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SYNOPSIS: The basic policy of Japan is to build nuclear reactor building on bed rock. But, in order to cope with the middle and long term siting problems it has become necessary to promote new siting technology from the standpoint of expanding the available range of site selections and effective utilization of land. The large scale field tests were conducted at the Tadotsu Engineering Laboratory, Kagawa Prefecture, of The Nuclear Power Engineering Test Center (NUPEC) as an entrusted project into the Siting Methods on Quaternary Deposits from the Ministry of International Trade and Industry (MITI), in order to verify the seismic stability of soil appertained to siting technology on quaternary deposits. Described in this paper is the results of the tests.

1. INTRODUCTION

The investigation of siting technology on quaternary sand and gravel deposits has been implemented to establish the methodology for the seismic design of the nuclear power plants to be constructed on the quaternary deposits in Japan as follows:

(1) Soil seismic stability verification tests using large-scale models constructed in field were conducted in which large shear strains equivalent to a shaking during a strong earthquake was induced on actual quaternary deposits.
(2) For the research regarding the seismic safety evaluation methods, a rational analysis method regarding siting of the nuclear power plants on quaternary deposits have been investigated and studied.
(3) For the feasibility study of the building and associated components, reviews have been conducted of the conceptual plant design that is appropriate for quaternary deposits siting and of the plant facilities concerning seismic safety.
(4) In general assessment, a seismic design guideline (draft) for application to the quaternary deposits siting will be prepared.

The investigation of the siting technology reported herein is entrusted project to NUPEC from MITI, and has been executed under the cooperation of academic and industrial groups.
The planning on soil seismic stability verification test was commenced in 1983, and large scale field tests were conducted from 1987 to 1988 at the Tadotsu Engineering Laboratory, Kagawa Prefecture, Japan, of The Nuclear Power Engineering Test Center (NUPEC).
For the field testing, two soil column models with the same 10m diameter but with different depths of 5m and 9m, a concrete block model weighing 3,000t with earth contact pressure equivalent of actual reactor building, and a reaction block weighing approximately 5,000t were built, and the verification of soil seismic stability was executed by dynamic and static loading tests. In succession from 1989, simulation analyses have been conducted by using data obtained from the field tests for establishing the seismic design methods for quaternary deposits siting.
This report presents the method of field testing and their results.

2. TEST PLAN

2.1 Objectives of tests

The objectives of the soil seismic stability verification tests are, for one thing, to obtain public acceptance by verifying the soil stability during large earthquakes by conducting field tests using large scale models that are made to have the condition as close as possible to an actual nuclear reactor building. Also, by means of the field verification tests the adequacy of the evaluation method for seismic safety of the quaternary soil siting and its rationality is confirmed. Moreover, to supplement the field tests, laboratory tests simulating the seismic input were performed using scaled-down models. The objectives and targets of the soil seismic stability verification tests are shown in Table 2.1.

2.2 Selected soils for tests

For the test deposit a diluvium gravel layer was chosen which has high possibility of becoming the bearing soil stratum when building a nuclear power plant on the quaternary deposits. As to the site for conducting the field test the ground within the premises of the Tadotsu Engineering Laboratory, in Kagawa

Prefecture, Japan, of NUPEC was selected. Because there was about 10m thick reclamation layer of dredged material on top of the selected test gravel soil, the ground was excavated to a depth of 11m for removing the near-surface soil and exposing gravel base to mount the test models. The arrangement plan of the test field and the test models are shown in Figs. 2.1 and 2.2. The underground water table was lowered by using a pump and controlled to remain at -1.5m from the excavated ground surface.

2.3 Test models

(1) Soil column model

The objectives of the soil column models are to investigate as natural ground condition as possible the dynamic soil properties in small strain levels and the deformation characteristics at large shear strain levels. The size of the soil column models was made as large as possible so that the non-homogeneity of the soil should not give large influence.
Such a test, in which the field soil is subjected to torsional loads, is the first attempt of this kind. The method of constructing the soil column models, its dimensions and the loading method were therefore determined by preliminary studies. As a result of considering three factors, namely, space for setting up two exciter sets, the loading moment produced by the load central distance between the two sets, and the shear strain of the soil column model obtainable by the loading moment, the two soil column models with the same diameter of 10m and different height of 9m and 5m were determined.
For the 9m depth model, the procedure of the test was first to apply the very small shear strain of 10^{-5} level in the dynamic phase of loading test and then, to perform a static phase of test by using a hydraulic jack and study the deformation characteristics under the large shear strain of 10^{-3} level. For the 5m depth model, only the dynamic test was performed to study the soil behavior in the intermediate shear strain of 10^{-4} level.
A concrete capping block was mounted on top of the soil column as a surcharge and to facilitate transfer of dynamic load from the exciter and static load from the hydraulic jacks. The thickness of the capping block was determined to be 3m (overburden pressure 7.2 t/m2) for the 5m depth model, and 5m (overburden pressure 12 t/m2) for the 9m depth model. The conditions for the determination were set such that (1) the predetermined shear strain could be applied in the soil column model (2) the overburden pressure prior to excavation was maintained and (3) the overburden pressure is large enough to prevent slippage between the capping block and the soil model. Also, a cross shaped shear key of 1.5m depth and 1m width was placed under the capping block to prevent slipping during the loading test.
The soil column model was constructed by excavating a ring-shaped trench 80cm in width and surfaces of the trench walls were re-faced by placing mudwater cement mortar of 20 - 30cm thickness. This mudwater mortar was mixed with bentonite providing a lean-mix in order to keep low stiffness so that the shearing stiffness of the soil column model will not change. Also, this mudwater mortar has the role of preventing drainage around the soil column model so that the excessive pore water pressure will not dissipate during the testing.
Twenty-seven rectangular rubber bags 140 cm and 110 cm wide, 16 cm thick and 5m or 9m long connected by steel frames were lowered into the ring-shaped trench and inflated with water whose pressure was maintained equal to the stepwised over burden pressure within the soil column according to the construction of the capping block. In this way, the test soil column was supported laterally by flexible bags without any restriction in developing shear deformation during the tests.

(2) Concrete block model and reaction block

The objectives of the concrete block model are to investigate the soil stability regarding sliding, settlement and inclination by means of the static loading tests at the large shear strain of 10^{-3} level under the model condition of earth contact pressure of approximately 50t/m2 which is similar to actual reactor building, and to collect data relevant to the soil-structure interaction by means of the dynamic loading tests. The concrete block model was designed to have a dimension in plan 8m x 8m at the lower part and 12m x 12m at the upper part as shown in Fig. 2.2. In this concrete block, a contact pressure of approximately 50t/m2 could be attained by the model of 10m height.
As a reaction block, a rectangular concrete block with a dimension of 16.5m x 16.5m x 8m was constructed beside the concrete block, as shown in Fig. 2.2. This block was also used for the vibration tests by exciting it with a pair of exciters mounted thereon.

2.4 Loading device

Firstly, a pair of exciters to be used for the dynamic loading tests was searched and the equipment satisfying two conditions below were selected.
(1) Two units with the same performance capacity in horizontal excitation, and with device to control the excitation phase between the two.
(2) Large excitation loading is available and has the range from low to high frequency.
Each of the selected exciter possessed the capacity; maximum eccentric moment of 630kg.m, maximum excitation of 10ton, excitation frequency of 0.2Hz to 20Hz, and with plan dimensions of 2.2m x 3.7m. Table 2.2 shows the specification and Fig. 2.3 shows the external view of the exciter.
Next, a pair of hydraulic jacks to be used for the static tests was selected from the standpoint of the size of the soil column model and concrete block model and the target shear strain.
The specification of the hydraulic jack is as shown in Table 2.3. Maximum loading is 600 ton, strokes of $\pm$150mm and weight of 3.1 ton. The external view of the hydraulic jack is shown in Fig. 2.4.

Fig. 2.1 General View of the Field Test Model

Table 2.1 Objective and Target of Soil Seismic stability Verification Tests

Test Item		Verification Item	Target Value of Maximum Shear Strain	
Soil Column Model Test	Static Load	o Evaluation of Soil Physical Property o Stability of Soil at Large Shear Strain Level by Large Scale Test	5m Model	9m Model
			-	Approx. 10^{-3}
	Dynamic Load	o Evalnation of Soil Physical Property	Approx. 10^{-4}	Approx. 10^{-5}
Concrete Block Test	Static Load	o Stability of Foundation Soil at Earth Contact Pressure of Actual Plant Level	Concrete Block	Reaction Block
			Approx. 10^{-3}	-
	Dynamic Load	o Interaction of of Structure and Soil	Approx. 10^{-4}	Approx. 10^{-5}

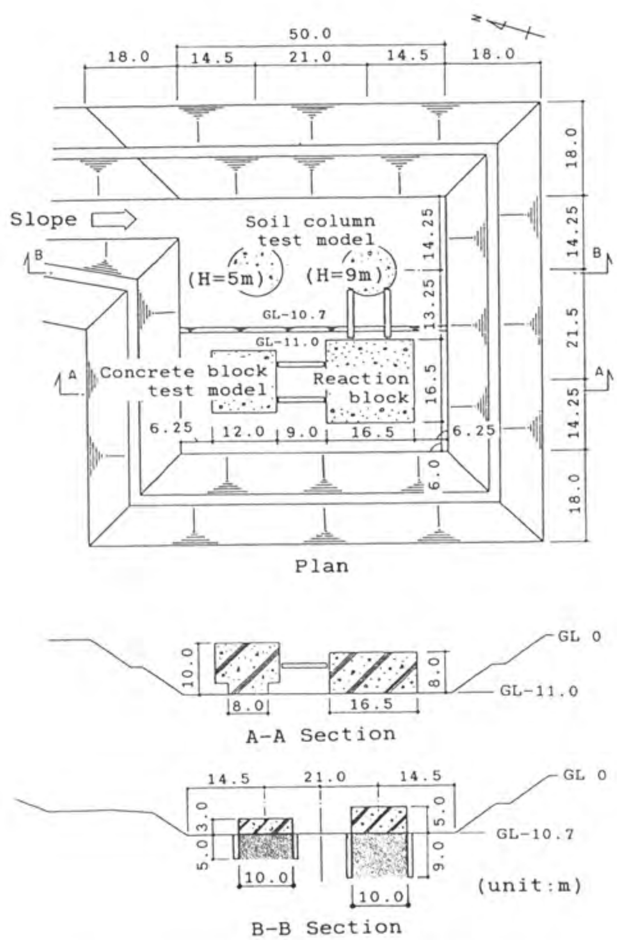

Fig. 2.2 Layout of the Field Test Model

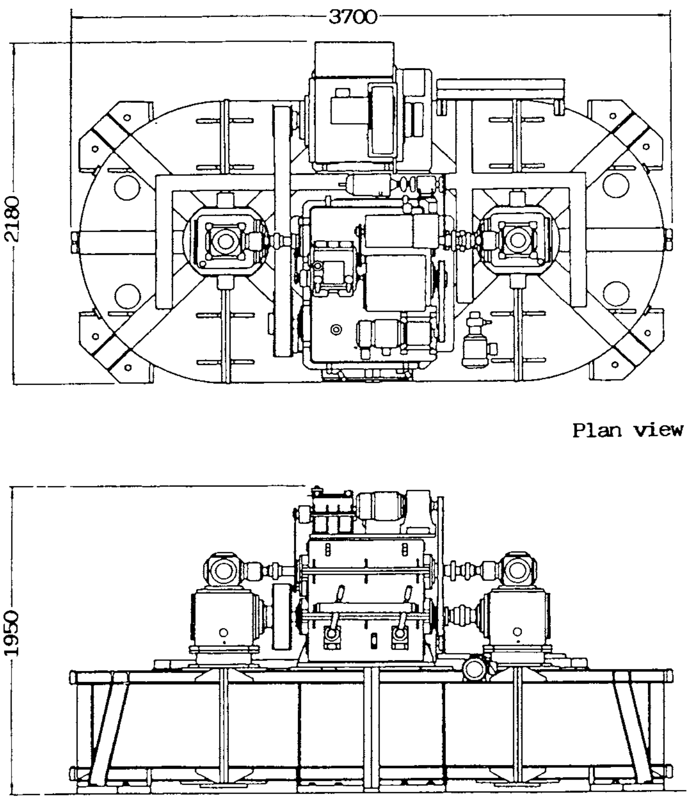

Fig.2.3 External View of Exciter

TABLE 2.2 Horizontal Exciter Specification

Exciting mechanism	2 eccentric masses in synchronous coplanar rotation
Exciting waveform	Sinusoidal
Maximum exciting force	10,000 kg.f (2 Hz or more)
Eccentric moment	0.63 ~ 630 kg.m (low and middle ranges) 0.26 ~ 260 kg.m (high range) Successive variable operation possible
Accuracy of moment setting	± 0.1 % of maximum value
Range of vibration frequency	Low range : 0.2 ~ 2.0 Hz Middle range : 0.8 ~ 8.0 Hz High range : 2.0 ~ 20 Hz
Accuracy of vibration frequency setting	Below ± 0.5% of maximum vivration frequency of each range
Accuracy of synchronous operation	Below 1 degree at earch range (using synchronous BL motor)
Dimensions	Width 3,700 mm Depth 2,180 mm Height 1,950 mm
Weight	Approx. 10 t
Power supply	200V,3 Φ ,70kVA

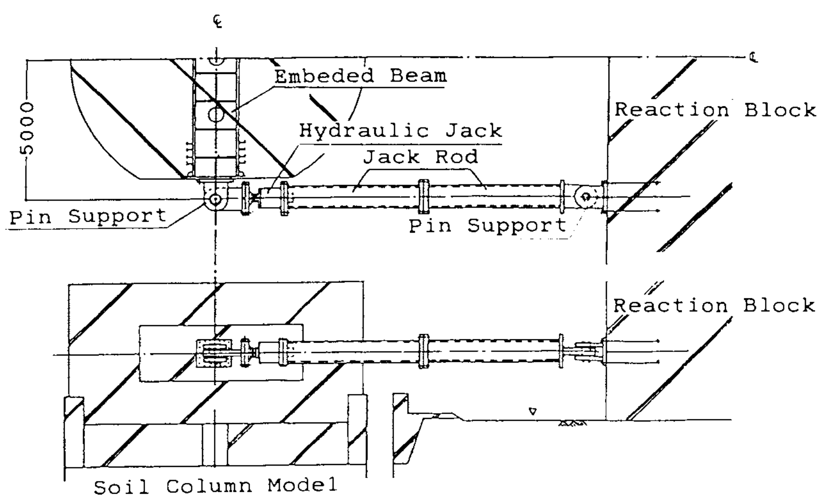

Fig.2.4 External View of Hydraulic Jack

Table 2.3 Hydraulic Jack Specification

Operating Maximum load	600ton
Maximum load	750ton
Stroke	±150mm
Operating Pressure	477.5kg/cm^2
Required oil amount	37.7ℓ
Weight	3.1ton

3. SOIL INVESTIGATION

3.1 Purposes of investigation

The purposes of the soil investigation are to clarify the engineering characteristics of the soil at the test site through depth of 11m to 20m, and to incorporate them into the evaluation analysis of the field test results.

3.2 Investigation items and methods

The major goals of investigation were to identify, (1) composition of the soil layers, (2) physical property of gravel layer, and (3) mechanical characteristics of the gravel layer. The specific investigation methods for the above items were; (1) boring, standard penetration test, large scale penetration test, elastic wave propagation test and plate loading tests, (2) grain size distribution analysis of soil and test for density of soil particles, and (3) static drained triaxial test, dynamic deformation test, dynamic strength test, test for obtaining volume change characteristics during cyclic shear and, isotropic compression and isotropic expansion tests.

3.3 Major investigation and results of measurements

As indicated in Section 3.2, the items of investigation and tests cover a wide scope, however, here only the major results are reported due to the limited number of pages.

(1) Soil profile of the test field

Shown in Fig. 3.1 are; typical soil layer composition of the test field, penetration resistance value (N and N_L) of the standard penetration test and the large scale penetration test, and the depth distribution of the shear wave velocity obtained by the down-hole method. The test site is composed of reclaimed soil of dredged material 11m thickness from the surface, and diluvial gravel layer at depth 11m to 20m. This gravel layer has Vs=380m/sec with N values within the range of about 40 to 50. The value of N_L in this gravel layer is between 15 - 40 and reflects the variation of the stiffness in the gravel layer.
Fig. 3.2 shows the plot of the relation between the values of N and N_L. From Fig. 3.2, the correlation between N and N_L of the reclaimed soil is not found. When sand layer is omitted, the data obtained is distributed between N_L = N and N_L = 1/3N, and correlation between the two can be acknowledged. By accumulating such data, the N_L value obtained by the large scale penetration test will become useful in the future for identifying the characteristics of gravelly soil.
For conducting in-situ verification tests, the 11m-thick reclaimed surface material was excavated and removed, and then the test models were built. In order to see what influence the excavation will exert on the mechanical property of the gravel layer, the standard penetration test and elastic wave propagation test by down-hole method were conducted again after the excavation. The results are shown in Fig. 3.3. It is clearly seen that the N value and the shear wave velocity have decreased due to the excavation. Regarding the influence of excavation on the soil property, it is considered complicated by factors such as, reduction in overburden pressure, rotation of principal stress and change in density. However, an attempt is made herein to explain it quantitatively by the relations excessed by Equations (1) and (2) as follows. These relations are based on assumption that the coefficient of earth pressure at rest, Ko, is invariable, and that the change in soil property is mainly due to the decrease in overburden pressure.

$$N,a = N,b \, \frac{\sigma v',a + 0.7}{\sigma v',b + 0.7} \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

$$Vs,a = Vs,b \left(\frac{\sigma v',a}{\sigma v',b}\right)^{1/4} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

herein
- N,a : N(after excavation)
- N,b : N(before excavation)
- Vs,a: Vs(after excavation)
- Vs,b: Vs(before excavation)
- σv',a: σv'(after excavation)
- σv',b: σv'(before excavation)

Equation (1) is a correction to N-value regarding overburden pressures which is based on Meyerhof's formula (Meyerhof, 1957). Equation (2) is a similar correction related to Vs, derived from the experimental formula by Hardin and Richart (1963). From Fig. 3.3, the change in N value and Vs due to excavation can be generally explained by the decrease in the effective overburden pressure.

(2) Extraction of undisturbed gravel sample

It is a well known fact from recent research results that the soil property obtained by laboratory tests are influenced largely by sampling method (Yoshimi et al, 1984; Hatanaka et al, 1988). Therefore, the in-situ freezing sampling method which has received high valuation in recent years regarding extraction of undisturbed sample of sandy and gravel soils was applied. According to the method shown in Fig. 3.4, 20 samples each 30cm in diameter and 60cm long, were extracted. Fig. 3.5 shows a photograph of a sample. The samples have average grain size of 3 - 17mm, with a maximum grain size of 70 - 150mm. Fine content is 0 - 3%, and dry density is 1.81 - 2.04t/m^3, and specific gravity is 2.64 - 2.65.

(3) Dynamic properties

1) Dynamic strength characteristics

The dynamic strength characteristics obtained by cyclic undrained triaxial test is shown in Fig. 3.6. For comparison, the test results on undisturbed and reconstituted samples are shown. From the figure it can be seen that the undisturbed samples show a high value of cyclic shear stress ratio of 0.44 required to cause a double-amplitude axial strain of 2.5% in 20

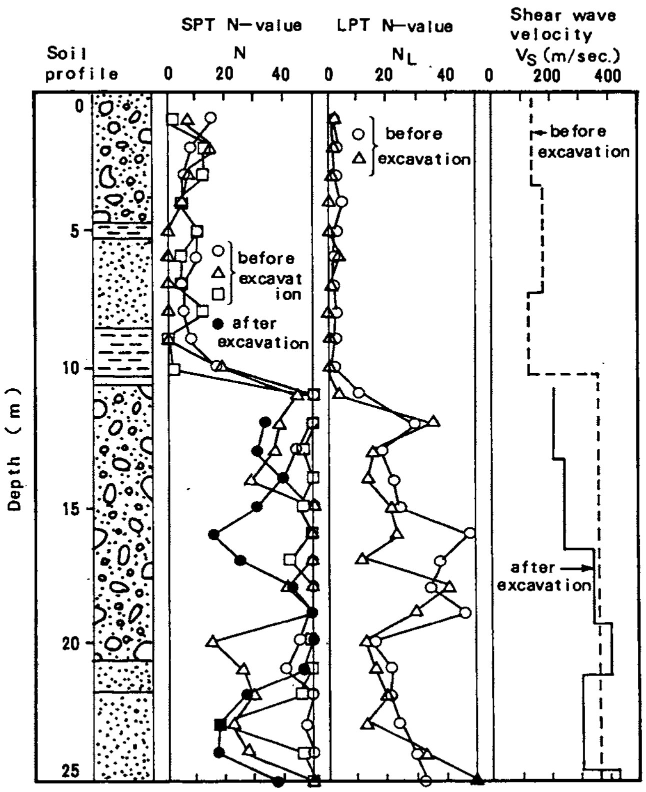

Fig. 3.1 Soil profile of test field

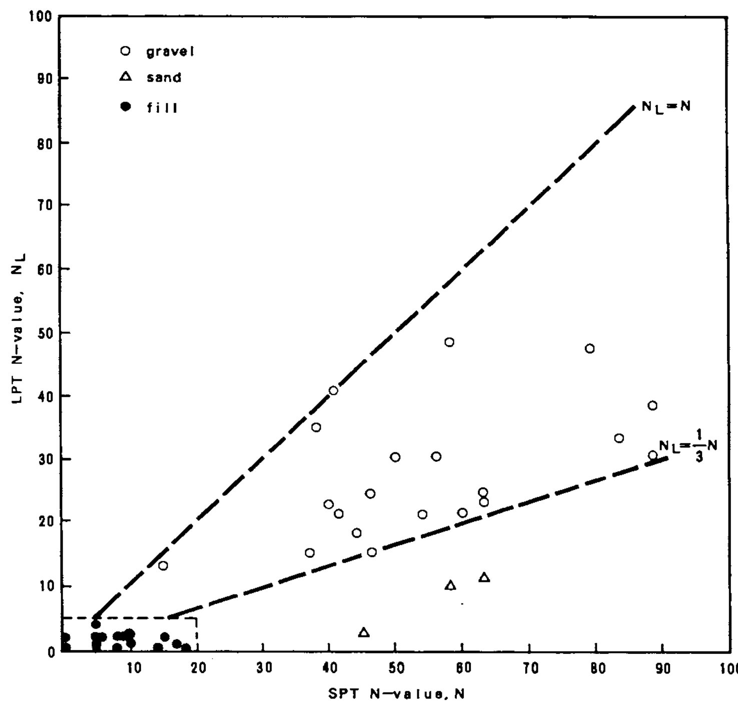

Fig. 3.2 Relationship between SPT N-value, N, and LPT N-value N_L

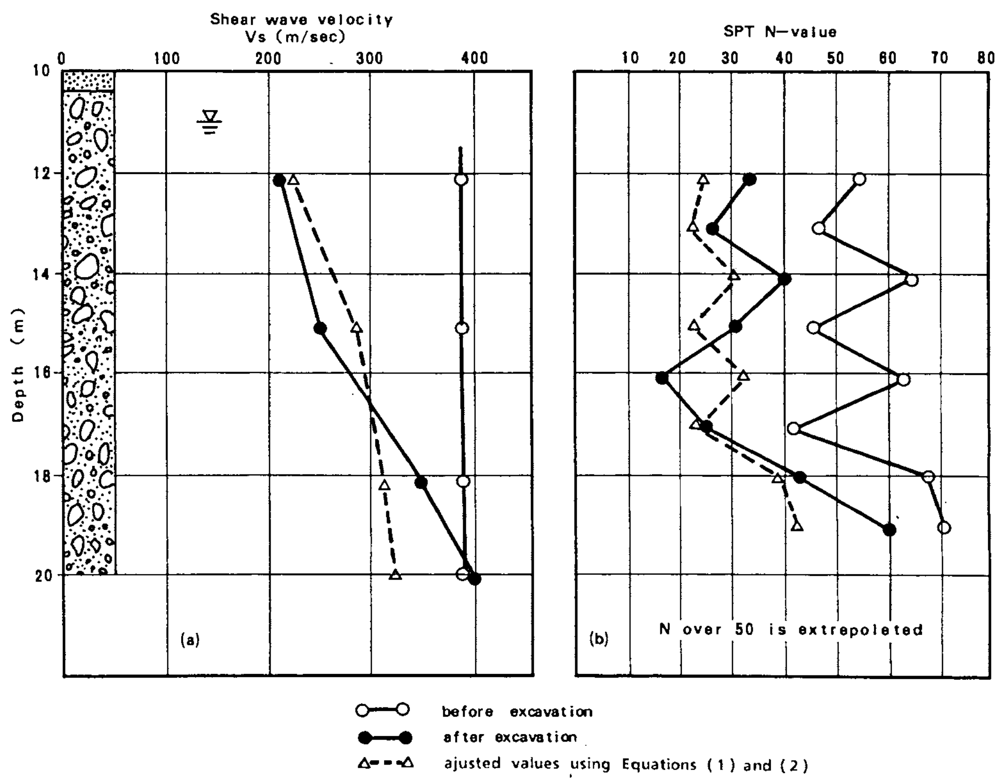

Fig. 3.3 Effect of overburden pressure on the SPT N-value, N, and shear wave velocity, Vs

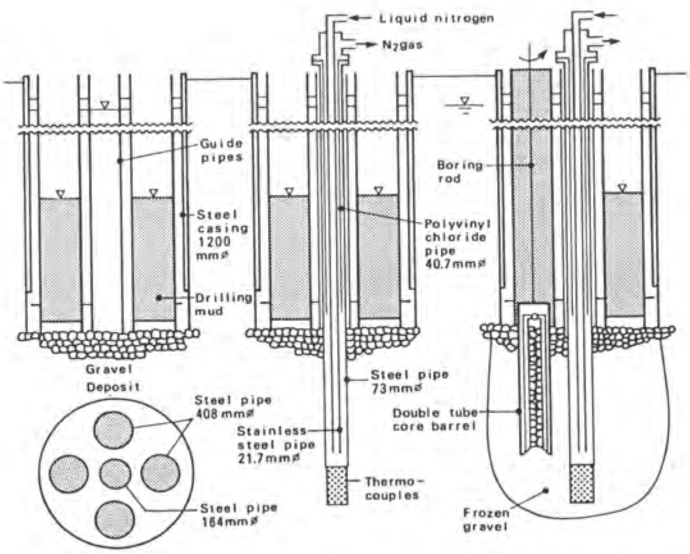

Fig. 3.4 Schematic drawing of in-situ freezing sampling method

Fig. 3.5 Undisturbed frozen gravel sample (30cm in diameter, 60cm high)

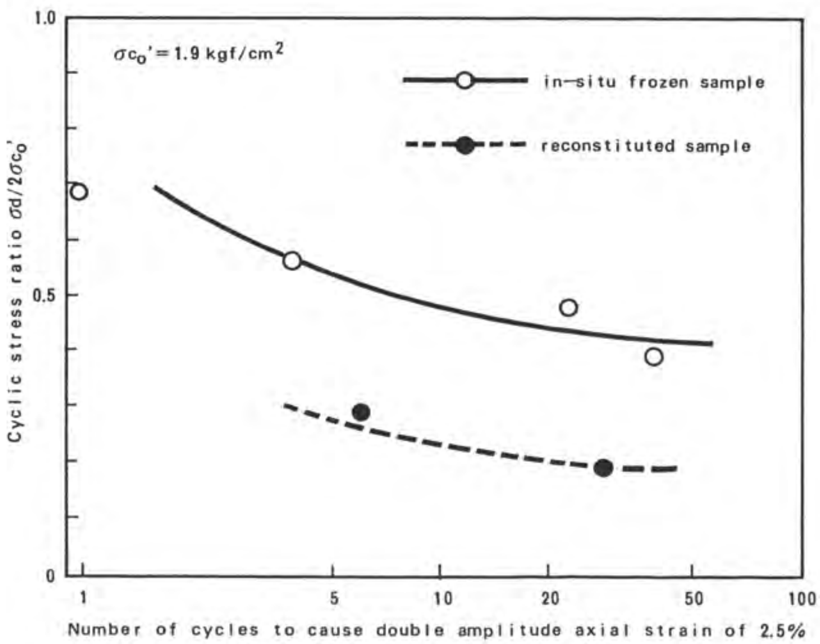

Fig. 3.6 Undrained cyclic strength of gravel

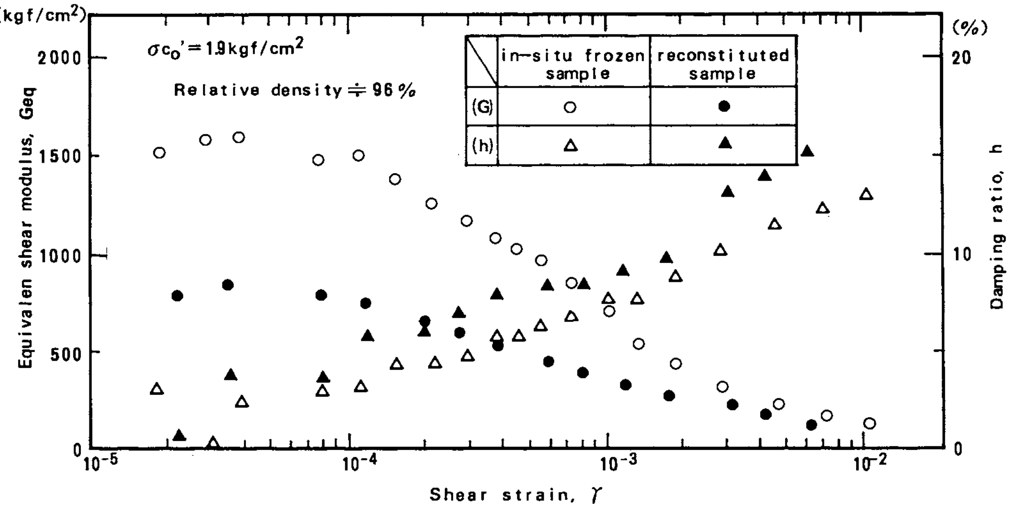

Fig. 3.7 Geq ~ γ, h ~ γ relationships

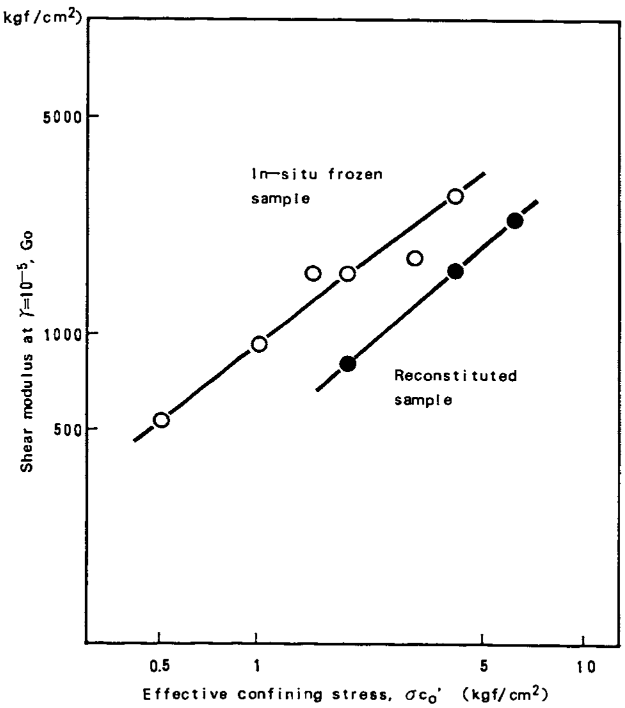

Fig. 3.8 Relationship between Go and $\sigma co'$

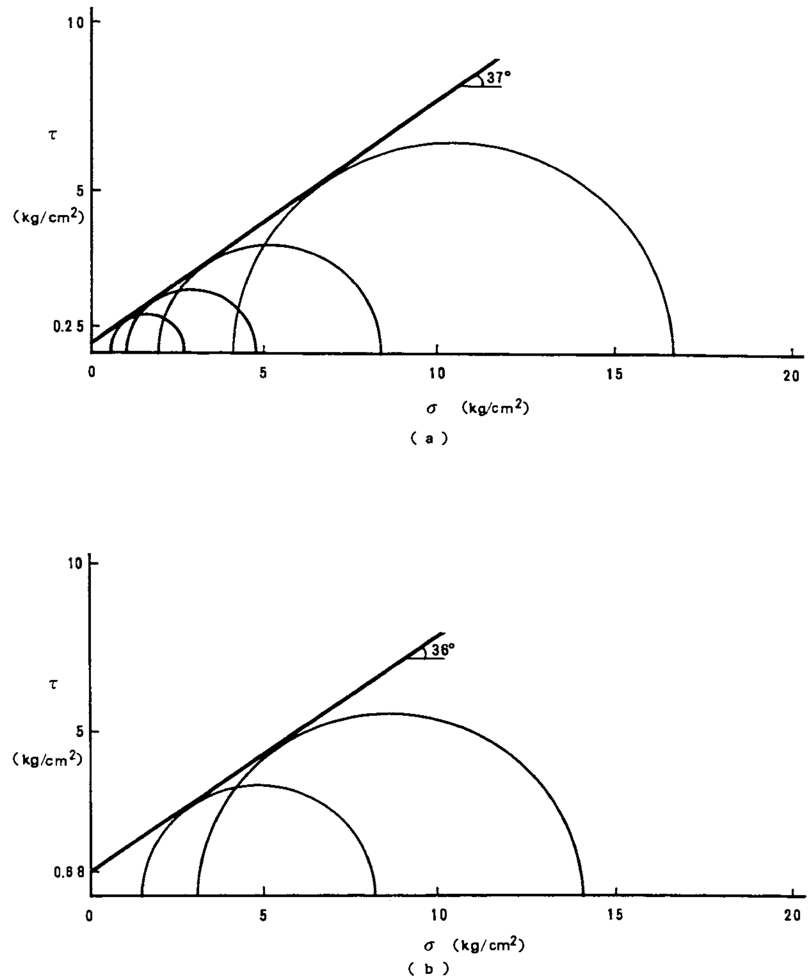

Fig. 3.9 Mohr's circle and yield locus enveloping the Mohr's circles

cycles of load application. The strength of reconstituted samples is only about one half of that of the undisturbed samples, and that the strength of the in-situ gravel soil cannot be correctly evaluated using the reconstituted soil.

2) Dynamic deformation characteristics

The dynamic deformation characteristics obtained by cyclic undrained triaxial test is shown in Fig. 3.7. For comparison, the result of reconstituted sample is also shown. The shear modulus, G, obtained by reconstituted samples is only about one half of that of the undisturbed samples, and as in the case with strength evaluation, it indicates that the deformation characteristics of the in-situ gravel soil cannot be evaluated by reconstituted samples. Fig. 3.8 shows the relation of the shear modulus, G, at very small strain and the overburden pressure, and on both logarithm coordinate the straight line relation is almost observed. The slope of their straight line is about 0.8, which is larger than the value of 0.5 commonly known for sand.

(4) Static characteristics

The Mohr circle at failure obtained by drained triaxial tests is shown in Fig. 3.9. The internal friction angle is 36 - 37 degrees, and cohesion is 2.5 - 6.8t/m2. The internal friction angle is quite smaller than the value estimated from N value using the empirical formula for sandy soils.
In contrary, there is some cohesion component that cannot be neglected.

4. DYNAMIC LOADING TEST OF SOIL COLUMN MODEL

4.1 Loading method

For the dynamic loading tests of soil column model, the two exciters mounted on top of the loading block were operated in opposite phase and torsional moment was applied to the capping block.
For the 5m depth soil column model the load application was conducted in 5 stages with different load intensity, 10t.m, 20t.m, 30t.m, 40t.m and 50t.m. Of these, for 10t.m, 30t.m and 50t.m tests the two manners of frequency change were employed, that is, in each of the tests with different levels of load intensity, the frequency was increased stepwise in one type of test (load pattern I) whereas the frequency was decreased in another type of test (load pattern D). For the tests of 20t.m and 40t.m, only the load pattern (I) was employed.
For the 9m depth soil column model the load intensity was varied in 3 stages, 5t.m, 10t.m and 15t.m, and two patterns of (I) and (D) were employed.

4.2 Measurement method

The measuring instruments used were servo-type accelerometers and pore water pressure gauges, and these instruments for the soil column test were buried in the boring holes prior to constructing the capping block.
The locations of the measuring instruments for the 5m depth soil column model are shown in Fig. 4.1, and those for 9m depth model in Fig. 4.2.
The accelerometers were installed to measure the response accelerations in the tangential direction of the capping block and the soil column model and to obtain the natural resonance frequency and corresponding mode of motion indicating the resonance curve. The average shear strain was calculated by integrating the acceleration twice and dividing the relative displacement of the soil column model by the layer height.
The pore water pressure gauges were installed, so that the dynamic fluctuation of the excessive pore water pressure could be measured, and the existence of dilatancy could be investigated in the soil column model.

4.3 Results of measurement

(1) 5m depth Soil column model

Fig. 4.3 shows the mode in acceleration in the tangential direction at the natural resonance frequency which was obtained from the test employing the load pattern (I) with 50 t.m load intensity. The capping block behaved uniformly as a rigid body in torsional mode.
The acceleration of the soil column model decreased from the top towards the lower part.
The resonant curves indicated clearly defined the peak and the phase angle changed smoothly from 0° to 180° and showing a value of about 90° at the resonant point. With increasing excitation force the resonance frequency becomes lower accordingly indicating decrease in stiffness of soil column.
In the 50t.m load intensity test, a shear strain of 1×10^{-4} was observed at the resonant point and achieved the target value of the test.
The excessive pore water pressure fluctuated at approximately twice the excitation frequency at the resonant point and effects of dilatancy appeared. However, accumulation of excessive pore water pressure was not observed.
From the test results it was found that stiffness of the soil column decreased and the damping increased with increasing load intensity and the excessive pore water did not accumulated at the shear strain level of 10^{-4}.

(2) 9m depth soil column model

Fig. 4.4 shows the mode in acceleration in the tangential direction at natural resonance frequency for the load pattern (I) with 15 t.m load intensity. The capping block behaved uniformly as a rigid body in torsional mode.
The acceleration of the soil column model decreased from the top towards the lower part.
The resonant curves indicated clearly defined the peak and the phase angle changed smoothly from 0° to 180° and showing a value of about 90° at the resonant point. The shear strain at the resonant point showed 2.6×10^{-5} satisfying the strain level of 10^{-5} which was the target value of the test.

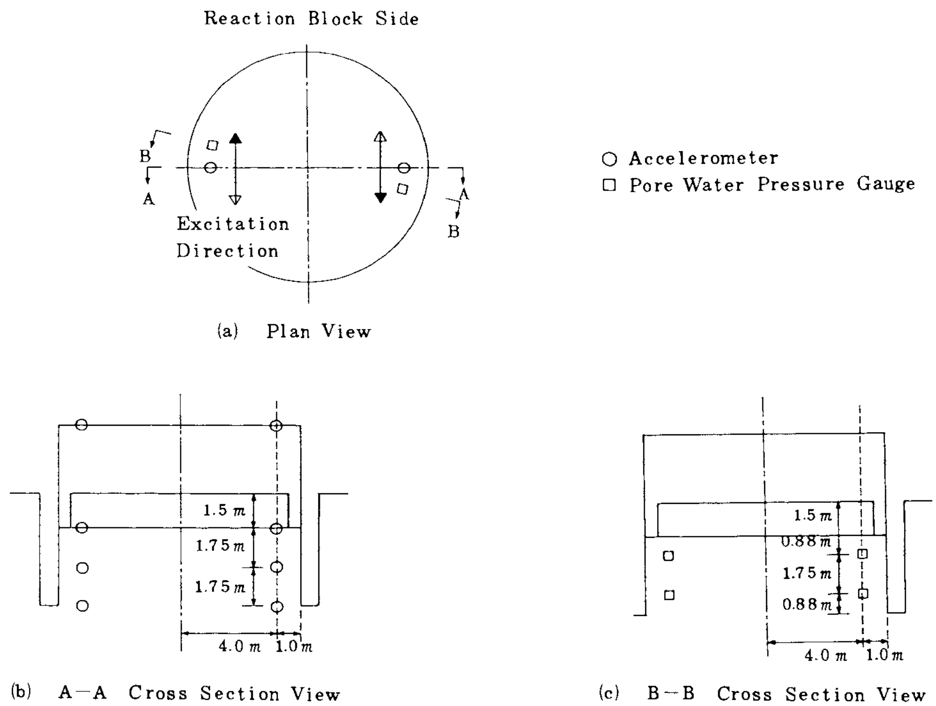

Fig. 4.1 Measuring Instrument Arrangement for Dynamic Load Test of 5m Soil Column Model

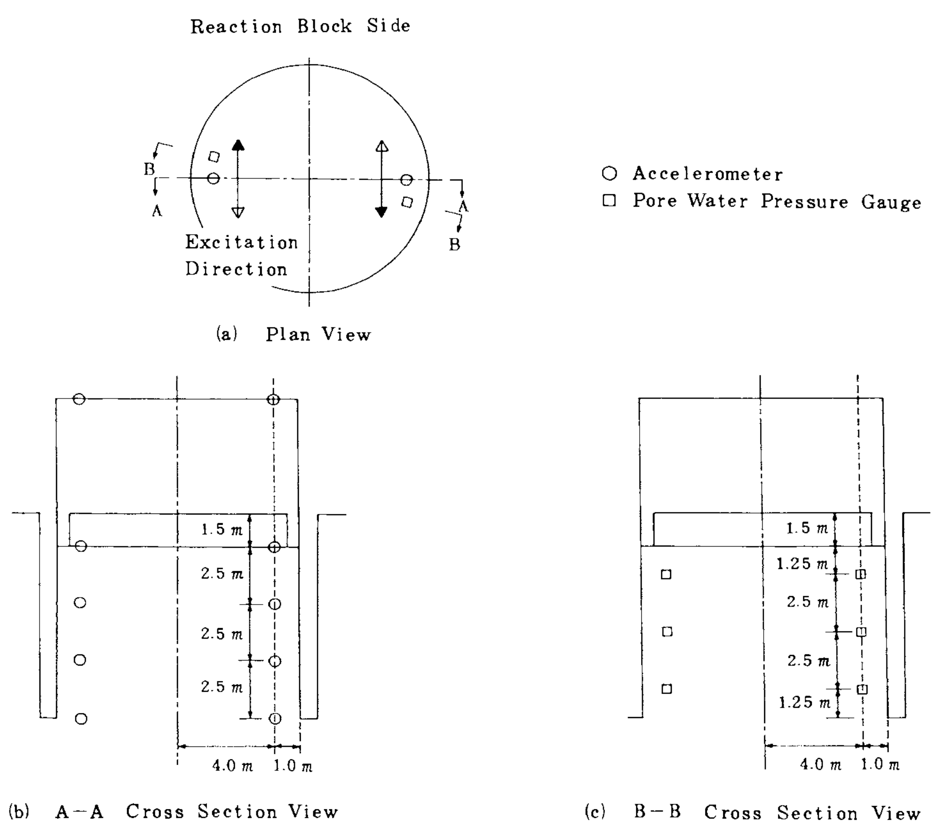

Fig. 4.2 Measuring Instrument Arrangement for Dynamic Load Test of 9m Soil Column Model

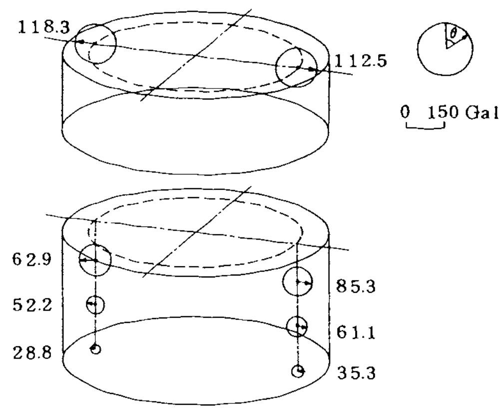

Fig. 4.3 Acceleration Mode at Natural Resonance Frequency
for 5m Soil Column Model
(50t·m Loading — Frequency Increasing Load Pattern)

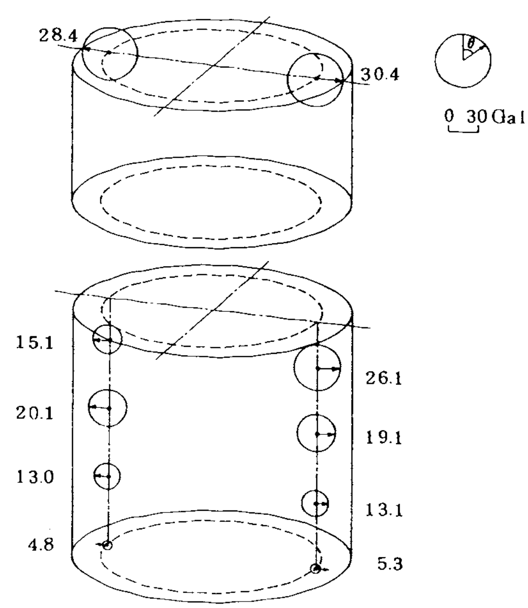

Fig. 4.4 Acceleration Mode at Natural Resonance Frequency
for 9m Soil Column Model
(15t·m Loading — Frequency Increasing Load Pattern)

The excessive pore water pressure fluctuated at about the same frequency as the excitation signals, and dilatancy was not appeared. The soil column model showed only a slight reduction in stiffness and little increase in damping, and excessive pore water pressure did not accumulated at the shear strain level of 10^{-5}.

5. STATIC LOADING TEST OF SOIL COLUMN MODEL

5.1 Loading method

For the static loading tests of the soil column model, a pair of hydraulic jacks ware installed between the capping block and the reaction block as shown in Fig. 2.2. The hydraulic jacks were operated in opposite phase and torsional moment was applied to the cap of the soil column.
The loading were carried out in 6 stages, 900t.m, 1,200t.m, 1,800t.m, 2,400 t.m, 3,000t.m and 3,600t.m, and 5 cycles of load with a triangular wave shape were applied in each stage.

5.2 Measuring method

The measuring instruments used were tilt gauges, pore water pressure gauges, and settlement gauges. The gauges for the soil column model were buried in the boring holes prior to constructing the capping block. The locations of measuring instruments are shown in Fig. 5.1.
The tilt gauges were installed at two places across the diameter of the soil column model at a distance of about 8m. The shear strain of the upper, middle and lower parts of the soil column model were obtained by converting the measured values of the tilt gauges.
The pore water pressure gauges were also installed near the tilt gauges, and the fluctuation of the excessive pore water pressure was monitored at the same depth as the tilt gauges.
The settlement gauges were installed at two places at a distance of about 8m along a line perpendicular to the line of tilt ganges installation. The compressive deformation in vertical direction were measured at depth of 2.5m, 5m and 7.5m.

5.3 Results of measurement

(1) Shear strain

The shear strains at the final cycle of 900t.m loading and that of 3,600t.m loading are compared, the shear strain in the upper one-third of the column length by the 3,600t.m loading was 14 times as much as the shear strain induced by the application of 900t.m load. In contrast to this in the middle and lower parts of the soil column, this difference was 6 times. It shows that the stiffness of the soil column model tends to decrease in a non-linear manner. This tendency was especially predominant in the upper part of the soil column model.
At the maximum loading of 3,600t.m, the shear strain attained was 3.8×10^{-3} in the upper part of the soil column model and $1.1 - 1.2 \times 10^{-3}$ at the middle and lower parts, namely the tests performance achieved the intended target of 10^{-3}.
Fig. 5.2 shows the distribution of shear strain along depth at final cycle of 3,600t.m loading. From this figure, it can be seen that the shear strain on the left and right sides show similar values, and that torsional shear deformation has occurred symmetrically. It is obvious that the shear strain was larger at the upper part of the soil column model. Fig. 5.3 shows the hysteresis curves of the shear strain at 3,600t.m loading, and this curve shows a spindle shaped pattern which comes from an inherent characteristics of the soil.

(2) Compressive deformation

It was found that when the loading moment was small the compressive deformation was also small, but, when the loading moment became 3,600t.m then the compressive deformation increased largely. During the maximum 3,600t.m loading the compressive deformation was 1.43mm in the upper layer of 2.5m thickness, and 1.68mm in the total layer of 7.5m thickness, which indicate that almost all of the compressive deformation occurred at the upper layer of 2.5m thickness.
Fig. 5.4 shows the time histories of the loading moment and compressive deformation for 3,600t.m loading. The soil column model is seen expanding during the loading back and forth and contracting during unloading. It is also seen that the compressive deformation fluctuated in twice cycles of the loading frequency. With increasing number of loading cycles the compressive deformation continued to accumulates. It is observed that the test soil column induced compressive deformation as a whole by repeating the expansion and contraction.
This can be explained by the dilatancy phenomenon inherent in dense sand or gravel.

(3) Excessive pore water pressure

Fig. 5.4 shows the time histories of the loading moment versus the excessive pore water pressure for the 3,600t.m loading. From this it is seen that the excessive pore water pressure decreased during loading back and forth but increased during unloading, and fluctuated at twice cycle of the loading frequency. That is, the dilatancy appeared, however, accumulation of excessive pore water pressure was not observed. The reason for this is due to operation of the hydraulic jacks. The operation of the hydraulic jacks was inevitably stopped for a while at the zero loading and this must have caused the dissipation of the excessive pore water pressure.
From the test results of aforementioned, it was found that at the shear strain level of 10^{-3} the test soil column indicated considerable reduction in stiffness, and with the repetition of loading the compressive deformation occurred due to dilatancy effects.

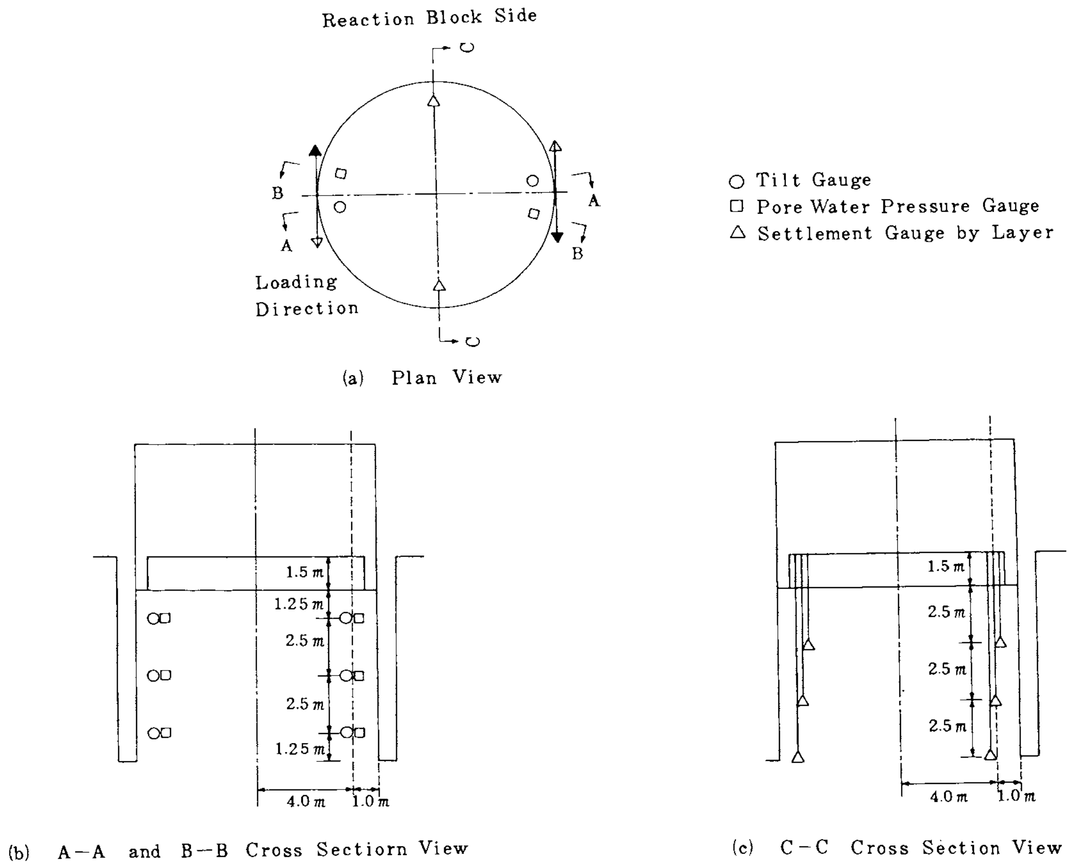

Fig. 5.1 Measuring Instrument Arrangement for Static Load Test of 9m Soil Column Model

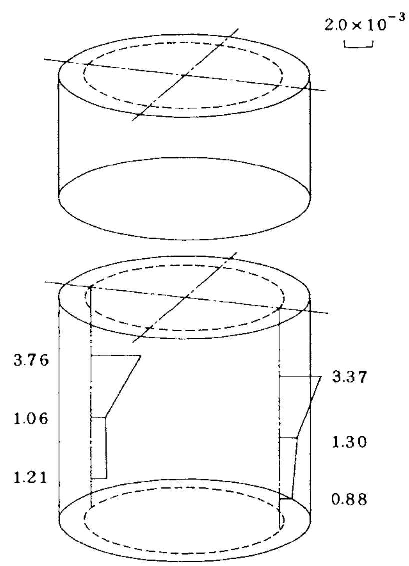

Fig. 5.2 Shear Strain Distribution for Static Load Test of 9m Soil Column Model
(3,600 t·m Loading—1/2 of Double Amplitude at Final Cycle

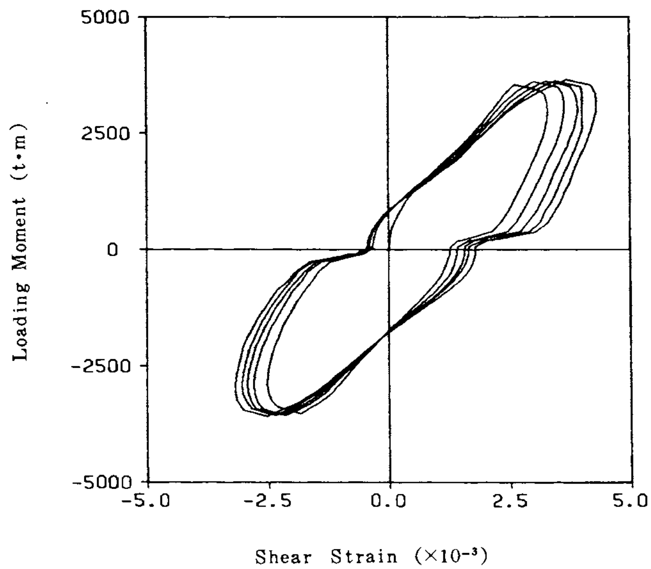

Fig. 5.3 Shear Strain Hysteresis Curve for Static Load Test of 9m Soil Column Model (3,600t・m Loading—Upper Part of Right Measuring Line)

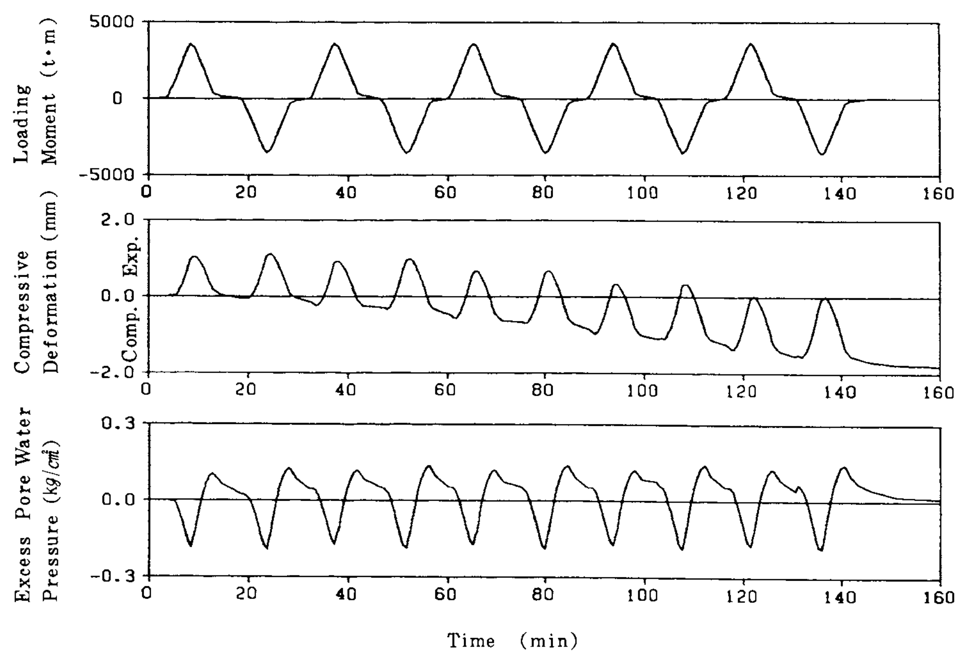

Fig. 5.4 Time History Waveforms of Loading Moment, Compressive Deformation and Excessive Pore Water Pressure for Static Load Test of 9m Soil Column Model (3,600t・m Loading)

6. DYNAMIC LOADING TEST OF CONCRETE BLOCK

6.1 Loading method

The dynamic loading tests were performed by mounting two sets of exciters on top of the test block and the reaction block independently. The exciters were installed in parallel and perpendicular to the direction of excitations as shown in Fig. 6.1. In the case of the dynamic loading test, the excitation was conducted at low load level with the objective of obtaining the dynamic soil-structure interaction characteristics within the elastic range. Applied loads by two sets of exciter were; 2ton for the test block, and 20ton for the reaction block. The excitations were performed by increasing frequency at an frequency increment of 0.1Hz.

6.2 Measurement method

In both case of the tests using test block and the reaction block, the standard point of observation was at the center on the top of the blocks and the measurement were conducted to observe the dynamic behaviors of the test block, the reaction block and the surrounding soil. In each case, excitations were conducted within the elastic range of the soil. The locations of measuring points are shown in Fig. 6.2.

6.3 Results of measurement

(1) Microtremors measurement

In order to know the soil characteristics of dynamic interaction at micro amplitude level the microtremor measurements were conducted for the test block and reaction block, as a preliminary study. The Fourier spectra of the microtremor measured at the tops of the test block and the reaction block are shown in Figs. 6.3 and 6.4. In the figures, the peak near the 1.0Hz is the predominant frequency of the soil and the natural frequency of the test block and reaction block can be read off as being 3.4Hz and 5.7Hz respectively.

(2) Dynamic loading test

Figs. 6.5 and 6.6 show the resonant curves and phase curves obtained at the top of the test block and the reaction block. Each of the curves are the average of three similar curves obtained at three points indicated in the figure. The amplitude of the resonant curves are normalized to that corresponding to 1 ton excitation, and the phase curves were indicated in terms of phase lag from exciting force.
Regarding the resonant curves, in all cases of two mutually perpendicular excitations for both the test block and the reaction block, the natural resonance frequency is shown to be predominant in the range of 1 - 20Hz. For the test block, the rocking mode was predominant, and for the reaction block the amplitude of sway and rocking modes of vibration was approximately the same.
When the response was observed in the reaction block due to excitation of the test block in Y direction, the sway mode of the reaction block was manifest at the natural frequency of 3.2 Hz which belongs to the test block, and later the torsion mode was predominant at a frequency of 8.5Hz.
In the measurement of the test block during the reaction block excitation in Y direction, the rocking mode was predominant at natural frequency of 3.2 Hz in the test block, and later the torsion mode was predominant at 5.8Hz.
After the excitation was finished with 2ton load, additional tests were conducted with 5ton and 10ton loads only near the vicinity of the natural resonant frequency of the test block in Y direction. As the excitation increased, the peak of the natural resonant frequency shifted toward the side of long period and the amplitude was also increased. Although the difference between 2ton and 5ton loading was small, the difference between 5ton and 10ton loading was especially large, and non-linearity effects was observed.

7. STATIC LOADING TEST OF CONCRETE BLOCK

7.1 Loading method

For the static loading test the target shear strain level of the soil was 10^{-3}, and while maintaining the stability of the test block the loads were increased in which the strain level could be obtained sufficiently. The loading were applied statically at the center of the gravity of the test block in both push and pull directions, and in 10 steps of 100ton increment up to the maximum 1,000ton. Moreover, at each loading steps the loading were repeated 5 times, as shown in Fig. 7.1.

7.2 Measurement method

Of the measuring instruments, the buried type tilt gauges and settlement gauges, etc. were buried in the boring holes prior to constructing the test block. The tilt gauges and lateral displacement gauges between the test block and reaction block were installed before the loading test was started. The measuring was conducted in real time by using digital instruments.
The items measured were the vertical displacement of the soil, tilt of soil, lateral displacement between the test block and reaction block, and the tilt of the test block.

7.3 Results of measurement

Regarding the vertical displacement of soil, when the load increased to exceed 500ton then the residual settlement after unloading became noticeable, and the amount of settlement showed a tendency to increase as the load increased, Fig. 7.2.
Regarding the lateral displacement between the test block and reaction block, when the load increased in excess of 500ton then the

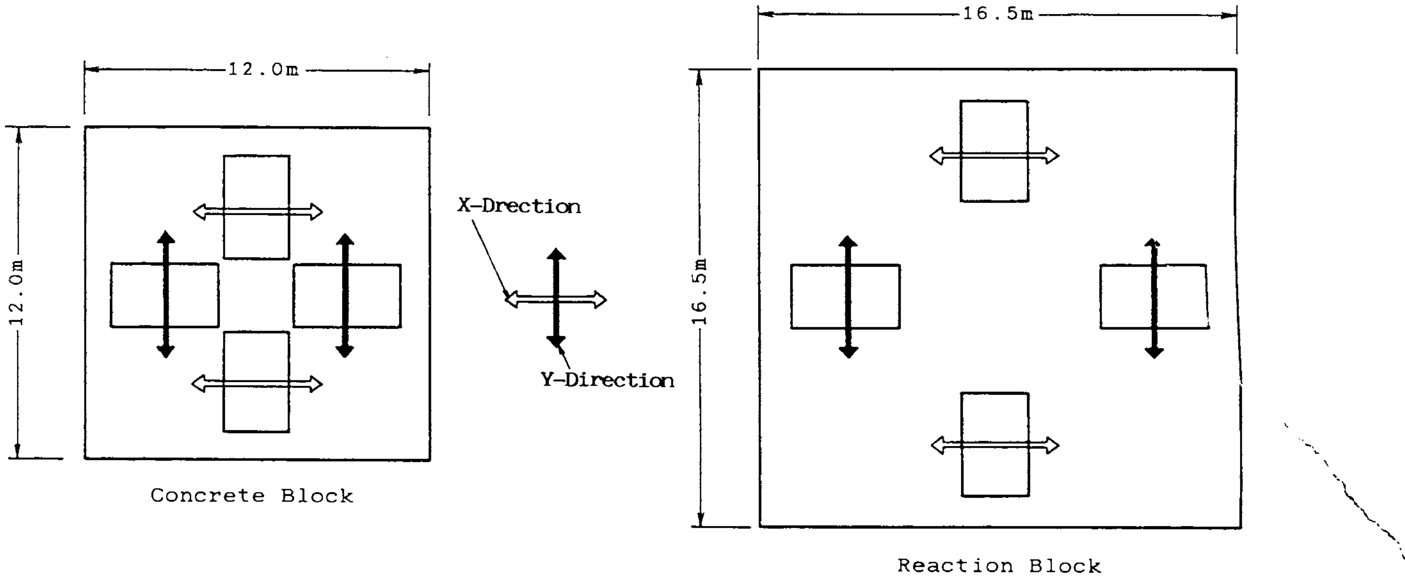

Fig. 6.1 Installed Location of Exciter

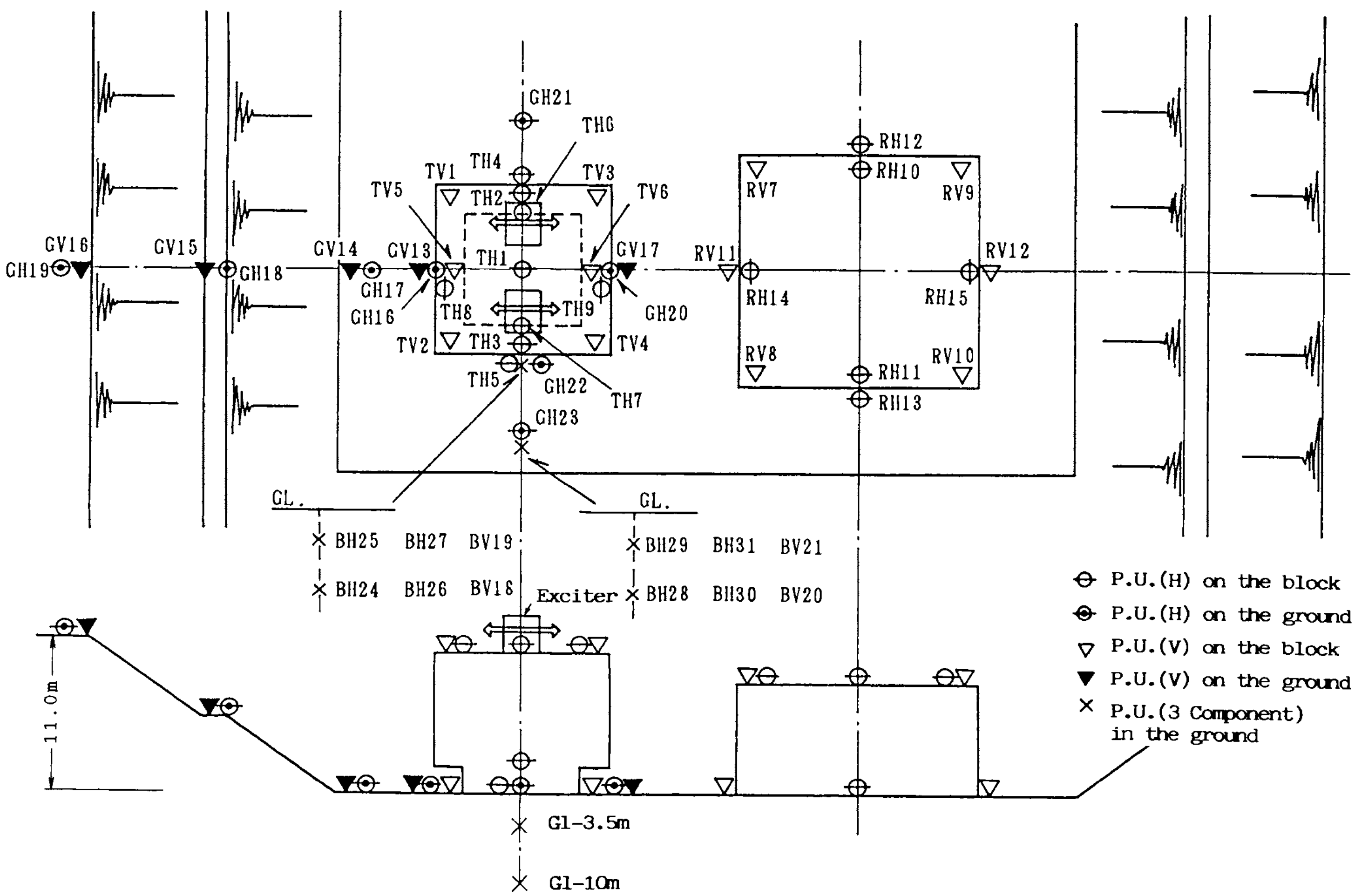

Fig. 6.2 Displacement Gauge Distribution

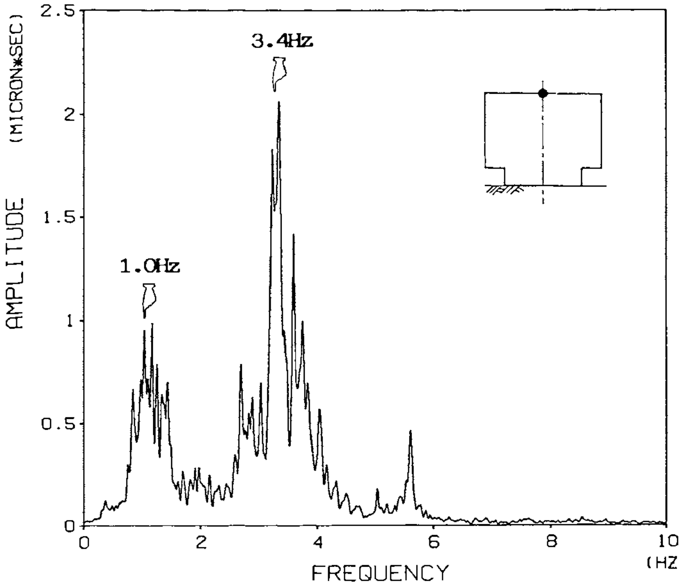

Fig.6.3 Fourier Spectrum of Microtremor
(Concrete Block)

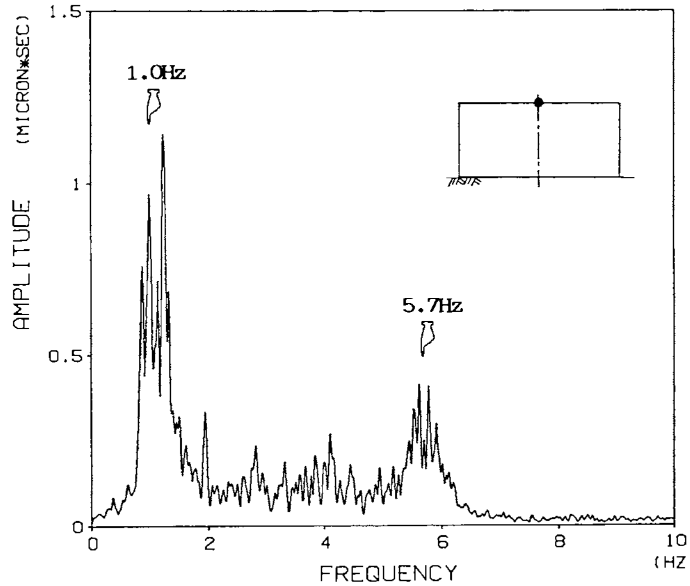

Fig.6.4 Fourier Spectrum of Microtremor
(Reaction Block)

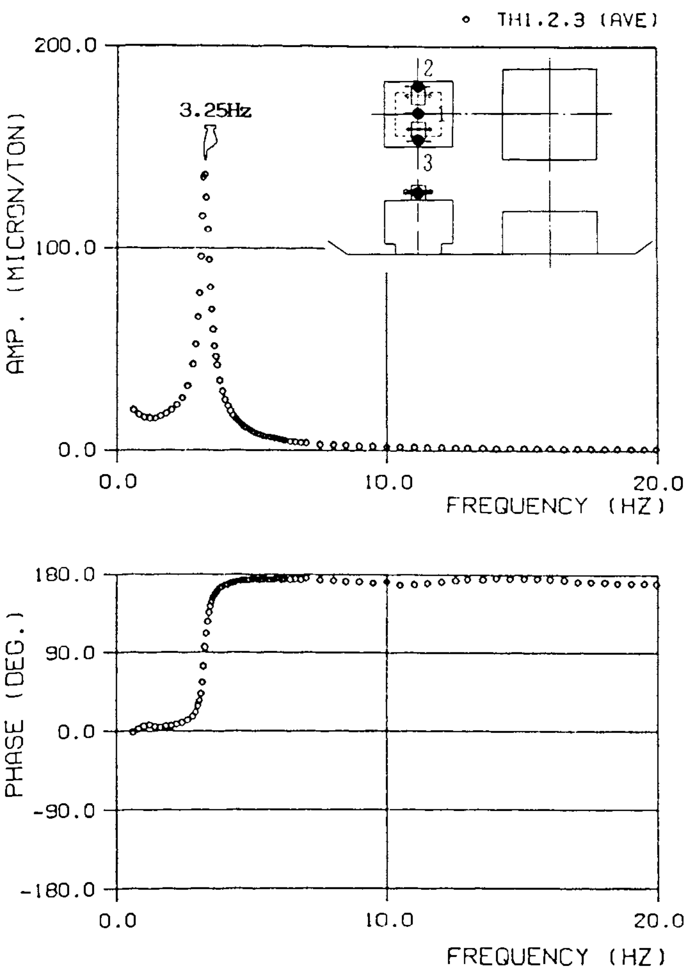

Fig.6.5 Resonance & Phase Curves
(Concrete Block)

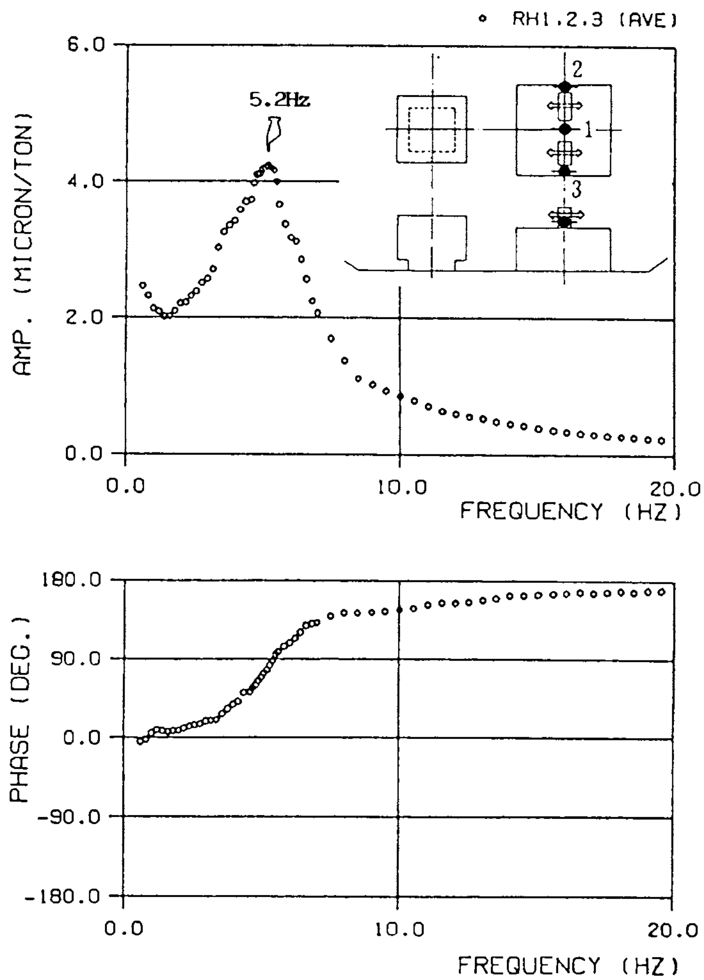

Fig.6.5 Resonance & Phase Curves
(Reaction Block)

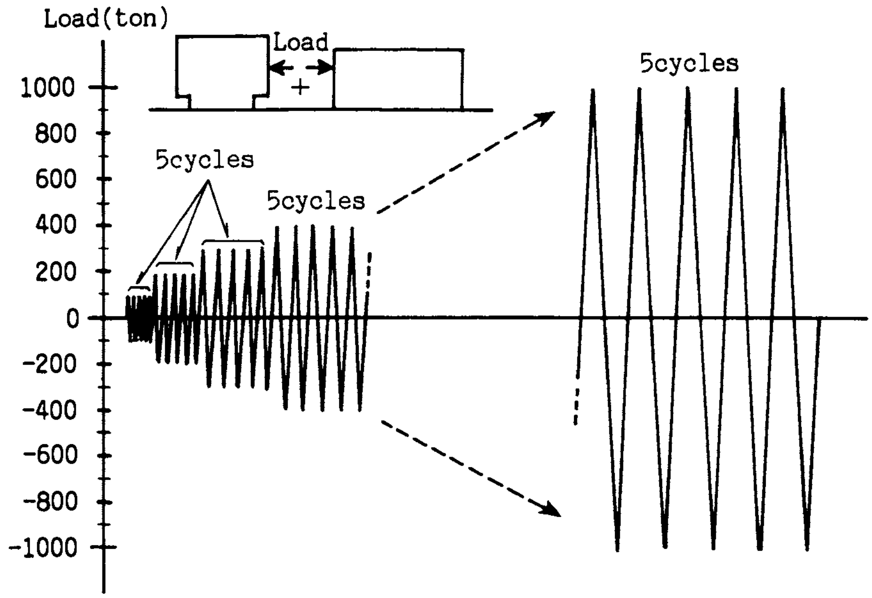

Fig. 7.1 Loading pattern

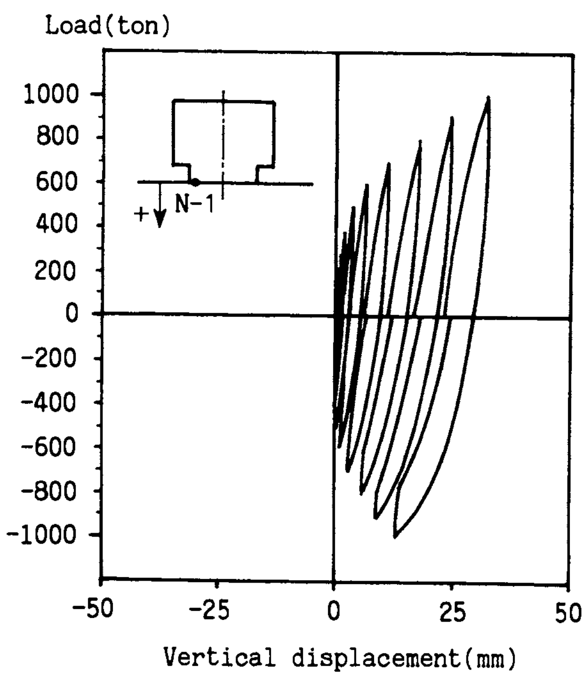

Fig. 7.2 Vertical displacement of soil
: N-1 point
: Fifth cycle of each loading level

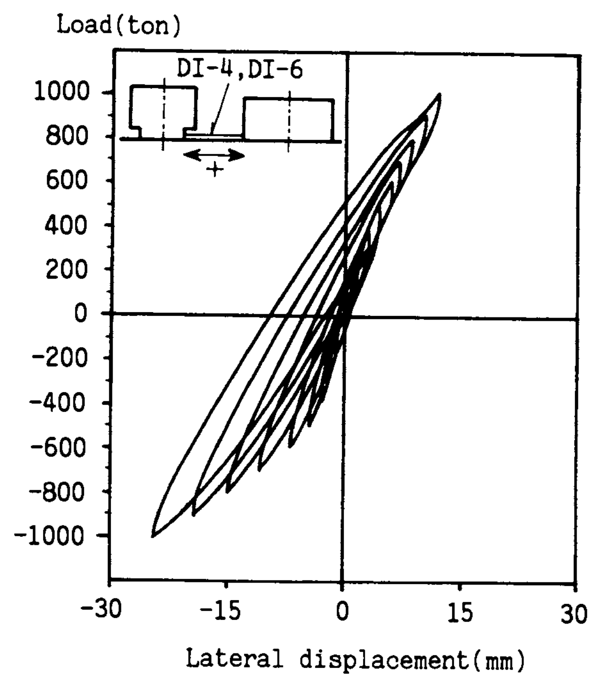

Fig. 7.3 Lateral displacement between concrete block and reaction block
: Average of DI-4 and DI-6 section
: Fifth cycle of each loading level

increase in the amplitude became noticeable and the non-linearity effect was observed to be remarkable. Moreover, a residual lateral displacement was occurred in the direction of pull load, as shown in Fig. 7.3.
In the same manner, regarding the tilt of the test block, the amplitude increased with increasing load, and residual tilt occurred in the direction of pull load.

8. CONCLUDING REMARKS

As a result of the field tests on seismic soil stability at a diluvial gravel site, the following items were clarified:

8.1. Soil investigation

As a result of the laboratory tests on high quality undisturbed samples, extracted by in-situ freezing method, the mechanical properties of gravelly soil, necessary in the verification test, have been mostly clarified. Furthermore, it is considered that the obtained new knowledge has been made available for the studies of siting of other structures on similar soil conditions.

8.2. Soil column model test

As a result of the soil column model tests, in which the torsional loading was applied directly to the field soil, many quantitative data for the strain dependency of the soil stiffness and the compressive deformation due to dilatancy were obtained. Excellent data were collected regarding repeated deformation characteristics from the level of micro shear strain to large shear strain of the gravelly soil in the field. Moreover, the gravelly soil did maintain stability at the level of the large shear strain, and it can be considered that the above soil possesses adequate earthquake resistance.

8.3. Concrete block test

As a result of dynamic loading tests, clear resonance curves were obtained for both the test block and the reaction block, and effective data necessary in the evaluation of soil - structure interaction on the quaternary deposits were obtained. Also, the resonance curves of interaction between the test block and the reaction block was shown clearly, and it indicates that an attention must also be paid to the influence of adjacent structures, when plural buildings will be constructed on quaternary soils.
As a result of static loading test, the stability was verified regarding bearing strength, settlement and sliding on gravelly soil at large shear strain level. Also, the soil residual settlement phenomenon that accompany repeated loading was observed, and it was acknowledged that this evaluation would be important in designing the nuclear power plant building.

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