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SYNOPSIS: Recently,structures are often sited on a soft ground. So, a friction pile and a floating foundation are considered a possible and promising one. The object of this paper is to study the dynamic behavior of soft clay ground-structure supported by friction pile foundation on the basis of the shaking table test. Major results are as follows; 1) The residual settlement of structure resulting from decreasing the bearing capacity of the ground, which is caused by the accumulation of the excess pore water pressure, is larger than that of ground without structures. 2) The initial shear stress is a main contributor of the grounds of the generating of the excess pore water pressure in the ground between piles.

INTRODUCTION

Most of the reclaimed land of waterfront developments is formed on soft alluvial ground, and the bearing stratum is generally deep due to the reclamation. This alluvial ground above the bearing stratum will be either unconsolidated or normally consolidated, after it has served. This condition can cause settlement in a structure and a pile due to negative friction when long-end-bearing piles are used in this type of ground.

When pile foundations first came into use, friction pile foundation were widely used in Japan, but tilting and differential settlement occurred in most of the structures because the bearing capacity of friction piles was not clearly understood[1]. After heavier structures and the construction of long-end-bearing piles became possible, friction piles were no longer used. However, recently most of the loading tests for end-bearing piles indicate that practically the total load is carried by friction force, and it is clear that the initial settlement depends on the skin friction of the pile[2]. As a result, friction piles are being reconsidered.

Morishige[3] investigated examples of friction piles and end-bearing piles used for the Tokaido Bullet Train in central Japan, and states that the differential settlements with friction piles foundation are less than those with bearing pile foundations. He says that the reason for this is that, with the bearing pile foundation, the number of piles used is fewer, the distance between piles is greater, and the negative friction from the consolidated stratum is the cause of the differential settlement.

In the case of buildings, short friction piles for shorter buildings are sometimes used, but because ground is not uniform or stable with pile lengths of 10m or less, long friction piles are beginning to be used. Sakaguchi[4] investigated long friction piles, and states that, when all strata are normally consolidated strata, tests are necessary to measure both the bearing capacity of the friction pile and the consolidation of the pile tip stratum and the surrounding soil, but that there is little need to consider the amount of consolidation settling in the case of over-consolidation caused by drawing ground water in the underlying strata. Also, he states that when the underlying strata is diluvial clay strata, this strata is good quality bearing strata, and that examinations on consolidation settling of these strata are unnecessary in most cases.

Also, Sakaguchi[5] points out that since the N-value is small, the over-consolidated alluvial clay and diluvial clay ground, which are usually overlooked for use as bearing ground, can actually be used as bearing strata for medium sized and short building and that considerable friction force can be expected. Utsunomiya et al [6] also state that there is almost no increased settlement with long friction piles installed in consolidated ground, and that they provide as much, or more, short-term safety as bearing piles.

As mentioned above, the long-term bearing capacity and settlement characteristics of friction piles is becoming better understood, but we cannot yet say that the behavior of friction pile foundations during and after earthquakes is well understood. In clay with a normal consolidation or low over-consolidation ratio, a softening phenomenon caused by excessive pore water pressure was confirmed by laboratory element tests[7], and it is thought that the evaluation of friction pile foundations during and after earthquakes is important.

In this paper, we describe a model test with a shaking table on a long friction pile foundation and structure. The experimental study was to investigate the behavior during earthquakes which use friction pile foundations in soft clay ground.

EXPERIMENTS

GENERAL FEATURES OF MODEL TEST

The experimental shear tank, structure and pile foundation, and the arrangement of measuring instruments used in the test are shown in Figure 1. The measuring instruments used were 16 accelerometers, 11 pore water pressure transducers and 5 displacement transducers.

The internal dimensions of the experimental shearing tank were 1.2m wide(shaking direction), 0.8m long and 1.0m high. The foundation and floors of a building, made of iron, were 150 mm wide(shaking direction), 750mm long, and 19mm thick. The walls of the building were phosphor bronze,and were 2.0mm thick, 750mm wide and 150 mm high. The outside diameter of piles was 25.15mm and inside diameter was 16.60mm. They were made of hollow acryl and a length of 400mm to make it a long pile according to the Broms formula[8]. The configuration of the piles was 2 rows in the shaking direction and 5 rows in length, and the center to center spacing between piles was 130 mm. A kaolin was used as the material for the clay ground model. Slurry of the kaolin that was boiled for saturating, was one-dimensionally consolidated at 10cm or 25cm in height, under a pressure 12.75kN/m^2.. This process was repeated six times, and then the clay ground model was prepared with a height of 965mm. The physical characteristics of the clay samples are that wet density ρ_t is 1.60 g/cm^3, water content ratio w is 38.9%, void ratio e is 1.15, degree of saturation S_r is 94.0%, and plastic index I_p is 16.6%.

The piles were set by using a vane to make holes in the ground, inserting piles and slurry after the ground was prepared, and then performing consolidation again.

The estimated size of the real object according to the method described by Kokusho et al , pertaining to the law of similarity, is summarized as follows. According to them, the stress and strain of the ground are as indicated by the Hardin-Drnevich model, and if the contraction scale ratio (l_p/l_m) of the model to the real object is assumed to be λ,the analogy of the frequency(ω_m/ω_p) becomes $\omega^{3/4}$. If the ground depth of the real object is estimated to be 30m, the primary natural oscillation frequency of the model is 17Hz according to the vibration test results described below, the primary natural oscillation frequency of real object is 1.3Hz, while the piles will have 12m and the diameter is 60cm, and the height of the structure is estimated to be 20m.

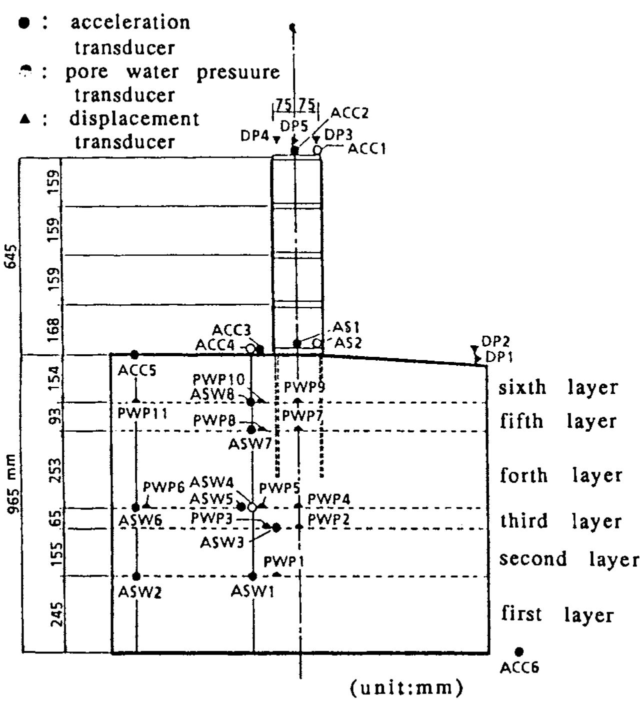

Fig. 1 Model of Structure and Ground

METHOD OF THE VIBRATION AND MEASUREMENT

The shaking table used was a 4m wide, 5m long electrohydraulic one-dimensional shaking table. Tow identical models were prepared. Vibration of the shaking table was increased in a step by step fashion, with a maximum value for acceleration being input. Table 1 shows the maximum input acceleration, measured at the base of the shaking table. The vibration time was for each step 90 seconds. For the Cases 1-4 and 2-2, the dissipation of the residual pore water pressure and residual settlement was measured for one or two days following the vibration .

TABLE 1 Maximum value of Input Acceleration

Case	m/s^2
1-1	0.635
1-2	1.025
1-3	2.225
1-4	2.716
1-5	3.640
2-1	2.430
2-2	3.189

TABLE 2 Cyclic Triaxial Test Details of Clay Samples

Case	Confining Pressure(kN/m^2)		Stress Ratio
	at consolidation σ_0	at loading σ_c	$\sigma_d/2\sigma_c$
1	20.	20. (1.0)	0.291, 0.287, 0.258
2	20.	15. (1.3)	0.255, 0.228, 0.289
3	20.	10. (2.0)	0.248, 0.276, 0.252
4	20.	5. (4.0)	0.424, 0.477, 0.445

(): OCR

THE LABORATORY TESTS OF CLAY SAMPLES

Undisturbed samples were taken from the experimental shear tank by block sampling, and the cyclic loading tests were performed with using the cyclic triaxial compression test. The test details are shown in Table 2. The $\sigma_d/2$ in the table represents the cyclic shear stress, and the σ_c is the constant confining pressure during the test. Each cases was performed three times with a different stress ratio. These tests were performed by a loading speed of 22.6 $KN/m^2/sec$ which is fast in comparison with that of the other studies[10,11].

Figure 2 shows the axial strain waves, excess pore water pressure waves, the relation of stress and strain, and the effective stress paths for Case 1, which was in the normal consolidation condition and had a stress ratio $\sigma_d/2\sigma_c$ of 0.526.

Given the waveforms of both the axial strain and the excessive pore water pressure, together with the repetition of the shearing stress, it is that axial strain and excess pore water pressure increase with oscillation.

From same the figure, one of shows the relation between stress and strain and the effective stress paths, the cyclic triaxial compression tests clearly show that the resistance force on the compression side is greater than that on the tension side, and that the accumulation of excessive pore water pressure, which causes the shift in the mean principle stress, is conspicuous in the loading on the compression side. The pore water pressure rise, which occurs during unloading, is thought to be caused by the time lag between the pore water pressure and the load.

Determining the liquefaction strength of sand is generally performed at when the double amplitude of the axial strain is about 5%. In these clay test, the accumulated excess pore water pressure is not equal to the confining pressure, but the value of the pore water pressure U is equal to the confining pressure. In order to observe the generation of the pore water pressure during cyclic loading, the value of cyclic loading is defined as N_l when U is equal to the confining pressure in this paper.

The relation between the accumulated excess pore water pressure ratio U_g/σ_c and the repetition number ratio N/N_l is shown in Figure 3. When the consolidation ratio is 4, there is hardly any rise in the excess pore water pressure and N_l cannot be calculated. From this figure, it can be seen that the accumulated excessive pore water pressure U_g increases along with the increase of N/N_l, regardless of whether the normal consolidation or the over-consolidation condition exists, and that if the OCR is less than 4 and that N/N_l is unity, the accumulated pore water pressure U_g is about 80% of the confining pressure.

The relation between the stress ratio and the number of repetitions is shown in Figure 4. The relation between the consolidation ratio and the number of repetitions is shown in Figure 5. From three figures

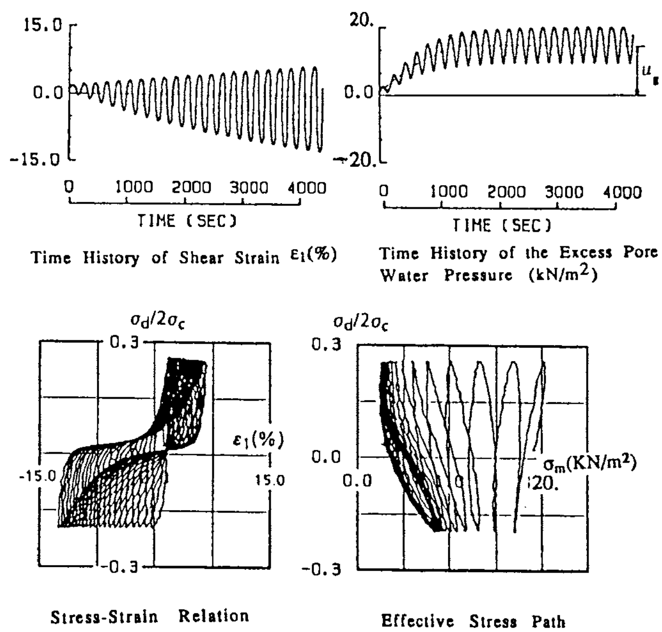

Fig.2 Typical Recoded Results of Cyclic Triaxial Test

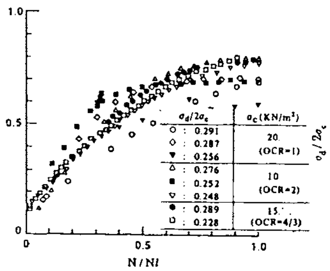

Fig.3 Excess Pore Water Pressure Generation obtained with the Dynamic Triaxial Compression Test

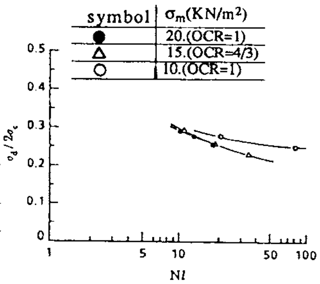

Fig.4 Relation of Over-Consolidation Ratio and Cyclic Number

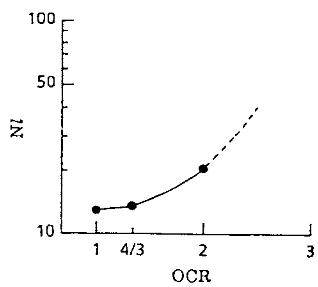

Fig.5 Relation of Excess Pore water Pressure Generation and Cyclic Number

contained Figure 3, we can see that with a decrease in the stress ratio, the number of repetitions increases, and that this tendency increases as the consolidation rate becomes higher. We can also see that the number of repetitions increases along with increases in the consolidation ratio. It is believed that the ground's degree of safety during an earthquake increases when the consolidation ratio increases.

TABLE 3 Constants of Ground and Pile

unit density(kN/cm³)	18.0
cohesion(kN/m²)	2.7
internal friction angle(degree)	26.6
constants Rf,n,K	0.63, 0.30, 70.0
constants G,F,D	0.39, 0.03, 2.56
unit density(kN/cm³)	11.5
Young modulus (kN/cm²)	2.5×10^5
thickness(cm)	0.057

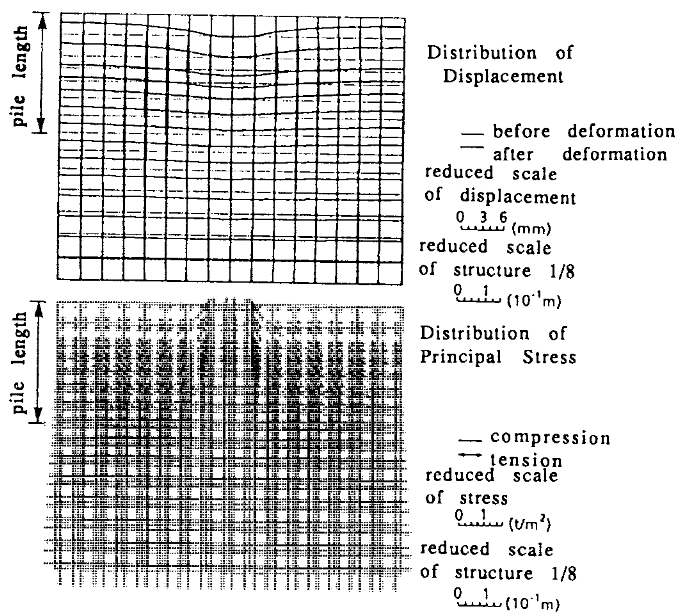

Fig.6 Results of Initial Stress Analyses

TABLE 4 Initial Principal Stress σ_{m0}(Pa) and Over-Consolidation Ratio OCR

point	σ_{m0}	OCR	point	σ_{m0}	OCR
PWP1	108	1.1	PWP7	56	2.5
PWP2	97	1.5	PWP8	54	2.9
PWP3	97	1.5	PWP9	47	3.0
PWP4	88	1.6	PWP10	43	4.3
PWP5	88	1.6	PWP11	29	6.8
PWP6	85	1.6			

TEST RESULTS AND CONSIDERATION

INITIAL STRESS ANALYSIS OF THE MODEL GROUND

To better understand the consolidation conditions of the model ground, the initial mean principal stress is calculated by static analysis. This analysis is a finite element method analysis and estimates the consolidation condition into which the frictional piles were set and, given that the friction piles are beam elements, evaluates the weight of the structure as top load weight on the piles. The nonlinear characteristic of ground elements can be approximated using the formula proposed by Duncan-Chang[12]. The constants used in the analysis are shown in Table 3. The ground constants were set by the results of physical tests and static triaxial pressure tests. As for boundary conditions, the side boundary and fixed and the bottom boundary is fixed.

The displacements and the initial stress distribution are shown in Figure 6. Table 4 shows the initial mean principle stress σ_{mo} which corresponds with the pore water pressure measuring point in the model tests and the consolidation ratio OCR. The consolidation ratio is calculated with a consolidation stress at 13.0 kN/m² This figure and table show that the effect of the ground stress on the structure varies with the depth of the piles. The consolidation ratio of the surface strata is 3.0 or more, while that of the lower strata is about 1.0, which is that of the normal consolidation condition.

STUDY OF EXCESS PORE WATER PRESSURE

Table 5 lists the maximum accumulated excess pore water pressure ratios U_g/σ_{mo} in each vibration test. This excess pore water pressure U_g was passed through a 0.2Hz low-pass filter, and the value read after the oscillation component was cut off.

The summary of results are as follows.

1) No accumulation of excess pore water pressure is noted Case 1-1 or 1-2, where the input maximum acceleration was up to 1 m/s². It is believed that no nonlinear characteristics from the excess pore water pressure occur.

2) The excess pore water pressure ratio at PWP1 is higher than that at other measuring points. This is thought to be because the consolidation ratio is near the normal consolidation condition at 1.1. In Case 1-5 and 2-2, where the maximum input acceleration was 3m/s² or more, the excess pore water pressure ratio reaches 80%. Judging from the element tests, it is thought that the excess pore water pressure almost reaches the limit. At the same time, the excess pore water pressure ratio at the other measuring points is 40% or less, and from

this it is clear that the degree of safety during an earthquake is higher in the over-consolidated condition than in the normal consolidation condition.

3) Looking at the excess pore water pressure ratio of the pile measuring points PWP7, PWP9 and others, it is clear that the excess pore water pressure between the piles is greater than in other places. In the ground between the piles, the structure load and initial shearing force resulting from the frictional force between the piles and ground act in a manner, it is thought, that easily generates excess pore water pressure.

For Case 1-4, Figure 7 shoes the pore water pressure waveforms of PWP1 at the deepest point, PWP5 in the center of the ground strata and PWP11 on the ground surface when vibration is applied. The waveforms are those that remain after the vibration component is removed by a 0.2 Hz low-pass filter.

In Figure 7, the pore water pressure at PWP1 rises in relation to the principal oscillation, and the accumulated pore water pressure, built up during oscillation, is constant after the oscillation is stopped. On the other hand, the pore water pressure at PWP5 and PWP11 do not rise conspicuously in relation to the principal oscillation. This difference is thought to correspond to the consolidation ratio described earlier.

In addition, the pore water pressure of PWP11 during the oscillation rises with an almost constant gradient, different than the phenomenon observed PWP1. In connection with this, Figure 8 shows the changes over time of the excess pore water pressure at PWP1 and PWP11 in Case 2-2, starting with the time that oscillation begins until the water pressure is dispersed. The vertical axis in the figure is the pore water pressure ratio, and the waveforms during the oscillation up to 100 seconds is the measuring point.

It can be seen from figures that the pore water pressure, after the vibration, is great near the ground surface, PWP11, and that the period of stability for the pore water pressure at PWP1, the deepest point, is long, and takes longest for the pore water pressure to disperse. As a

TABLE 5 Accumulated Excess Pore Water Pressure (U_g/σ_{m0})

point \ Case	1-1	1-2	1-3	1-4	1-5	2-1	2-2
PWP1	0.00	0.00	0.09	0.36	0.86	0.09	0.81
PWP2	0.00	0.00	0.05	0.16	0.37	0.05	0.38
PWP3	0.00	0.00	0.06	0.17	0.47	0.05	0.38
PWP4	0.00	0.00	0.10	0.22	0.21	0.06	0.14
PWP5	0.00	0.00	0.08	0.20	0.29	0.05	0.40
PWP6	0.00	0.02	0.09	0.15	0.30	0.04	0.41
PWP7	0.01	0.11	0.22	0.25	0.29	0.11	0.47
PWP8	0.00	0.03	0.05	-	-	0.02	0.10
PWP9	0.04	0.11	0.35	0.38	-	0.22	0.35
PWP10	0.04	0.14	0.09	0.11	-	0.09	0.28
PWP11	0.04	0.05	0.02	0.11	-	0.14	0.30

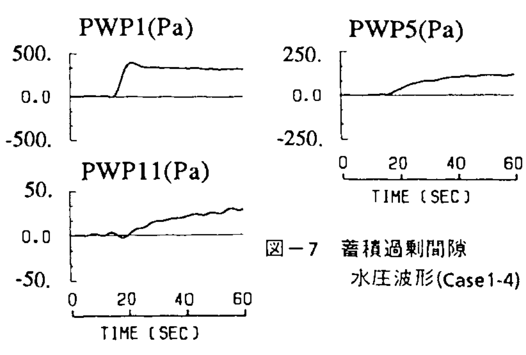

Fig.7 Time History of Accumulated Excess Pore Water Pressure(Case 1-4)

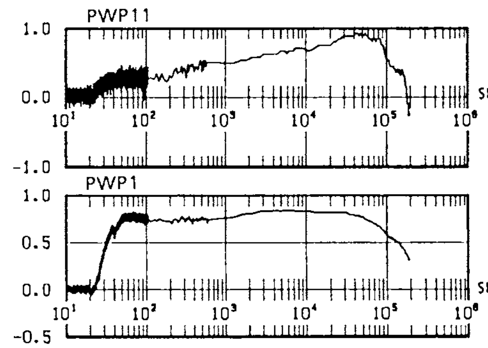

Fig.8 Time History of Excess Pore Water Pressure(Case 2-2)

result, it is believed that the pore water pressure dispersion starts near the ground surface, and that the water pressure rises from the inner ground strata toward the surface of the ground,

STUDY OF ACCELERATION RESPONSE

For both Case 1-1, which is in an almost linear state, and Case 2-2, which has a high degree of nonlinear characteristics, Figure 9 shows the acceleration waves and Fourier spectrals for both the ACC5 at the ground surface and ACC2 at the top edge of the structure model. It is observed that as input acceleration exceeds the waveform, the frequency component decreases. And it is further understood from the spectrals that there is no overall tendency for a strong relation between the predominant frequency of the ground and that of the structure in Case 1-1. In Case 2-2, the predominant frequency of the ground and that of the structure almost correspond to each other.

These are thought to be the effects of the nonlinear characteristics of the ground. To describe this in detail, the transfer functions related to the input acceleration ACC6 of Case 1-1 and 2-2 are shown in Figure 10.

The points which can be drawn from the figures are as follows.

1) In Case 1, the peaks of predominant frequency can be observed as 17Hz and 34Hz at ACC 5 on the ground surface. These are the primary and secondary natural frequencies of the ground. The ground has the same frequency characteristics as the foundation of the structure, but at the upper edge of the structure, the characteristics of the structure's components dominate, and thus it is thought that the effects of the ground are small.

2) In Case 2-2, the predominant frequency of ACC5 is around 10 Hz. Hence there is a tendency to shift toward a lower frequency in comparison to Case 1-1. The foundation of the structure has the same frequency characteristics as the ground. This phenomenon is the same as in Case 1-1. Thus, the input motion from the foundation structure is considered as being supported by the ground. As in Case 1-1, peaks at the top edge of the structure in Case 2-2 are around 22 Hz, but their magnification is smaller, and the effect on the ground is relatively greater than that of Case 1-1.

STUDY OF THE DISPLACEMENT RESPONSE

In Figure 11, the horizontal and vertical displacement waves of the structure are shown. It is observed from these figures that residual displacement is generated in the horizontal and the vertical planes by the principle oscillation of the earthquake, and that while the settlement of the structure in the up and down directions is taking place, a rocking motion occurs. Next, the amount of settlement occuring in the ground and top of the structure in Case 1-3, 1-4 and 2-1 is shown in Table 6. Compared to the amount of settlement on the ground, the amount of the settlement of the structure is quite large. It is believed that the settlement of the stricture is larger because the accumulated pore water pressures between the piles are larger than that of the natural ground as above mentions.

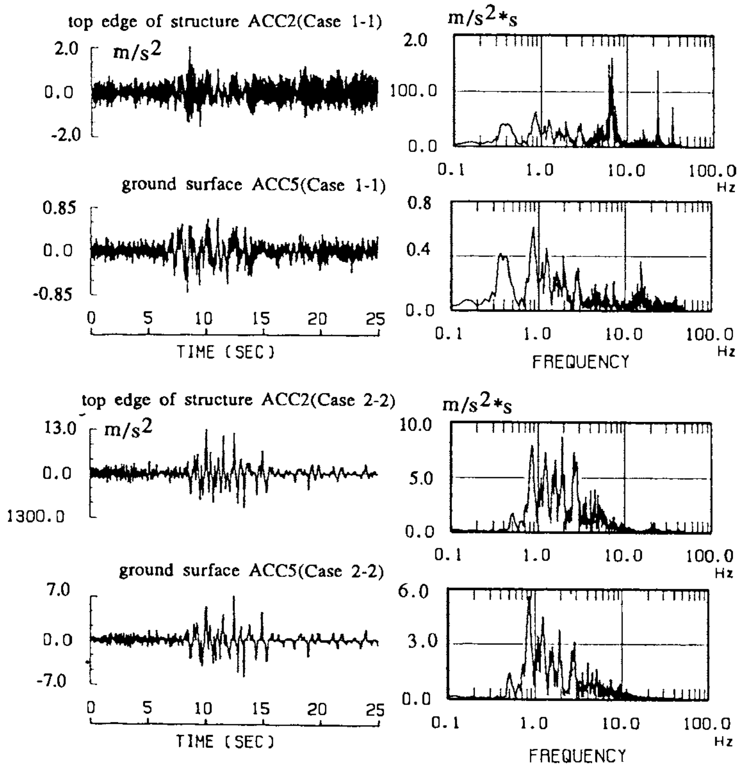

Fig.9 Waves of Acceleration and Fourier Spectals

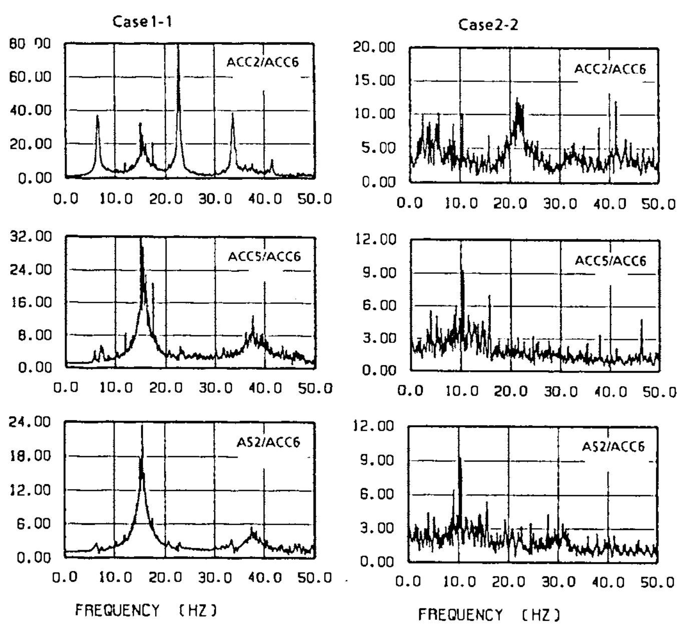

Fig.10 Frequency Transfer Function

TABLE 6 Residual Displacement(cm)

Case / measuring point	1-3	1-4	2-1
DP2(ground)	0.04	0.11	0.16
DP3(structure)	0.24	0.46	0.33
DP4(structure)	0.10	0.39	0.21

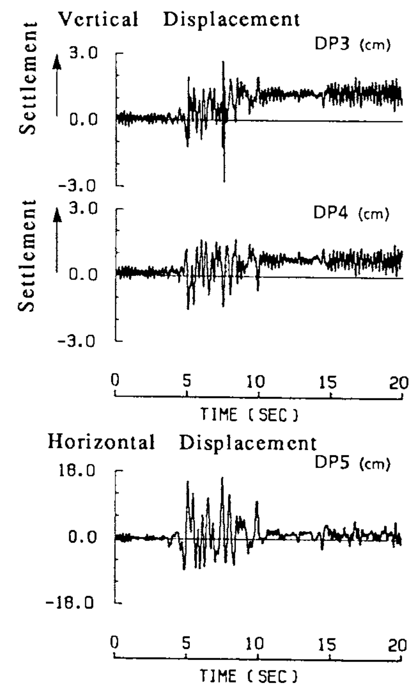

Fig.11 Waves of Structure Displacement(Case 2-1)

CONCLUSIONS

In this paper, we have described an experimental study of the model test with a shaking table. to investigate the dynamic behavior of soft clay ground-structure supported by friction files foundation The main results are summarized as follows.

1) In cyclic loading tests with kaolin clay, regardless of whether a normal consolidation condition or over-consolidation condition was used, the excess pore water pressure increased when subjected to repetitive stress, but the accumulated excess pore water pressure is approximately 80% of the value of the confining pressure.

2) After cyclic loading tests and shacking table tests, it is believed that the safety of the over-consolidated strata during an earthquake is high. The accumulation of the pore water pressure varies according to the consolidation condition of the strata. A pore water pressure rise can be observed in ground with normal consolidation when exposed to principle vibration, but the rise in pore water pressure in over-consolidated condition in relation to the principal vibration is not conspicuous.

3) Excess pore water pressure is easily generated in the ground between the piles due to the shear stress resulting from the friction generated by the structure load.

4) Due to the softening of the ground that accompanies the accumulation of excess pore water pressure, the high frequency component of the ground response is decreased, and the predominant frequency shifts toward a low frequency. In the structure response, the predominant frequency is the same frequency as it was in the linear response, but in comparison with the linear response the magnification is small, and the effect on the ground is large. The input force to a structure with a foundation structure is supported by the ground.

5) Residual deformation in a structure is generated by the principal force of an earthquake in both horizontal and vertical planes. Also, compared to the amount of residual settlement on the ground, the amount of settlement within the structure is quite large.

It is concluded that softening of clay ground and generation of the excess pore water pressure due to cyclic loading of earthquakes should be examined. The study should be continued and the test results simulated with numerical analyses.

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