DYNAMIC TESTS OF SOIL EMBANKMENTS

by

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SYNOPSIS

A result of dynamic tests of full-scale embankments on a large shaking table are reported in this paper. The study was carried out to investigate the dynamic behaviour of soil embankment and to analyse its stability under dynamic loading, by means of the circular arc method. It is demonstrated that the dynamic response of embankment was appreciable under the few hertz of frequency, the natural frequency decreased while the damping factor considerably increased with increase of the acceleration amplitude, and thus the circular arc analysis which takes into consideration the response of the embankment is recommendable for examination of its stability against earthquakes.

INTRODUCTION

Soil embankments with approximately trapezoidal cross sections are typical structures, including the earthfill dams railway and highway embankments, and levees. Recently in Japan population and industry are on the sharp increase in the coastal areas which are protected by levees from flood caused by high tides and tsunami. Aginst such casualties the authors planned to improve the earthquake proof design for soil embankment by applying to the model test procedure. One of the difficulties of the model test on soil structures is to satisfy the smilarity between the model and the prototype, and the special feature of collapse on the actual embankment was hardly ascertained by means of the observation of vibrations caused by the artificial excitation. Nowadays analytical procedures have made great advances using computer. What is much needed now is reliable results from experiment. To meet this need full-scale models on a specially constructed shaking table were tested. The results given here in this paper, we expect, will be informative for the actual designing work and also applicable to other similar structures with appropriate interpretation.

From the previous studies it was found that the dynamic response of the embankment was considerable and so the stability analysis should be recommendable with consideration of acceleration distribution in the embankment.(1),(2) The circular arc analysis, in which seismic coefficients proportional to acceleration distribution are considered, is a static method modified by the dynamic effect, but it is one of the practical procedures to judge the resistibility of the embankment against earthquakes. The authors compared the response calculating method, viz. the lumped-mass analysis and the finite element analysis, with the test results, and examined their applicability to the related embankment.

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VIBRATION TEST PROCEDURE

Shaking Table As shown in Fig.1, the shaking table used for the experiments consisted of a reinforced concrete box and eight vertical steel pipe piles supporting the box. The size of the box was 14.5 m long, 6 m wide and 3 m high. This structure is almost considered as a single-degree-of-freedom system with a natural frequency of about 4 Hz, similar to that of earthquakes.(3) The rotating mass type vibration generator with 100 tons of maximum force at frequency of 9 Hz was installed on the floor slab extending from the one side of the box. According to the preliminary test it was demonstrated that this table could produce sufficient sinusoidal movement in the horizontal direction.

Models Two kinds of soils were used to make the model embankments. The mechanical properties of the soils are given in Table 1. Six embankments with different conditions were constructed in the shaking box. As shown in Fig.2, they have a trapezoidal cross section with symmetrical slopes and they are respectively 2.5 and 3.5 m high and both 5.64 m long. Models were made layer by layer of 50 cm thickness and compacted by an electric vibrator or a wooden compactor.

Vibration Tests The models constructed on the shaking table were subjected to several series of sinusoidal vibrations perpendicular to the axis of the embankment. During the first series the exciting force of the generator was kept 5 tons constant for the increasing frequency from 2 to 10 Hz. After that it increased in turn to 10 and 15 tons. In the final test on each model the maximum exciting force the generator could produce was applied very rapidly with increasing of frequency for the purpose of observing the collapse of the embankment. As well as the table acceleration the accelerations at 14 points in the models were measured at each 50 cm high at the center, near the side walls and the slopes, by accelerometers as shown in Fig.2. Deformation of embankments was measured after each series of vibrations.

MEASUREMENT OF SHEAR WAVE VELOCITY

Prior to the vibration tests the velocity of shear wave propagation through embankments from top to bottom was measured by applying horizontal impacts to the top. The shear wave was recorded by accelerometers at the center of models, and its velocity was determined by time lag and distance between neighbouring transducers. The shear modulus is gained as the product of the specific density and the squared shear wave velocity. The measured velocity and calculated shear moduli are given in table 2. As is evident from it shear wave velocity increases as the shear wave propagates downwards. It is known that the modulus of elasticity and that of shear rigidity increase proportionally with increase of the confining pressure in the range of more than about 0.4 kg/cm^2 of confining pressure.(4) Table 2 indicates that these moduli were more sensitively influenced by the change of pressure under a very small confining pressure.

DYNAMIC BEHAVIOUR OF EMBANKMENTS

Table Acceleration The shaking table itself was a vibration system similar to one-degree-of-freedom system with a natural frequency of about

4 Hz. Therefore the table acceleration changed as the frequency changed though the exciting force was kept constant. At the final test on each model acceleration increased very rapidly with the maximum exciting force of the generator. The produced table acceleration in tests 10 and 11 is given in Figs.3 and 4. The maximum table acceleration produced in each model was within the range of 200 to 300 gals which were almost as large as the ground acceleration of a destructive earthquake. Therefore it could be judged that the dynamic behaviour of the models in the tests is that of the proto structure under a strong earthquake.

Acceleration Response Figs.7 and 8 are examples of acceleration response of the top of the model obtained at the center. The general feature of the response curves of sandy models is that the resonant frequency, in which response value is maximum, in each series of excitations decreases with increase of the exciting force, viz. increase of the ground acceleration. The maximum response values also decrease with increase of the acceleration. These facts show the non-linear dynamic characteristics of sandy embankments. On the other hand the response of clayey models shows considerably complicated aspects. The models were strongly confined at both sides by the adhesion of soil to concrete walls and they behave the three dimensional motion under the vibration. Consequently the response characteristics of the clayey embankment in this experiment are hardly expressed by the two dimensional modeling. The maximum accelerations at top amounted to about 1600 gals in the sandy model and 2700 gals in the clayey model. This large amount of acceleration must carefully be taken into consideration in the designing of the structure.

Lumped-Mass Analysis One of the reasons why the lumped-mass system analysis was chosen was that the calculation procedure was comparatively simple and would be carried out by use of either a digital computer or an analog computer. The embankment is subdivided into horizontal layers and idealized by a series of lumped-masses interconected to weightless springs that resist lateral deformations. The models are thought of herein as systems with seven or five masses and springs as shown in Fig.11. The damping of the system is proportional to the relative velocity between the adjacent masses, and the damping ratios of each layer in a model are assumed to be equal.

Finite Element Analysis The finte element method is a powerful extension of matrix structural analysis procedures for obtaining digital computer solutions as to problems of the continuum. Many valuable results on the dynamic behaviour of the earth dam have been gained by using the procedure.(5),(6) Dynamic responses of embankments gained by the two dimensional analysis were calculated for the purpose of comparing with measured values and calculated ones by lumped mass analysis. Further three dimensional dynamic behaviour could be taken into consideration because the embankments were not long enough to be regarded as a two dimensional body. The Young's modulus of each element of the model was determined by the shear modulus obtained by the shear wave velocity and the Poisson's ratio assumed to be 0.3 for all models.

Comparison of Response Curves As the first step of the calculation by the lumped-mass system, the stiffnesses of the springs determined by the measured shear wave velocity were used. However the calculated natural frequency of the first order differed from the observed one. Then, a constant factor was multiplied to the above-mentioned stiffnesses, and the calculation was repeated. In such a way the stiffnesses of the springs of the lumped-mass system was determined which led the same natural frequency to the observed one. In this paper the multiplier is called Shear Modulus Ratio. In the next step the damping was selected to give the equal response value to the measured one at the resonant frequency. A constant damping ratio was used for all layers of the model.

In the calculation of the two dimensional finte element analysis, the same procedure was employed for determination of the strength factors of the models, and then the multiplier for the Young's modulus and the damping ratio were decided. For the calculation of the three dimensional finte element analysis the same values of the Young's modulus and the Poisson's ratio as that of the two dimentional analysis were used. The acceleration responses of the models are shown in Figs.5 and 6. The response ratios in the figures are that of accelerations at 50 cm under the top of the embankment to the table accelerations.

In Figs.12 and 13 are shown the distributions of accelerations measured and calculated. The distributions obtained by the lumped-mass analysis and the finite element analysis agree well with the measured one. This fact indicates that, even though the multipliers were introduced, the shear moduli and the Young's moduli obtained from the shear wave velocity are close to the actual values of the embankment, and the simple way employed in this series of experiments is a very good procedure for the measurement of the shear wave velocity. In those figures the calculated distributions in which was assumed a constant shear modulus throughout the model are also shown, and they differ considerably from the measured ones. The approximate values of the maximum response of each test were listed in Table 3 in which the first resonant values were only given as to Tests 11 and 12.

Multiplier and Damping Ratio Fig.14 shows the shear modulus ratios which are plotted against the table accelerations. Generally, the resonant frequencies decreased with increase of the exciting forces as shown in Figs.7 and 8. It is supposed that the shear rigidity of the embankments decreased with increase of table accelerations. Namely Fig.14 indicates that the shear modulus ratios range from 0.2 to 0.7, and the shear rigidity rapidly decreases in the range of under 100 gals of the table accelerations and did not change that far in the range over these values. The Young's modulus ratios, determined in the same way as that used in the lumped-mass analysis are also plotted in the figures. As shown in Fig.15 the damping ratios did not increase that far in the range under 100 gals of the table accelerations but out of the range a considerable increase was observed with the increase of the table accelerations. The damping ratios determined by the finite element analysis are also plotted in the figures. The significant features of Figs.14 and 15 evidence the non-linear behaviour of soil under dynamic motion and the figures suggestive of the analysis of the embankment.

STABILITY ANALYSIS OF EMBANKMENTS

Circular Arc Analysis Judging from a practical point of view, the circular arc analysis is the most common of the stability analyses of the slopes. To apply the dynamic effect of the earthquake to this analysis was adopted the seismic coefficient method, in which the seismic coefficient is defined as the ratio of acceleration acting on the structure to .

that of the gravity, and the seismic force acts statically as the product of the coefficient and the mass of the structure. Conveniently this method makes the analysis simpler by means of the replacement of the dynamic problem by the static one and is thus acceptable as a means of the calculation of the rigid structure.

As the dynamic response of the soil embankments was considerable, the accelerations measured in every 50 cm of thickness of the embankments were used for the distributed seismic coefficients in the computation. The angle of internal friction and the cohesion of soil were given by the conventional triaxial compression test and the safety factor of models were calculated as to the conditions:

Case 1. No seismic force acting;
Case 2. The seismic coefficient at the shaking table acting throughout a whole model;
Case 3. The maximum seismic coefficients measured at each level during a cycle of vibration;
Case 4. The seismic coefficients measured at each level in an instant during a cycle of vibration;

For an example the safety factors obtained in the way described above are listed in Table 4. Fig.10 shows the location of the sliding arc with the minimum safety factor. As soils of all the models had more or less a cohesive component, the location of the sliding arcs were deep under the slopes even though the seismic coefficients were very large at the upper parts of the models. As to the models of tests 3 and 4 with comparaively high natural frequencies of first order, the results of Cases 3 and 4 were almost the same. But as to the models of 3.5 m high the phase differences between bottom and top were appreciable; therefore the results of Case 4 were small compared with that of Cace 3.

Failure of Embankments Although the collapse with sliding surface due to the vibration did not happen in this experiment, the embankments suffered such damage as settlement and collapse at crest. As to some models, as a large amount of deformation was observed from the window at the side wall, the potential sliding surfaces were determined and compared with the calculated one as shown in Fig.9. Judging from the cracks on the slopes and the difference of accelerations at the center and nearby the side wall, the potential sliding surface at the center could be similar to the calculated one. In the sandy models the accelerations at the table and the crest were 200 ∿ 300 and 600 ∿ 1000 gals respectively; the settlement of 5 ∿ 11 per cent of the height; in the clayey models they were about 200 and 2000 gals respectively; and the settlement of the height was measured 4 ∿ 5 per cent. In the case of the destructive excitation, the cracks occured and increased in mumber on the crest and slopes, and finally small-scale failure happened at the upper part of the models. If we apply these facts to the proto structures, there may correspond to a very serious situation, and the features of damage upon the models were similar to the typical one caused by the strong motion of earthquakes.

CONCLUSIONS

The results of the experiments and the analyses concerning the large-scale models of the soil embankments are as follows:

1) In the results of vibration tests the natural frequency decreased and

the damping factor increased with increase of the acceleration. This is well explained by non-linear characteristics of soil.

2) The validity of the lumped-mass modeling for embankment was indicated by the comparison of the experimental results and the finite element analysis.

3) The shear moduli of soils calculated from the observed natural frequency differed from the observed shear wave velocity. But the relation of the two values was found to be qualitatively accounted for in terms of the accelerations.

4) At the collapse-causing tests the potential sliding surfaces in embankments were observed and analysed based on the circular arc method which takes the acceleration distribution into consideration.

5) Though the safety factors for sliding given by the circular arc method were more than 1, the settelement, cracks and small-scale collapse at the crest were observed, which could be judged as a kind of damage from the practical point of view.

6) As the dynamic response of soil embankments was considerably significant, it was proposed that the stability against earthquakes should be examined in view of their acceleration response to earthquakes if the embankments are higher than certain values.

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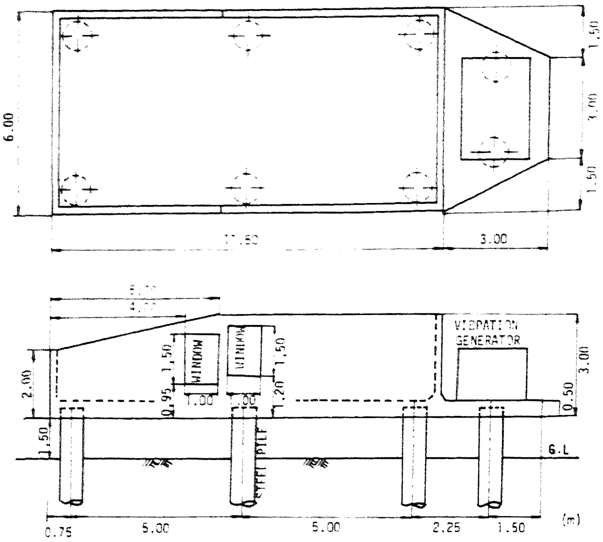

Fig.1 Shaking Table

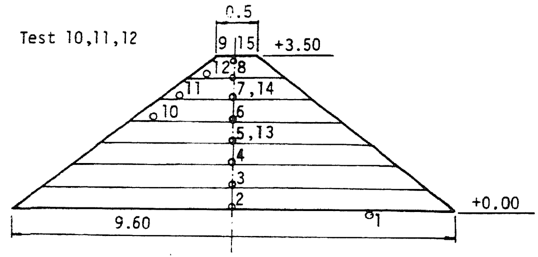

Fig.2 Embankment and Accelerometers

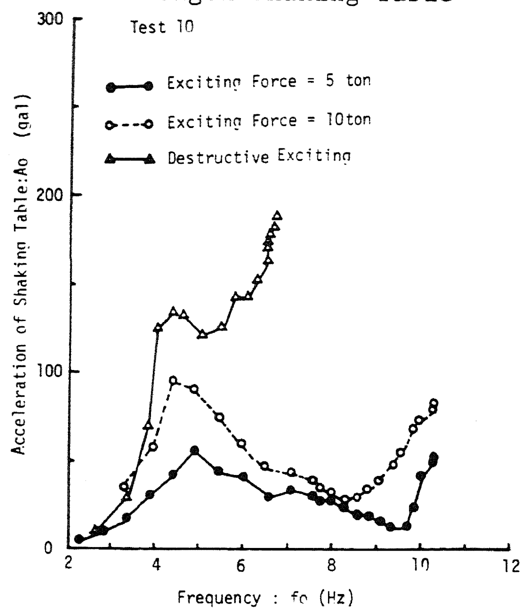

Fig.3 Table Acceleration (Test 10)

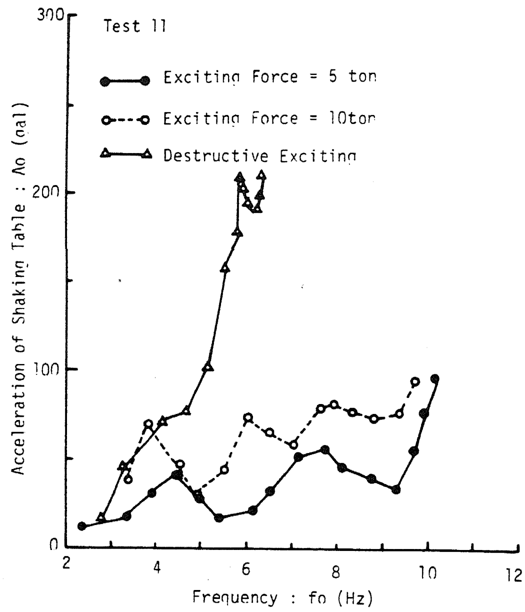

Fig.4 Table Acceleration (Test 11)

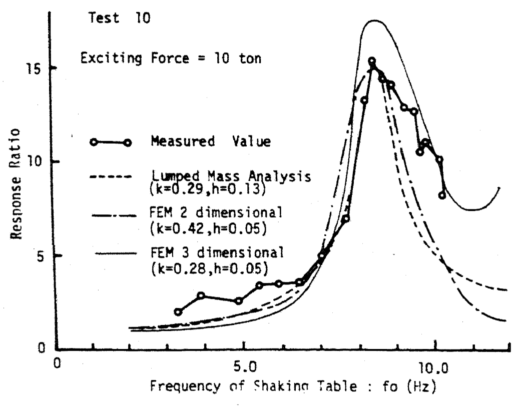

Fig.5 Response Curves (Test 10)

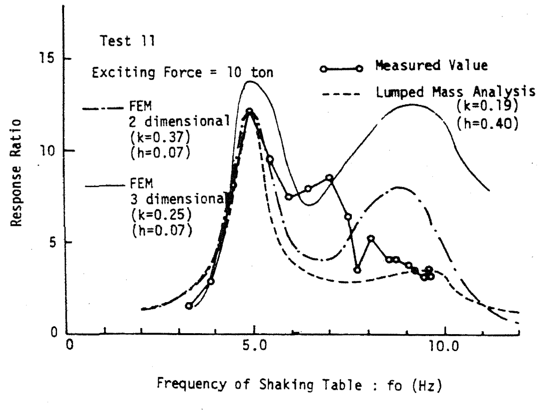

Fig.6 Response Curves (Test 11)

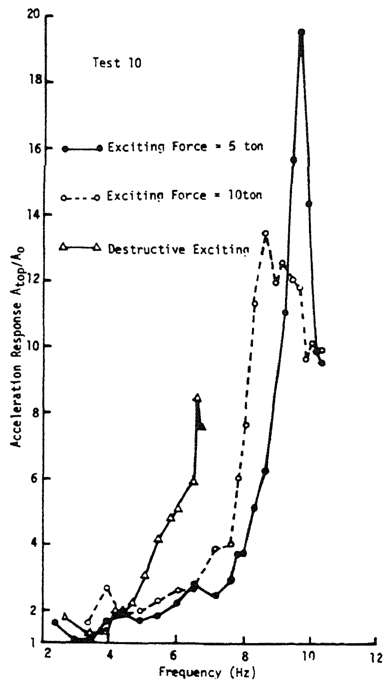

Fig.7 Acceleration Response at Top (Test 10)

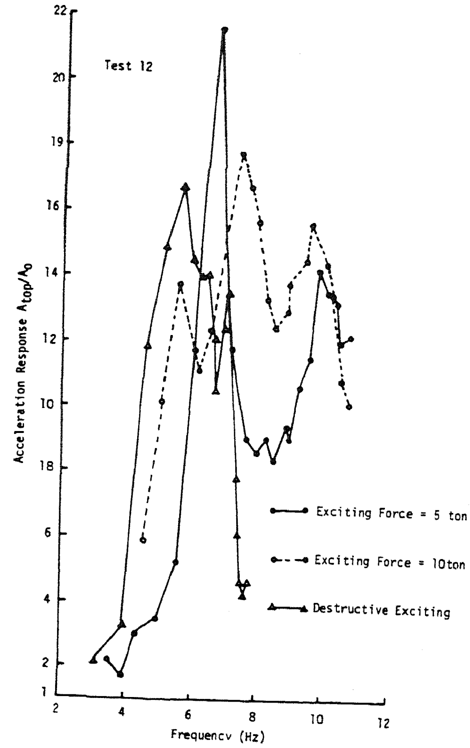

Fig.8 Acceleration Response at Top (Test 12)

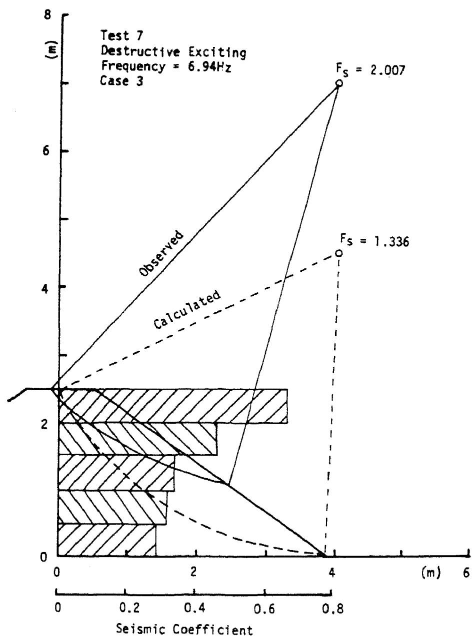

Fig.9 Sliding Surface Observed and Calculated (Test 7)

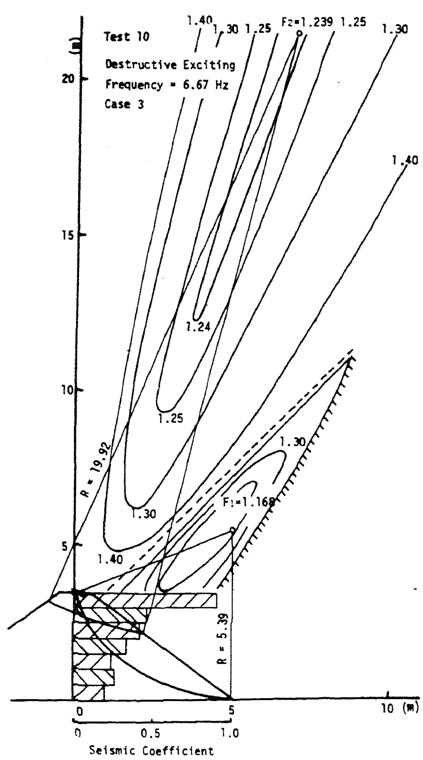

Fig.10 Minimum Safety Factor (Test 10)

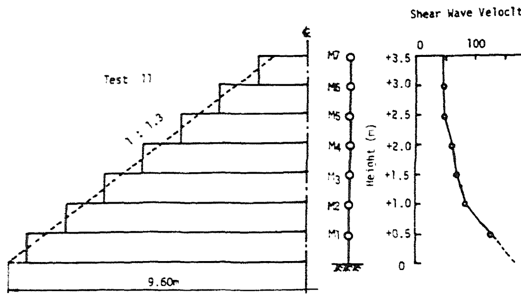

Fig.11 Lumped-Mass Modeling and Shear Wave Velocity

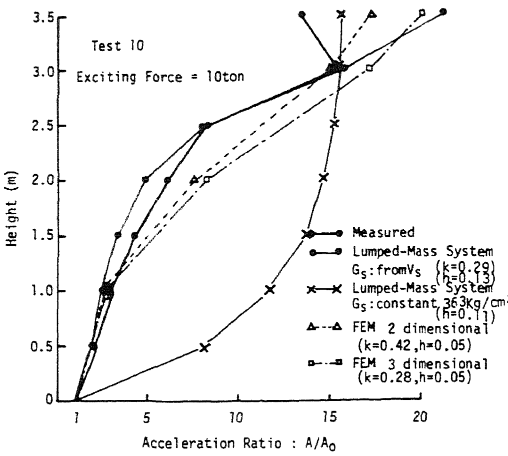

Fig.12 Acceleration Distribution (Test 10)

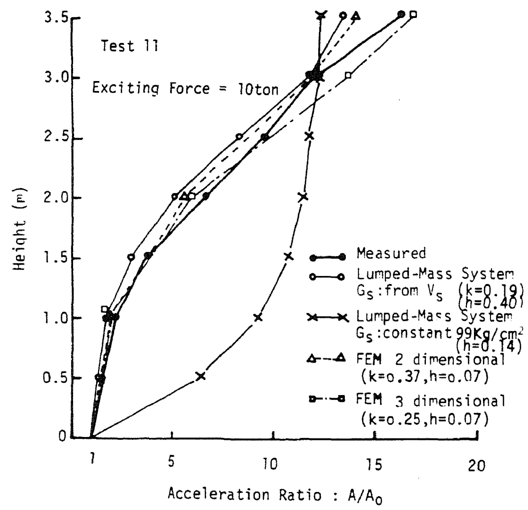

Fig.13 Acceleration Distribution (Test 11)

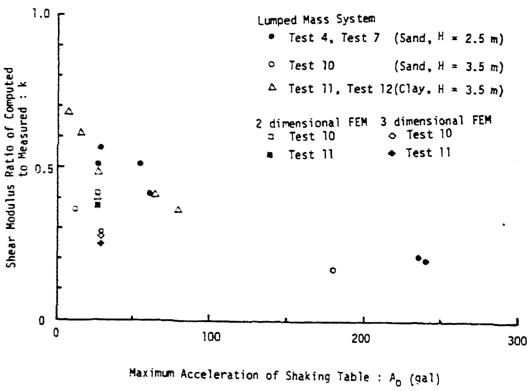

Fig.14 Shear Modulus Ratio

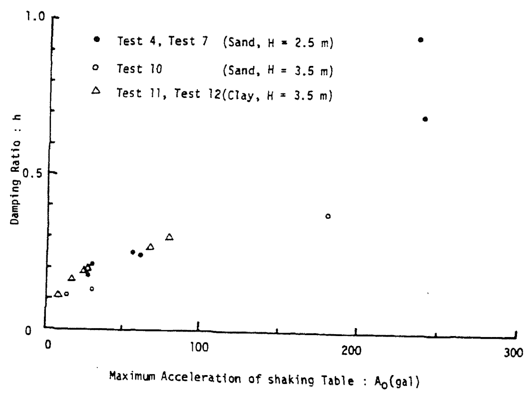

Fig.15 Damping Ratio

	Soil Classification	Specific Gravity	Effective Grain Size (mm)	Uniformity Coefficient
Sandy Soil for Test 4,7,10	Sand	2.64	0.160	2.28
Clayey Soil for Test 11,12	Sandy Clay	2.68	0.0015	60.00

Table 1 Mechanical Properties of Soils

		Elevation (m)					
		0.5 ~ 1.0	1.0 ~ 1.5	1.5 ~ 2.0	2.0 ~ 2.5	2.5 ~ 3.0	3.0 ~ 3.5
Test 4	V_s	111		99	97		
	G_s	183		146	140		
Test 7	V_s	142	114	99	—		
	G_s	328	211	159	—		
Test 10	V_s	179	139	109	104	81	83
	G_s	514	310	191	174	105	111
Test 11	V_s	125	83	68	61	46	45
	G_s	209	92	62	50	28	27
Test 12	V_s	100	82	83	68	48	44
	G_s	134	90	92	62	31	26

V_s: Shear Wave Velocity (m/sec) G_s : Shear Modulus (kg/cm^2)

Table 2 Shear Wave Velocity and Shear Moduli

	Test 3, 4, 7			Test 10			Test 11, 12		
Exciting	F = 5	F = 10	D E	F = 5	F = 10	D E	F = 5	F = 10	D E
Frequency (Hz)	10 ~ 11	10	6 ~ 7	9.5	8.5	6.5	6 ~ 6.6	5 ~ 5.5	5 5.6
Response	11	8 ~ 9	3 ~ 4	20	14	9	13 ~ 22	15	13 ~ 17

F : Exciting Force (ton), DE : Destructive Exciting

Table 3 Approximate Acceleration Response

	Frequency (Hz)	Safety Factor : F_s			
		Case 1	Case 2	Case 3	Case 4
Test 3	5.50	2.15	1.62	1.41	1.44
Test 4	6.00	2.12	1.56	1.43	1.45
Test 7	6.94	2.04	1.46	1.34	1.40
Test 10	6.67	1.72	1.38	1.17	1.31
Test 11	7.01	2.31	1.79	1.43	1.88

Table 4 Comparison of Safety Factors