DYNAMIC RESPONSE OF A LARGE-SCALE SHAKING TABLE FOUNDATION AND ITS SURROUNDING GROUND

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SYNOPSIS

This paper describes an investigation of the dynamic response of a large-scale shaking table foundation and its surrounding ground, when the loaded table was driven horizontally and vertically in sinusoidal motions. In addition, it includes the measured earthquake response of them. The measured data are compared with the theoretical prediction.

SHAKING TABLE AND FOUNDATION

In 1970, a 15 m x 15 m shaking table was constructed by the National Research Center for Disaster Prevention, Governmental Agency. This table is driven in one direction, horizontal or vertical by four hydro-electro actuators. The maximum loading weight is 500 tons for horizontal motion and 200 tons for vertical motion. The table weight is 160 tons. The horizontal actuators have the maximum force of 360 tons, so that they actuate full load up to a maximum acceleration of about 0.5 g. The limtation of table motion is illustrated in Fig. 2. The structure containing the shaking table consists of the open box-type reinforced concrete foundation and the superstructure of steel frame, as shown in Fig. 1. The upper surface of the foundation is the ground floor and is levelled to the table floor. The bottom of the foundation is spread by the footing slab and the total area is 39 x 25 m^2. The foundation rests on the firm sand layer at a depth of 8.2 m from the ground surface and was backfilled to its surface. The total weight of the foundation, equipments and superstructure is about 9000 tons, in which the weight of the superstructure is only 200 tons. The soil weight backfilled over the footing slab was about 5000 tons. The soil profile of the site is found in Fig. 3. It consists of loam and silt of about 8 m thick, fine sand of about 10 m thick and underlying alternate layers of fine sand and dense silt. The ground-water level is located at about 0.5 m below the ground surface. In-situ measurement of velocity was conducted twice by means of the well-shooting. The first time was at the beginning of the construction and the second time was at the vibration test of the foundation. It resulted the S-wave velocity is 80 to 150 m/sec in the top layer, 230 to 250 m/sec in the intermediate layer and 400 m/sec in the underlying layers, as indicated in Fig. 3.

FORCED VIBRATION TESTS

Horizontal vibration test The tests were carried out three times, the first was in Jun. 1973, the second was in Sept. 1973 and the third was in May 1974. In the first and third tests, the shaking table was loaded by the sand box of 380 tons and driven horizontally in sinusoidal motion of acceleration amplitude 300 gals and 100 gals with varying frequency. The sand was so densely filled up in the box as not to be boiled. In the second test,

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it was vibrated without loading in the acceleration amplitude of 70 gals. Thus, the exciting forces were 160, 54 and 10 tons, respectively. These forces are applied to the foundation at a height of the actuator axis, which is located above the center of gravity of the foundation. It causes the foundation to undergo the coupled motion of sliding and rocking. Thus, the measurement points of the foundation were placed on its upper surface near the edges with a distance of 12.5 m from the centroid axis of the floor plan, as marked by "A" in Fig. 1. At the same time, motions at the neighboring ground surface and underground were measured, but are not described in the present paper, because of space limitation.

Fig. 4 is a plot of ratio of the foundation acceleration to the table acceleration as a function of frequency. It is found that there are peaks, at which the values exceed the weight ratio of the table to the foundation, 0.06. Fig. 5 shows the resonance curves of the horizontal and vertical components, where the displacement amplitude u in micron is divided by the exciting force amplitude in tons. It is seen that the curves for the horizontal component have no definite trends with variation of magnitude of the exciting force and all curves possess the analogous feature having the highest peak at 3 Hz and thereafter gradually falling as inversely proportional to the frequency, though small heaves appear at 6 Hz and 9 Hz. However, the vertical displacements increase slightly with increase of the exciting forces. In addition, the peaks at 6 Hz and 9 Hz appear more obviously than those in the horizontal component, but the peak at 3 Hz disappears. Judging these results, it may be considered that the present foundation can be expressed by a sliding and rocking model which has the natural frequencies of 6 Hz and 9 Hz for the first and second modes, respectively. The other peak at 3 Hz would be generated by excitation of the top layer of the surrounding ground and its resulting effects on the foundation. From the curves for the vertical component, the critical damping ratio can be roughly estimated as 0.2 for the first mode by using the half-power method.

Vertical vibration test The shaking table was driven without load in the sinusoidal motion of vertical acceleration of 350 gals. Frequency response curves are shown in Fig. 6, in which the curve designated by "A" has two peaks at 2.5 Hz and 6 Hz, though they are rather rounded. As a whole, the vertical displacement at the center of the bottom is larger than those at both sides of the top floor. From this reason, the mode of vibration is presumed to have a shape alike to that of a beam supported on an elastic foundation. However, the peak emphasizing this flexural mode appears at somewhat higher frequency, as found in the curve of horizontal displacement at the top floor.

EARTHQUAKE OBSERVATION

The earthquake observation started in Oct. 1974. Accelerometers of electro-magnetic type are installed on the foundation, neighboring ground surface and in the boreholes of depth 10 m and 40 m. The latter instruments for the ground motions are located at a distance of 50 m from the center of the foundation on a line of the forcing direction of the table, as indicated in Fig. 1. The earthquakes, during which the maximum acceleration exceeded 10 gals, were 9 times in the first half a year. The earthquake data and measured maximum accelerations are ~~found in Table~~ 1. Roughly speaking, the accelerations at the depth 10 m are approximately equal to those at the foundation, while the accelerations at the ground surface are 2.5 times as large as that at the foundation. Among the nine earthquakes, the six broke outnear the site and have the epicentral distances of less than 20 km. The records taken on these earthquakes are characterized by a similar configuration including a lot of high frequency contents. The other two earthquakes were of the comparatively large Magnitude more than 6, but were of the longer epicentral distances. The Fourier spectra of acceleration for the first group are averaged and illustrated in Fig. 7. It is seen that the peak at 3 Hz appears in each spectrum of motions at the foundation, 10 m depth and 40 m depth. Furthermore, the spectra at the 10 m depth have another peak at 6 Hz in both N-S and E-W components. The spectral ratio of motion of the foundation to that of the 10 m depth reveals the pronounced peak at 3 Hz, as found in Fig. 8.

COMPARISON WITH THEORY

In general, the dynamic characteristics of the foundation vibration are roughly evaluated by the dimensionless frequency $\omega_o r_o/V_s$, in which ω_o is the natural circular frequency of the foundation, r_o is the equivalent radius of the contact area with soil and V_s is the S-wave velocity. In the present case, $f_o = 6$ Hz, $r_o = 17.6$ m and $V_s = 250$ m/sec, so that $\omega_o r_o/V_s = 2.65$. This value is too large to use the ordinary mass-spring-dashpot model. In addition, the mass of the foundation is comparable in magnitude with the mass of the excluded soil, so that it might be easily influnced by the stratification of soils. From these reasons, the analysis was conducted by means of the finite element method with the following procedures:

1. The ground layer, which is assumed to be supported on a rigid base, is divided into a number of horizontal sublayers, according to the soil stratification.

2. The analytical model consists of a cylindrical soil column assumed in the ground and the surrounding soils spread laterally, as shown in Fig. 9. The soil column includes the foundation and soil beneath it, and has the cross-sectional area equal to the foundation area.

3. The linearization of displacements in accordance with the finite element method, is applied to the direction of thickness of layers.

4. The soil column is represented by a Timoshenko beam, whose nodal forces and displacements are specified at column circumferences intersecting with interfaces of layers.

5. The dynamic stiffness of reaction of the surrounding ground is obtained in the matrix form by the way such as developed by J. Lysmer(1) in the two-dimensional problems and recently by E. Kausel et al.(2) for the three-dimensional problems.

6. For simplicity, the Poisson's ratio of soils is assumed as 0.5.

7. The internal damping ratio h of soils is taken in the form

$$G_j = G_j^*(1 + i2h), \quad i = \sqrt{-1}, \ h = 0.05 \text{ and } 0.1, \ j = 1, 2, \ldots, N$$

where G_j^* is the shear modulus of soil estimated from the S-wave velocity.

8. The rigid base of the ground layers is assumed for mathematical convenience, while the actual layers are supported on the half-space. Hence, it is better that the sinusoidal input to be imposed on the rigid base is taken in accordance with the bottom response of an one-dimensional shear beam supported by a dashpot, the other end of which is acted by the input $u_g e^{i\omega t}$, as illustrated in Fig. 10.

Two analytical models are presented in Fig. 11. Model II is more detailed, but more costly than Model I. The point A, at which the motion of the foundation was calculated, was so selected as to correspond to the actual measurement point. Fig. 12 shows the theoretical response curves for Model II subjected to the horizontal excitation, comparing with the measured results. Obviously, the theory gives somewhat smaller values than the measured, especially in the range of 6 Hz to 9 Hz of the horizontal component, and it does not indicate another peak at 9 Hz. The possible reasons for these discrepancies are thought to be the decrease of apparent shear modulus of soil beneath the foundation, effects of the backfill and lack of division of layers in the analytical models. The similar comparison of vertical displacements of foundation acted by the vertical excitation is presented in Fig. 13. The theory shows the trend in the measured data. The frequency response of the foundation motion due to the seismic input $u_g e^{i\omega t}$ is shown in Fig. 14, together with that of motion at the ground surface far from the foundation. At this time, Model I was analyzed. For comparison of thus calculated results with the measured, the spectral ratio of motion of the foundation to that of the ground surface are demonstrated in Fig. 15. Both results shows a qualitative agreement.

SUMMARY

The principal results are summerized as follows:

1. The present foundation subjected to the horizontal excitation undergoes the sliding and rocking motion, whose natural frequencies are 6 Hz and 9 Hz for the first and second modes, respectively.

2. Another peak at 3 Hz is produced by the action of the top layer of soil of the thickness 8 m.

3. During earthquakes, the maximum horizontal accelerations at the foundation were approximately 1/2.5 times those of the ground surface and were nearly equal to those at the depth 10 m.

4. The present analysis method is applicable to the practical problems.

REFERENCES

1. John Lysmer and Lawrence A. Drake, "A Finite Element Method for Seismology," Method in Computational Physics, Vol. 11, Academic Press, Inc. 1972

2. Eduardo Kausel, Josè M. Roësset and Günter Waas, "Dynamic Analysis of Footings on Layered Media," Journal of the Engineering Mechanics Division, ASCE, Vol. 101, No. EM5, Proc. Oct. 1975, pp. 679-693

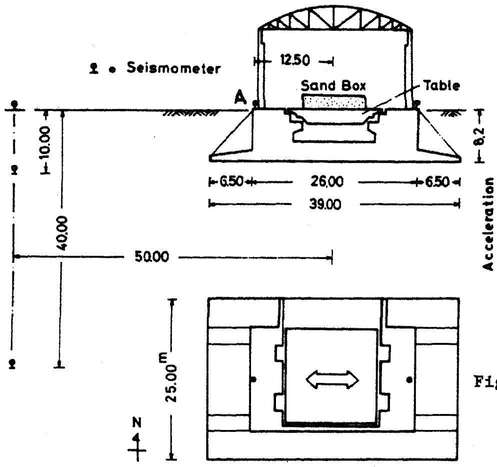

Fig. 1 Outline of Foundation and Location of Seismometers

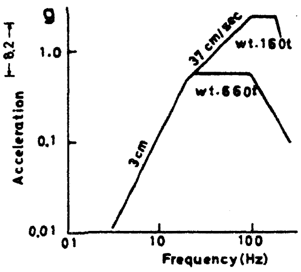

Fig. 2 Limitation on Dynamic Performance, Horizontal

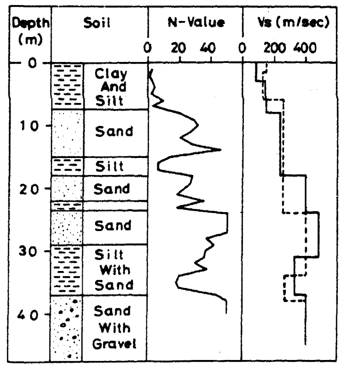

Fig. 3 Soil Profile, N-Values and V_S

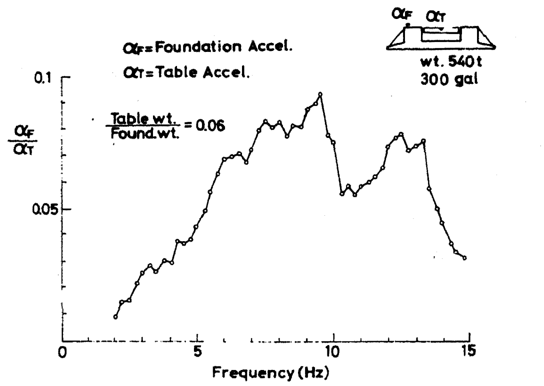

Fig. 4 Acceleration Ratio of Foundation to Shaking Table

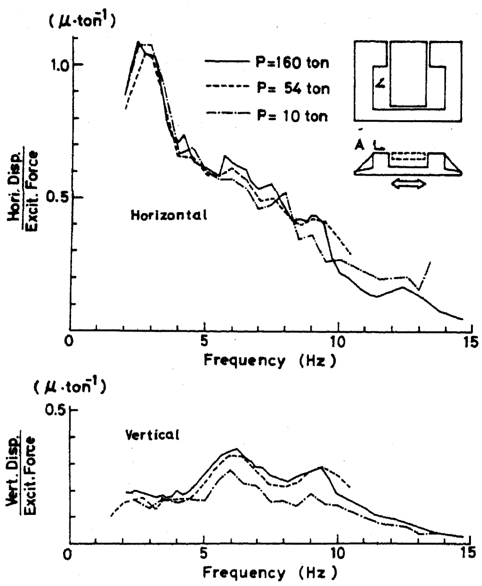

Fig. 5 Resonance Curves for Horizontally Forcing Test

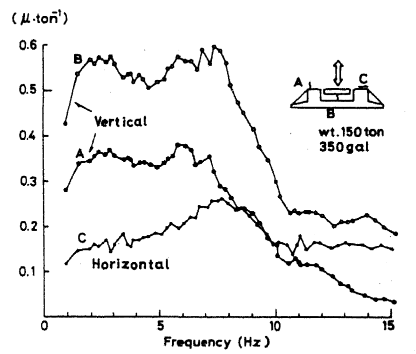

Fig. 6 Resonance Curves for Vertially Forcing Test

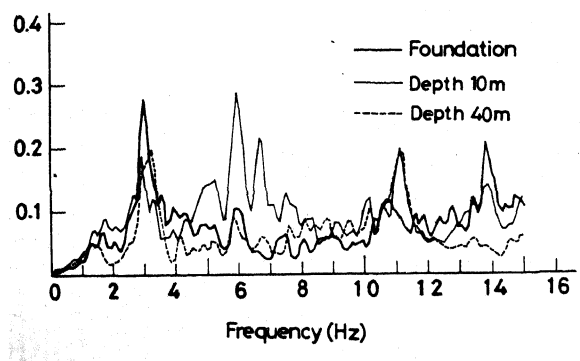

Fig. 7 Averaged Fourier Spectra of Accelerations Measured during Earthquakes, E-W Component

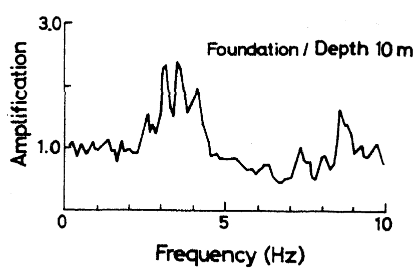

Fig. 8 Spectral Ratio, Found./10 m Depth

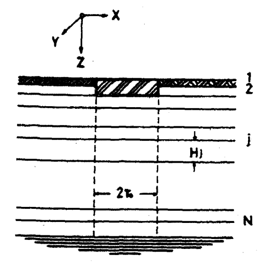

Fig. 9 Mathematical Model

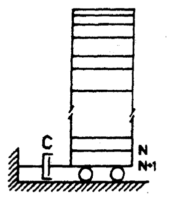

Fig. 10 Shear Beam Supported on Dashpot, as Substitute of Half-Space

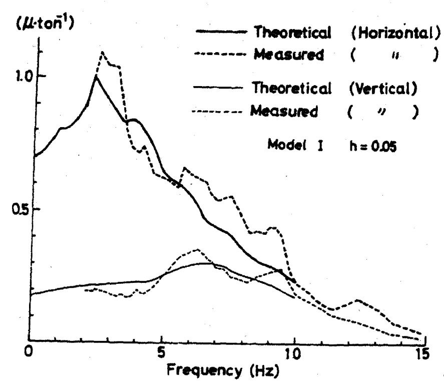

Fig. 12 Comparison of Theoretical Resonance Curves and Measured, Horizontal

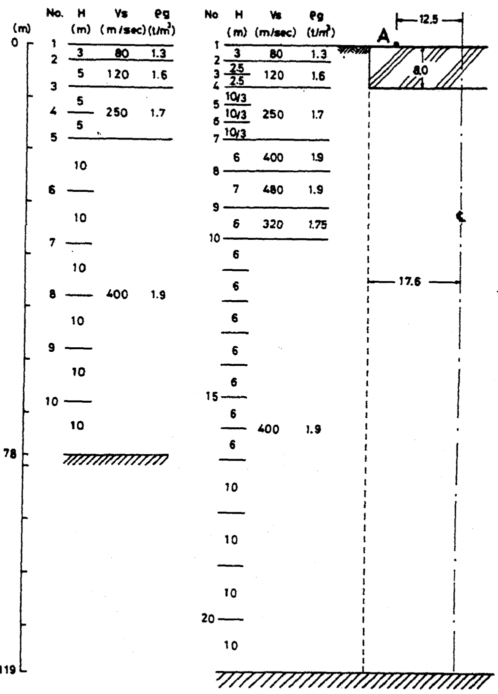

Fig. 11 Analytical Models

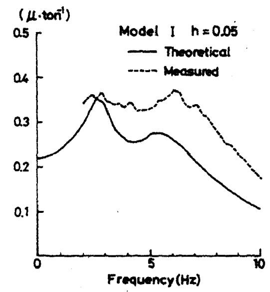

Fig. 13 Comparison of Theoretical Result to Measured, Vertical

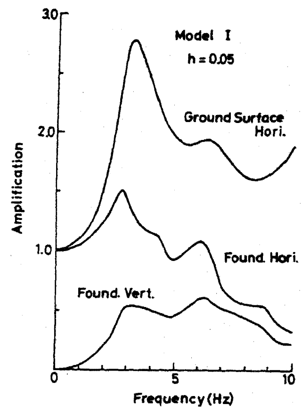

Fig. 14 Theoretical Amplification Spectra of Motions at Foundation and Ground Surface against Input $u_g e^{i\omega t}$

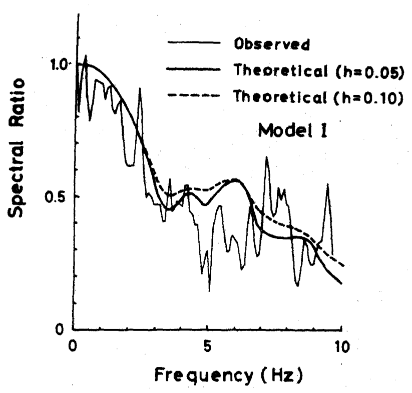

Fig. 15 Spectral Ratio of Horizontal Motions of Foundation to Those of Ground Surface, Theoretical and Measured

Table 1 Earthquake Data and Maximum Accelerations

Date / Distance / Magnitude	Oct. 9, 74 / 19 km / 4.6			Oct. 9, 74 / 19 km / 4.8			Oct. 31, 74 / 14 km / 4.6			Nov. 16, 74 / 100 km / 6.4			Nov. 30, 74 / 700 km / 7.6		
Direction	N-S	E-W	U-D	N-S	E-W	U-D	N-S	E-W	U-D	N-S	E-W	U-D	N-S	E-W	U-D
Found.	-	8.0	11.0		10.0	10.0	13.5	13.1	12.2	14.7	16.8	5.6	9.7	6.7	4.8
Depth 0 m	-	-	-	-	-	-	-	-	-	-	-	-	12.5	11.2	4.2
10 m	7.5	5.8	8.3	11.1	7.2	12.4	18.5	12.0	12.5	12.9	18.5	6.3	8.4	6.9	4.5
40 m	5.6	6.7	6.3	8.1	5.7	7.3	16.0	16.5	8.0	6.3	11.6	5.1	5.0	4.2	3.3

Date / Distance / Magnitude	Feb. 8, 75 / 33 km / 5.5			Feb. 28, 75 / 20 km / 3.7			Mar. 30, 75 / 10 km / 5.4			Apr. 12, 75 / 8 km / 5.0		
Direction	N-S	E-W	U-D	N-S	E-W	U-D	N-S	E-W	U-D	N-S	E-W	U-D
Found.	7.8	8.7	6.0	6.0	8.4	9.0	13.0	18.7	13.0	over	36.8	30.1
Depth 0 m	23.0	24.0	12.6	29.0	32.0	10.8	36.0	41.0	21.0	over	over	36.0
10 m	10.7	9.0	5.2	7.5	15.0	6.0	12.7	17.2	9.0	49.6	31.0	23.9
40 m	4.8	6.0	3.4	5.0	9.0	-	8.8	12.0	7.0	over	over	15.0