EVALUATION OF RESISTANCE TO LIQUEFACTION CAUSED BY EARTHQUAKES IN SANDY SOIL STABILIZED WITH QUICK-LIME CONSOLIDATED BRIQUETTE PILES

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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 popular methods are those which increase their density. Vibration and impact methods are commonly employed, but these often pose a problem of noise and vibration in urban areas. Results of previous experiments have shown that Quick Lime Consolidated Briquettes (QCB), a soil stabilizer made of quicklime and cement placed to form cylinders, significantly increases resistance to liquefaction by utilizing the effects of water absorption, swelling, and hardening. In the present study, shaking table tests were performed to evaluate the earthquake resistance of soil treated with QCB. Results of these test showed that the response acceleration and excess pore water pressure in QCB-treated soils were scarcely affected by the excitation of 200 Gals. The soil settlement due to shaking was about one tenth of that for the untreated soil, proving the effectiveness of QCB during earthquakes.

Key words: compaction, earthquake, earthquake resistance, liquefaction, sandy soil, soft ground, soil stabilization (IGC: E8/E14/K5)

INTRODUCTION

In addition to subsurface compaction, several internal compaction techniques are conventionally used to stabilize deep layers of sandy soil. Of these techniques, vibrocomposer 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 (QCB) is a composite soil stabilizer for improving soft ground (Ito et al., 1990). It effectively prevents liquefaction, and it requires no vibration or impact. Static experiments previously performed have confirmed that QCB's absorption, swelling, and hardening effects reduce the water content and void ratio of loose sandy ground (Ito et al., 1989, 1992). Hard piles quickly form, increasing the ground density. Several studies have been conducted on the measures against liquefaction of embankments built on saturated sandy

ground (Koga et al., 1990, Nasu et al., 1985; Enami et al., 1989). In those studies, techniques used as the measures against liquefaction were different from QCB piles. In the present study, shaking table tests were performed to evaluate the earthquake resistance achieved by the QCB method.

EXPERIMENTAL METHOD

Test Outline

A saturated sandy sample was prepared by evenly spreading air-dried sea sand (grain size and properties as shown in Fig. 1) of 50 cm deep in a transparent container of which dimensions are 200 cm long, 100 cm wide, and 100 cm deep. A thin layer of fluorescent red sand was placed every 10 cm apart both horizontally and vertically, forming a check pattern, to allow ground displacements to be observed during and after shaking. Water was percolate upwards from the bottom of the container to produce saturated sandy ground with a relative

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density of almost 60% ($Dr \approx 60\%$).

Two similar saturated sandy ground samples were prepared, and a predetermined number of QCB piles ($\phi = 4$ cm, $L = 40$ cm, driving pitch = 20 cm) were formed by filling cylindrical vinyl bags with QCB, and depositing them in casings previously driven into the ground in a regular square placement pattern. The vinyl bags were then broken to activate the QCB, and the piles were left for one week to cure. A 20 cm-deep embankment was then placed on both the treated and untreated samples (Ito et al., 1991, 1992). The experimental parameters are shown in Table 1.

1) In case 1, the embankment was placed on the central portion of the untreated saturated sandy ground, and the behavior of the ground and the embankment during shaking were observed.

2) In case 2, 30 QCB piles ($\phi = 4$ cm, $L = 40$ cm, spacing pitch = 20 cm) were placed in the central portion of the sandy ground, and the embankment was placed in the same position as for case 1. The behavior of the treated

and untreated portions, and the embankment, were investigated.

3) In case 3, 25 QCB piles ($\phi = 4$ cm, $L = 40$ cm, spacing pitch = 20 cm) were placed in a half of the saturated sandy ground, and the embankment was formed on one end of the sample. The behavior of the boundary between the treated and untreated portions, and of the embankment, were investigated. Figure 2 shows cases 1, 2, and 3 and the locations of the measuring instruments.

EXPERIMENTAL CONDITIONS IN ACCORDANCE WITH THE SIMILARITY RULE

Test models were prepared based on a prototype comprising a 2-m-high embankment on a 5-m-thick soft sand layer. Experimental conditions were determined by taking into account the similarity of the model to the prototype. A similarity rule can be obtained by using one of the following methods: π theorem, the ratio of powers, and a governing equation.

All these methods produce a dimensionless quantity, and the similarity rule is obtained by using the same dimensionless quantity in two systems. The π -theorem method applies Buckingham's π theorem. The ratio of powers method produces an independent dimensionless quantity by using a ratio of important powers. The governing equation method renders the equation and boundary conditions dimensionless (Fukuda et al., 1985). In this experiment, we adopted the ratio of powers method, as shown in Table 2, because the physical quantity determined by the power was dominant.

Fig. 1. Grain size and properties of sea sand

EXPERIMENTAL RESULTS AND COMMENTS

Excess Pore Water Pressure in the Reaction of QCB Piles

Figure 3 shows the change in pore water pressure measured at the points of P-2, P-3, and P-4 during the chemical reaction of the QCB piles. The locations of the

Table 1. Test parameters

Model	Ground condition	Shaking Classification	Input Maximum Acceleration	Shaking Wave Form	Frequency	Shaking Time
CASE 1	*Saturated Sandy Ground *Embankment on unimproved soil	Liquefiable Shaking	200 Gal	Sine Wave	2.0 Hz	10 sec
CASE 2	*Saturated Sandy Ground *Embankment on improved soil	Liquefiable Shaking	200 Gal	Sine Wave	2.0 Hz	10 sec
CASE 3	*Saturated Sandy Ground *Embankment on improved soil	Liquefiable Shaking	200 Gal	Sine Wave	2.0 Hz	10 sec

Fig. 2. Test models and the location of measuring instruments

Table 2. Similarity rules involving the nonlinear ground response

	Model/Prototype	
Length	$1/\lambda$	0.1
Frequency	$\lambda^{3/4}$	5.623
Density	1	1
Dynamic Displacement	$1/\lambda^{3/2}$	0.032
Dynamic Strain	$1/\lambda^{1/2}$	0.316
Stress	$1/\lambda$	0.1
Acceleration	1	1

λ : Scaling Factor

Fig. 3. Excess pore water pressure during the chemical reaction of the QCB piles (In case 3)

measured points are shown in case 3 of Fig. 2. P-1 is situated in the untreated soil. P-2, P-3 and P-4 are located in the treated soil. P-2 is 20 cm away from the toe of the embankment; P-3 is beneath the toe of the embankment; and P-4 is under the embankment.

The QCB's simultaneous absorptive and swelling effects indicated the maximum excess pore water pressure at about 0.8 kN/m^2 (8.0 gf/cm^2) (Ito et al., 1991).

Horizontal Accelerations, Excess Pore Water Pressure, and Crest Settlement

Figure 4 shows the time history of the horizontal accelerations (200 Gals/2 Hz/Sine wave), excess pore water pressures, and crest settlements for cases 1, 2 and 3 (Ito et al., 1992).

The accelerations were measured at A-1 (shaking table), A-2 (embankment crest), and A-3 (ground surface). The following observations were made:

1) In case 1, the horizontal acceleration at A-1 was 200 Gals, and the response accelerations at A-2 embankment and A-3 were amplified to 300 Gals by the soft ground.

Seven seconds after the excitation, the pore water pressure were about 2.9 kN/m^2 (30 gf/cm^2) at P-2 and 2.0 kN/m^2 (20 gf/cm^2) at P-1. An embankment settlement was about 8 cm (40%).

2) In case 2, the response acceleration at A-2 was 200 Gals, the same as that at A-1. The response acceleration at A-3 was amplified to about 300 Gals. Excess pore water pressures seven seconds after the excitation were about 0.8 kN/m^2 (8 gf/cm^2) at P-2 and 1.5 kN/m^2 (15 gf/cm^2) at P-1. The embankment settled only by 6 mm (3%).

3) In case 3, an excess pore water pressure nine seconds after the excitation was 2.9 kN/m^2 (30 gf/cm^2) at P-1.

The response acceleration at A-3 was amplified to about 300 Gals. Excess pore water pressures seven seconds after the excitation were 1.0 kN/m^2 (10 gf/cm^2) at P-2, 0.8 kN/m^2 (8 gf/cm^2) at P-3, and 0.6 kN/m^2 (6 gf/cm^2) at P-4 respectively.

The response acceleration at A-2 was 200 Gals, the same as that at A-1. An embankment settlement was only 7 mm (3%). QCB treatment produced a firm composite

Fig. 4. Horizontal acceleration, excess pore water pressure, and crest settlement

soil with increased density, and increased the strength of the embankment ground. The efficacy of QCB was also supported by a small increase in response acceleration and excess pore water pressure.

Coefficient of Dynamic Excess Pore Water Pressure

The ratio of dynamic excess pore water pressure (γ_{ud}), obtained from the ratio of effective overburden pressure (U_d) at each measurement location to the mean value (σ_{vo}) of excess pore water pressure, was related to shak-

ing time as shown in Fig. 5.

In general, $\gamma_{ud} > 0.5$ is taken to represent the occurrence of liquefaction (Iai et al., 1991). The following points were noticed from Fig. 5.

- 1) In case 1, γ_{ud} was over 1.0 at P-1 five seconds after the excitation, indicating that the site was fully liquefied. The maximum γ_{ud} was 0.6 at P-2 six seconds after excitation, indicating the occurrence of liquefaction.
- 2) In case 2, γ_{ud} was almost 0.6 at P-1 six seconds after the excitation, indicating the occurrence of liquefaction.

Fig. 5. Change in dynamic excess pore water pressure coefficient with time

The maximum γ_{ud} was about 0.2 at P-2 in the treated ground, indicating no liquefaction.

3) In case 3, γ_{ud} was over 1.0 at P-1 four seconds after the excitation, indicating full liquefaction.

The maximum values of γ_{ud} were 0.38 at P-2, 0.27 at P-3, and 0.16 at P-4, respectively indicating no liquefaction.

Thus, with QCB treatment, voids were reduced and ground density was increased (Ito et al., 1989, 1990). Fur-

thermore, excess pore water pressures normally induced by vibration were suppressed (Ito et al., 1992), indicating that little liquefaction occurred.

Deformation of Embankment and Ground

Figure 6 shows the deformation of the models due to shaking.

It may be pointed out the following points:

- 1) In case 1, the ground softened and the entire embankment settled. The base ground spread laterally and heaved, deforming the whole ground area.
- 2) In case 2, little deformation was observed in the treated ground under the embankment. However the untreated ground deformed due to liquefaction.
- 3) In case 3, the treated ground under the embankment suffered little deformation. However, the boundary between the treated portion and the untreated portion deformed slightly. The untreated portion seriously deformed due to liquefaction. The treated ground under the embankment, i.e., composite ground, was strengthened because of the increased density and the QCB's hardening effect (Ito et al., 1989, 1992).

Noticeable displacements were observed only at the toes of the slopes (Ito et al., 1992).

Extent of Improved Area

The improvement of the ground extends in both the depthwise direction and horizontally. The depth of the improvement can be made to correspond to the thickness

CASE 1

GROUND CONDITION

	Place	W _s (%)	ρ_t (g/cm ³)	ρ_d (g/cm ³)	e	Dr (%)
Before	NO. 1	21.04	1.845	1.524	0.760	59.8
Shaking	NO. 2	20.24	1.835	1.526	0.757	60.2
After	NO. 1	22.94	1.870	1.521	0.763	59.3
Shaking	NO. 2	22.18	1.865	1.526	0.757	60.2

CASE 2

GROUND CONDITION

	Place	W _s (%)	ρ_t (g/cm ³)	ρ_d (g/cm ³)	e	Dr (%)
Before	NO. 1	20.73	1.838	1.522	0.762	59.5
Shaking	NO. 2	16.11	1.954	1.683	0.594	81.3
After	NO. 1	21.12	1.875	1.548	0.733	63.3
Shaking	NO. 2	16.20	1.952	1.680	0.596	80.9

CASE 3

GROUND CONDITION

	Place	W _s (%)	ρ_t (g/cm ³)	ρ_d (g/cm ³)	e	Dr (%)
Before	NO. 1	18.80	1.820	1.532	0.751	60.9
Shaking	NO. 2	15.05	1.940	1.686	0.591	81.6
After	NO. 1	19.16	1.865	1.565	0.714	65.7
Shaking	NO. 2	15.59	1.952	1.689	0.588	82.0

Fig. 6. Displacement of ground due to vibration

of soil that is estimated likely to liquefy, but the extent of improvement in the horizontal direction is not so easy to determine. It is considered that it is necessary to improve the soil within an area that will contribute to the stability of structures.

One problem with various improvement methods designed to increase density concerns the behavior of the boundary surface between the treated and untreated ground during an earthquake.

Under conditions where an external force is applied to the soil comparatively quickly with respect to the permeability of the soil, such as during an earthquake, the stability of soil and structures at this boundary is determined by the undrained shear strength of the sand, and this undrained shear strength varies with changes in the effective stress.

However, it has become necessary to consider not only the component due to seepage flow as the excess pore water pressure (U_d) occurring during an earthquake, but also the lowering of the excess pore water pressure caused by the tendency (positive dilatancy) of the sand particle structure to undergo volumetric swelling as it shears. With existing techniques, it is difficult to express this phenomenon quantitatively by a simple model. Therefore, when determining the extent of the area that is compacted, we must apply an engineering interpretation from a macroscopic viewpoint.

The pressure in the boundary between the treated and untreated ground is subjected to pressure increase from the liquefied portion. Steady seepage analysis of this area

suggests that excess pore water pressure is produced, as shown in Fig. 7 (Iai et al., 1991). Where U_d/σ_{vo} is over 0.5 (σ_{vo} is effective overburden pressure), i.e., the square ABCD in Fig. 7, the reduction in stability must be estimated. In fact, in the shaking test for case 3, the area within triangle ACD was shown to be unstable.

Thus, in examining the stability of the ground and buildings, the instability of this portion should at least be estimated, and in determining the extent of the improved area, this portion should be considered as liquefiable.

FEM ANALYSIS

The ground strengthened with QCB piles was loaded with the embankment and horizontally vibrated, and a stress-strain analysis was carried out to obtain the characteristics of the treated ground and the hardened piles. The analysis was performed based on the linear elastic model.

This simple analysis was conducted to examine the effectiveness of QCB piles because the QCB piles are quite different in its material from those used in the conventional techniques such as sand compaction piles and cement columns.

Model for Analysis

For case 3, untreated ground and treated ground four weeks after treatment were analyzed.

A cross-sectional view of a two-dimensional model composed of 302 elements and 182 nodes is shown in Fig. 8.

Analytical Parameters

Table 3 shows the soil parameters used in the analysis.

These parameters were decided based on the results of static experiments. As in the shaking table test, the horizontal acceleration K_s was assumed to be approximately 0.20 (maximum horizontal acceleration $\alpha_{max} \approx 200$ Gals, 20 Hz, sine wave).

Analytical Results and Comments

Figure 9 shows the ground displacement, the max-

Fig. 7. Excess pore water pressure ratio (U_d/σ_{vo}) indicating the extent of the improved area

Fig. 8. Model for analysis

Table 3. Soil parameters used in analysis

	Unit Weight γ_t (kN/m ³)	Modulus of Elasticity E (kN/m ²)	Poisson's Ratio ν
Banking Materials	18.6	98,100	0.35
Unimproved Saturated Sandy Soil	17.7	27,468	0.46
Improved Saturated Sandy Soil	18.6	117,720	0.34
Hardened QCB Pile	16.7	2,648,700	0.26

*1 kN/m³=0.1 gf/cm³, 1 kN/m²=0.01 kgf/cm²

imum shear stress and the major principal stress contours. The horizontal displacement of the embankment on the treated ground was only 4 mm. The maximum shear stress was the greatest in the treated portion, i.e., 33.4 kN/m² (0.34 kgf/cm²).

The major principal stress in this portion was 755.4 kN/m² (7.7 kgf/cm²) (Ito et al., 1992). After four weeks of curing, the shear, compressive, and bending strengths were 294.3 kN/m² (3.0 kgf/cm²), 3433.5 kN/m² (35 kgf/cm²), and 1373.4 kN/m² (14 kgf/cm²), respectively. These results show that ground improved with QCB can bear stress concentrations caused by the vibration levels employed in this experiment.

TEST RESULTS AND DISCUSSION

In this test, models were subjected to 20 cycles at a horizontal acceleration of 200 Gals with a duration of ten seconds. Expressed as earthquake accelerations corresponding to intensities determined by the Japanese Meteorological Agency, this is equivalent to a seismic intensity V of 80 to 250 Gals. If model/prototype ratios of 1 for acceleration, 6 for vibration frequency, and 0.3 for the strain (derived by the similarity rule) are substituted, this would correspond to an earthquake of seismic intensity V continuing for 60 seconds, and it is thought that settlement of the embankment would be about three times in the prototype. Since the settlement of the embankment model on the treated ground was 3%, this corresponds to 9% in the prototype. This may not be regarded as insignificant in practice. However, the settlement of the embankment model on the untreated ground was about ten times as large as this. In consequence, QCB piles are judged to be effective to reduce the settlement of the embankment about one tenth of that for the untreated ground.

A sine-wave excitation was employed in the test, but such acceleration amplitudes of the same amplitude do not occur repeatedly in an actual earthquake. If the excitation experienced during an earthquake is considered to consist of random wave, the excitation used in the test was much stronger than that of a real earthquake.

CONCLUSIONS

This series of the tests has confirmed that, if QCB piles are driven into a saturated sandy soil that is considered likely to liquefy during an earthquake, the action of the QCBs in absorbing water and swelling will compress the voids in the soil and thus increase its density. After the swelling, the strength of the soil as a composite soil containing hardened piles is increased, thus dramatically increasing the liquefaction strength of the soil. We have verified that this method can be expected to be effective when used as a technique for preventing liquefaction in the foundations of large structures.

We have also verified that the area within which the ground is improved must extend as far horizontally as it will contribute to the stability of structures, in the same

CASE 3

Fig. 9. Ground behavior 10 seconds after excitation

way as improvement methods where the density of the soil is increased by compaction.

It should be noted that there will be variations in the vibration characteristics at the boundary between the ground that is treated by the addition of QCB piles and the ground that is not thus treated, depending on excess pore water pressure and liquefaction of the untreated ground. As a result, it is considered that the inertial force applied to the treated ground will vary, but further detailed investigation will be necessary to determine to what degree this will occur, and also to determine resultant effects such as those on the treated ground and on the stability of the structures.

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