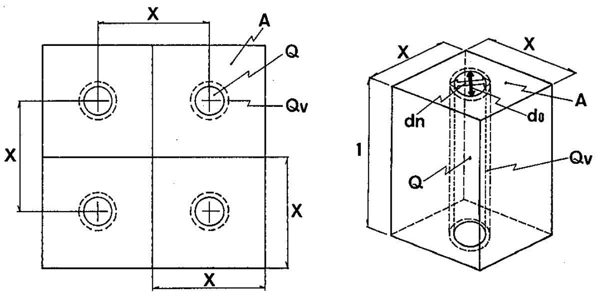

Fig. 3. Influence chart of a QCB pile

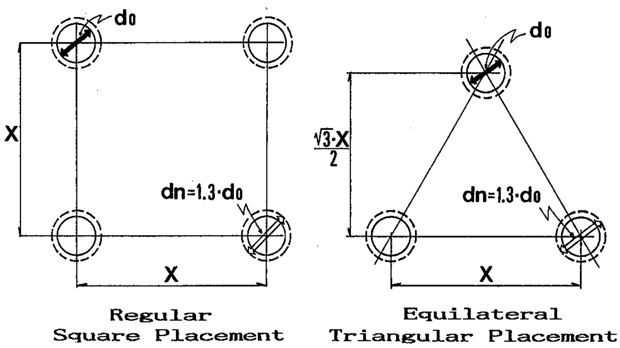

Fig. 4. Placement of QCB piles

pitch in regular square placement and equilateral triangular placement are indicated in Table 1, Fig. 5(A), and Fig. 5(B).

In design and construction, based on Figs. 1(A) and 1(B), the relation between N_0, overburden pressure (P), and the relative density of untreated soil (D_{r_0}) may be obtained from N_0, the grain size analysis of sand before treatment (D_{60}, U_c), and the specific gravity test on soil particles. When the relationship between D_{r_1} and e_1 has been obtained, the decrease in void ratio (Δe) is obtained from Fig. 1(B). Furthermore, the relationship between the initial diameter of a QCB pile (d_0) and pile pitch (X) is obtained from Figs. 5(A) and 5(B). All these factors are necessary for design and improvement work in which QCB piles are used to counter liquefaction.

Table 1. Swelling ratio of QCB pile in unit soil

Initial diameter of Pile d_0 (m)	Regular Square Placement — Pile Pitch X (m)										
	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4	2.6	2.8	3.0
0.2	0.053	0.037	0.027	0.021	0.016	0.013	0.011	0.009	0.008	0.007	0.006
0.3	0.120	0.083	0.061	0.047	0.037	0.030	0.025	0.021	0.018	0.015	0.013
0.4	0.213	0.148	0.109	0.083	0.066	0.053	0.044	0.037	0.032	0.027	0.024
0.5	0.333	0.231	0.170	0.130	0.103	0.083	0.069	0.058	0.049	0.042	0.037
0.6	0.479	0.333	0.244	0.187	0.148	0.120	0.099	0.083	0.071	0.061	0.053
0.7	0.652	0.453	0.333	0.255	0.201	0.163	0.135	0.113	0.096	0.083	0.072
Initial diameter of Pile d_0 (m)	Equilateral Triangular Placement — Pile Pitch X (m)										
	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4	2.6	2.8	3.0
0.2	0.061	0.043	0.031	0.024	0.018	0.015	0.013	0.010	0.009	0.008	0.007
0.3	0.139	0.096	0.070	0.054	0.043	0.035	0.029	0.024	0.021	0.017	0.015
0.4	0.246	0.171	0.126	0.096	0.076	0.061	0.051	0.043	0.037	0.031	0.028
0.5	0.385	0.267	0.196	0.150	0.119	0.096	0.080	0.067	0.057	0.048	0.043
0.6	0.553	0.385	0.282	0.216	0.171	0.139	0.114	0.096	0.082	0.070	0.061
0.7	0.753	0.523	0.385	0.294	0.232	0.188	0.156	0.130	0.111	0.096	0.082

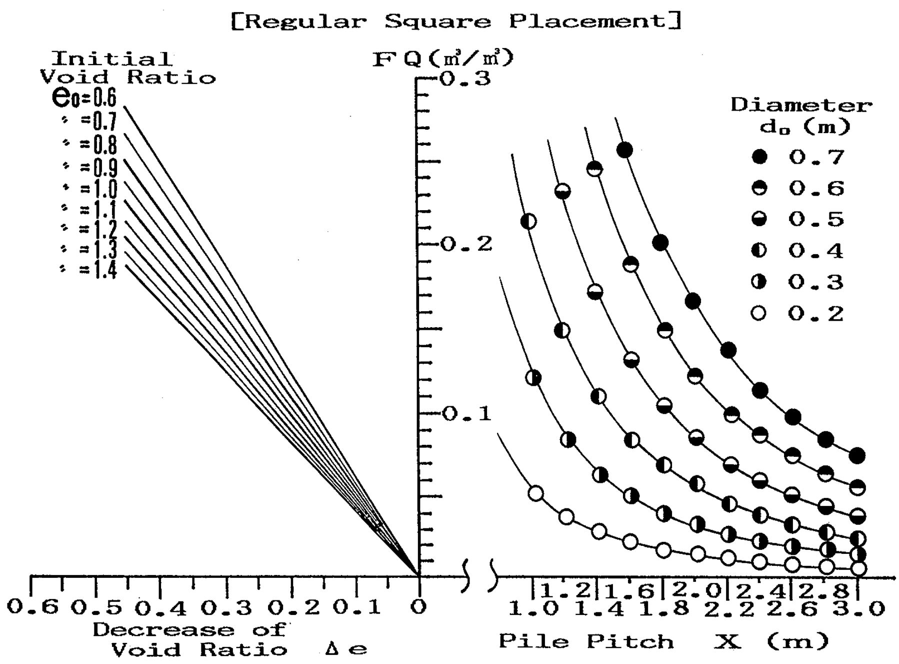

Fig. 5(A). Relationship between the decrease in void ratio, the diameter of QCB-piles, and the pile pitch

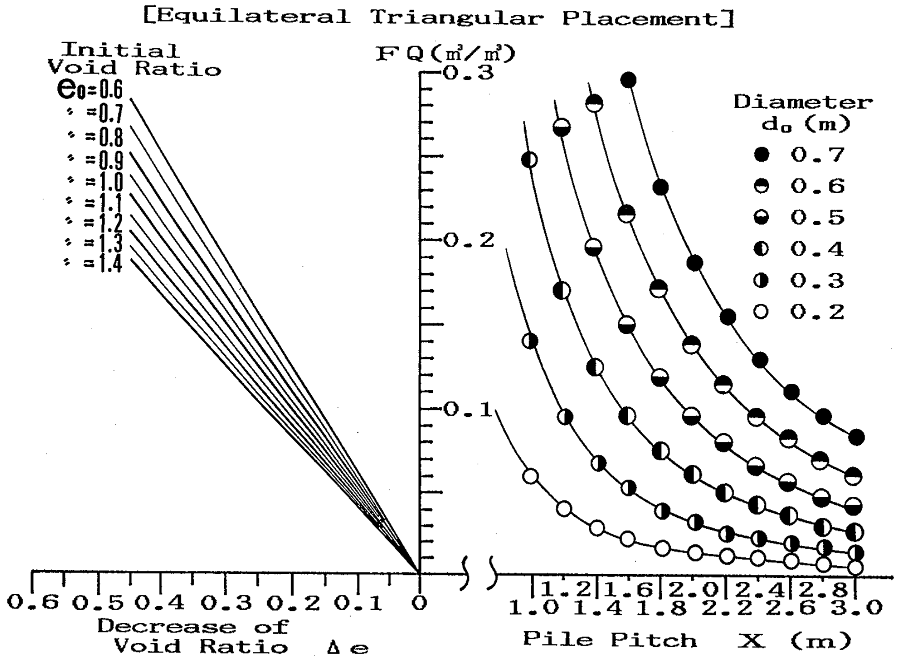

Fig. 5(B). Relationship between the decrease in void ratio, the diameter of QCB piles, and the pile pitch

EXPERIMENTAL INSPECTIONS

Indoor and in-situ tests were carried out to verify the relations described in the previous section. In this section, the validity of information obtained from these relations is examined.

Indoor Tests[4)]

Cubic test containers 50×50×50 cm were filled with water to one-third of their capacity, and dried sea sand was then added. The supernatant water was absorbed to provide saturated soil. QCB piles (vinyl containers filled with QCB, $d_0=4$ cm, $L=40$ cm) were driven into the sand in the regular square placement pattern at pile pitches of $4\cdot d_0$, $5\cdot d_0$, and $6\cdot d_0$. The swelling pressure of the ground, the pore water pressure, and the densities were measured before and after swelling (Fig. 6). The physical properties of the sea sand are indicated in Fig. 7. The maximum and minimum void ratios of the sea sand were measured by the measuring cylinder method. The swelling pressure of the ground and the pore water

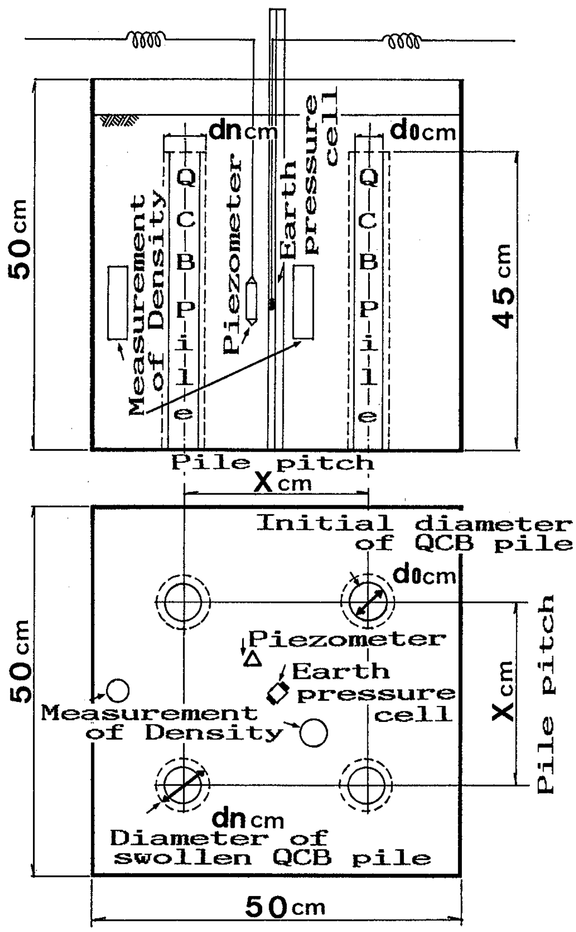

Fig. 6. Test container

pressure are shown in Table 2, and the other results are shown in Table 3. The relation between the void ratio and the relative density is indicated in Table 4. The relative density and the decrease in void ratio were found to be satisfactory, as indicated in Table 4, Fig. 1(B), and Fig. 5(A) based on results using a 1/10 scale model.

In-situ Tests[5)]

Outline of test performance

An in-situ test was carried out at the Fukumuro-Aza-Takasago area, an area of very soft soil in Sendai. The underground water level is high (0.5 m), and liquefaction during earthquakes is considered to be inevitable.

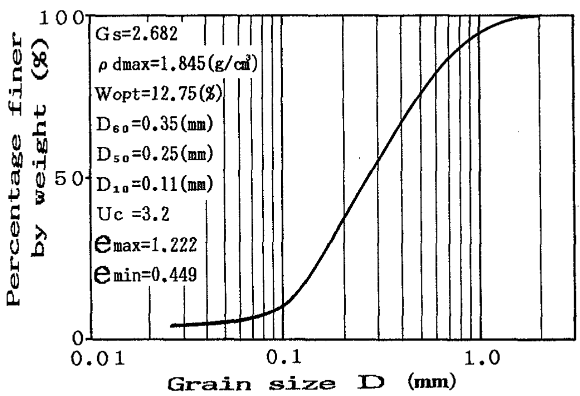

Fig. 7. Grain size distribution and soil properties

As shown in Fig. 8, the grain size distribution of the soil was classified into a range in which liquefaction may occur, and the possibility of liquefaction was confirmed.

Nine QCB piles 0.45 m in diameter and 2.5 m long were placed at a pitch of 1.6 m in the

Table 2. Swelling ground pressure and pore water pressure

Time	Earth Pressure (kgf/cm^2)			Pore Water Pressure (kgf/cm^2)		
	$4 \cdot d_0$	$5 \cdot d_0$	$6 \cdot d_0$	$4 \cdot d_0$	$5 \cdot d_0$	$6 \cdot d_0$
0 min	0.00	0.00	0.00	0.000	0.000	0.000
5	0.10	0.04	0.02	0.015	0.010	0.008
10	0.34	0.13	0.04	0.025	0.021	0.016
20	0.49	0.15	0.09	0.020	0.025	0.022
30	0.35	0.16	0.11	0.013	0.019	0.026
40	0.30	0.18	0.13	0.010	0.014	0.024
50	0.26	0.21	0.15	0.008	0.012	0.020
1:00 hr	0.24	0.21	0.16	0.005	0.010	0.017
1:30	0.20	0.23	0.16	0.003	0.008	0.012
2:00	0.16	0.19	0.14	0.002	0.006	0.010
2:30	0.13	0.16	0.12	0.002	0.003	0.008
3:00	0.11	0.13	0.11	0.002	0.003	0.007
3:30	0.10	0.11	0.10	0.002	0.003	0.007
4:00	0.09	0.09	0.09	0.002	0.003	0.006
4:30	0.08	0.09	0.08	0.002	0.003	0.006
5:00	0.07	0.08	0.08	0.002	0.002	0.006
5:30	0.06	0.08	0.08	0.002	0.002	0.005
6:00	0.06	0.07	0.08	0.002	0.002	0.005
6:30	0.05	0.06	0.07	0.001	0.002	0.004
20:00	0.05	0.06	0.06	0.001	0.002	0.004
22:00	0.05	0.05	0.05	0.001	0.001	0.003
24:00	0.05	0.05	0.04	0.001	0.001	0.002

$1\ kgf/cm^2 = 98.1\ kN/m^2$

Table 3. Influence due to the change of the diameter of pile and pile pitch

Pile pitch	X	(cm)	$4 \times d_0 = 16$ cm		$5 \times d_0 = 20$ cm		$6 \times d_0 = 24$ cm	
Diameter of pile	d_0	(cm)	4 cm		4 cm		4 cm	
Wet density	ρ_{t0}	(gf/cm^3)	1.935		1.926		1.931	
Void ratio	e_0		0.752		0.755		0.753	
Water content	W_0	(%)	26.4		26.0		26.2	
Saturation ratio	Sr_0	(%)	94.2		92.4		93.3	
Dry density	ρ_{d0}	(gf/cm^3)	1.531		1.529		1.530	
Relative density	Dr_0	(%)	60.8		60.4		60.7	
Average diameter of swollen pile		(cm)	4.9		5.0		5.1	
Test Container			Inside	Outside	Inside	Outside	Inside	Outside
Wet density	ρ_{tn}	(gf/cm^3)	2.085	2.006	2.068	1.996	2.046	1.987
Void ratio	e_n		0.574	0.655	0.599	0.669	0.629	0.687
Water content	W_n	(%)	22.4	23.8	23.3	24.2	24.3	25.0
Saturation ratio	Sr_n	(%)	104.6	97.4	104.3	97.0	103.5	97.6
Dry density	ρ_{dn}	(gf/cm^3)	1.703	1.620	1.677	1.607	1.646	1.590
Relative density	Dr_n	(%)	83.8	73.4	80.6	71.5	76.7	69.3
Earth pressure	P_n	(kgf/cm^2)	0.490	—	0.230	—	0.160	—
Pore water pressure	U_n	(kgf/cm^2)	0.025	—	0.025	—	0.026	—

$1\ gf/cm^3 = 9.81\ kN/m^3$ $1\ kgf/cm^2 = 98.1\ kN/m^2$

Table 4. Void ratio and relative density

Pile Pitch	Before Treatment		After Treatment	
	Void Ratio e_0	Relative Density D_{r0}	Void Ratio e_n	Relative Density D_{rn}
$4\times d_0$	0.752	60.8%	0.574	83.8%
$5\times d_0$	0.755	60.4%	0.599	80.6%
$6\times d_0$	0.753	60.7%	0.629	74.7%

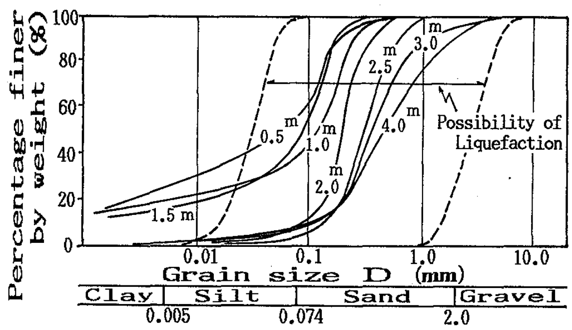

Fig. 8. Grain size distribution corresponding to depth

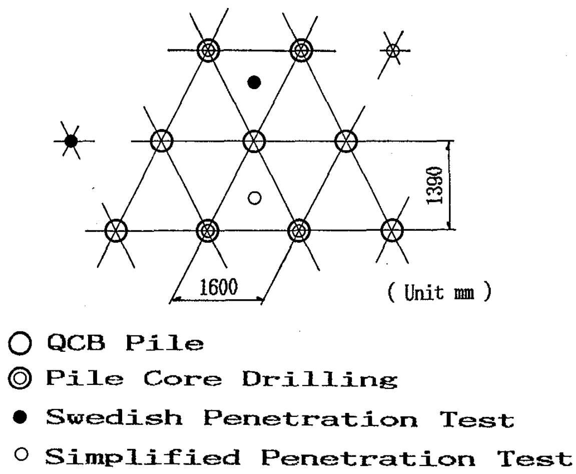

Fig. 9. Placement of piles and test locations

equilateral triangular placement pattern (Fig. 9). Several tests were carried out before the piles were placed and at 2-week and 4-week intervals following QCB treatment.

〈Soil tests〉

Physical test of soil

Swedish penetration test

Simplified penetration test

〈Tests on QCB piles〉

Box shear test

Unconfined compressive strength test

Note-1) A simplified penetration test was employed as a sample of the standard penetration test method, but the hammer weight and the drop height were changed. The energy used in each case was obtained from

$$N_s \fallingdotseq 5\times N$$

Therefore,

$$N\text{-value} \fallingdotseq N_s/5$$

In these equations, N_s represents the number of times the hammer hit the ground in the simplified penetration test.

Note-2) The internal friction angle (ϕ) of the sandy soil was obtained from Dunham's equation[6] ($\phi=\sqrt{12\cdot N}+20$).

The results of these tests indicate that the QCB piles as placed absorbed soil water, swelled, reduced soil voids, and increased soil density (Fig. 10). The N-value also increased in the penetration test.

The N-value before QCB treatment and that 2 weeks after QCB treatment are shown in Fig. 11. The N-value was improved to a range secure against liquefaction after QCB treatment. Thus, such piles seem to be a promising way of countering liquefaction. Furthermore, as shown from the cores of hardened piles 2 and 4 weeks after they were driven, the shear properties (Fig. 12) and compressive proper-

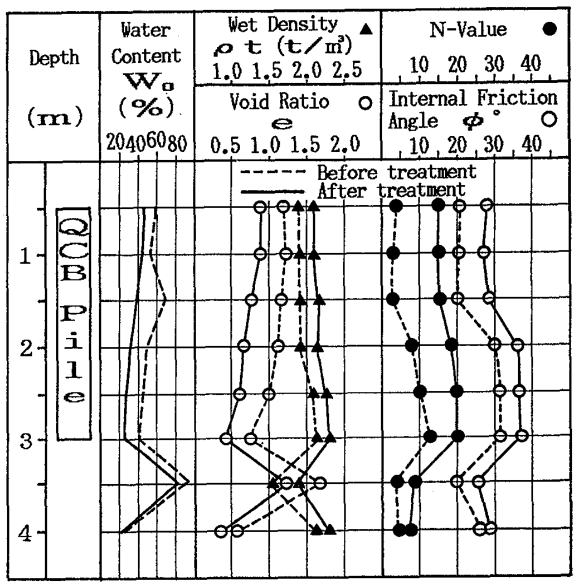

Fig. 10. Effect of soil stabilization

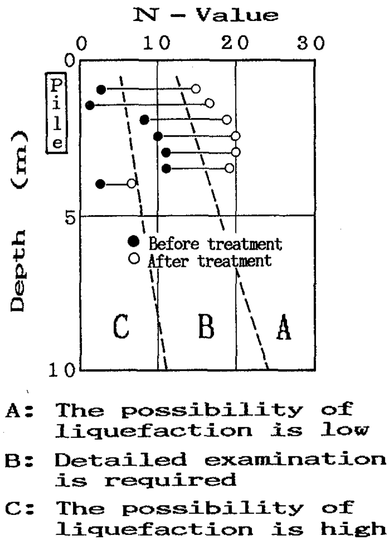

Fig. 11. Liquefaction risks and N-value

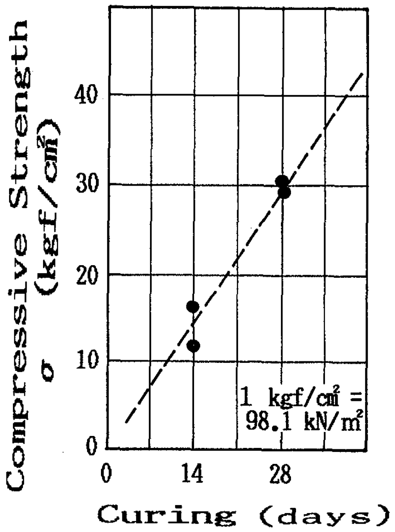

Fig. 13. Compressive strength of hardened QCB piles

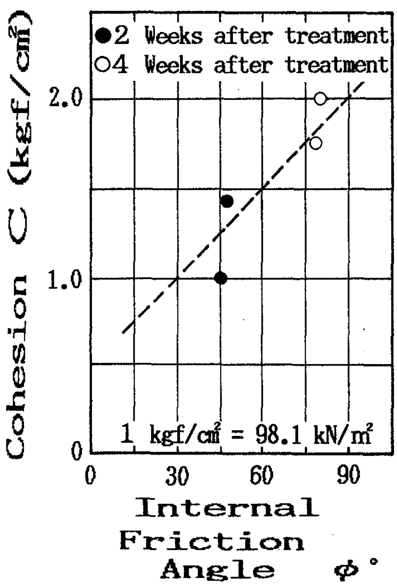

Fig. 12. Shear properties of hardened QCB piles

ties (Fig. 13) of QCB piles were found to be superior to those of quick-lime piles, which are conventionally employed for soil improvement. These results also indicate that QCB piles are effective as a soil improvement measure.

In the in-situ test in which 0.45-m-diameter QCB piles were placed at a pitch of 1.6 m, the results were as follows: e_0 was in the range of 1.2 ~ 0.8, and e_1 was in the range of 0.9 ~ 0.4. Therefore, the decrease in the void ratio (Δe) was in the range of 0.3 ~ 0.4. This was almost identical to the results indicated in Fig. 5(B).

CONCLUSIONS

The problem of foundation failure due to liquefaction is a relatively recent phenomenon in Japan. Thus, few construction methods have been developed to effectively prevent its occurrence. Although studies have been executed on the mechanism of liquefaction, countermeasures remain relatively underdeveloped.

This report examined the decrease in void ratio and the increase in N-value and the resulting increase in relative density after QCB piles were placed into saturated sandy soil which had a high risk of liquefaction. It also examined information for improving design and construction work.

Soil surrounding QCB piles was found to consolidate in a short time, and the increment in density due to the moisture absorption and swelling effect of QCB was observed (static consolidation). The soil pore water pressure showed a slight increase because the moisture absorption and swelling of QCB were equilibrated. The QCB piles formed an aggregate-like material in the soil in a short time, thus enhancing the stability of the composite soil. QCB was shown to increase soil strength and to prevent liquefaction.

We examined the application of the QCB pile method in which internal soil compaction of saturated loose sandy soil can be achieved with vibration- and noise-free work. Progressive studies will be continued to establish the QCB method as a standard procedure and to apply it to other types of soils with composite structure.

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