Soil-Structure Interaction of the Elevator Tower and of Concrete Footings

by

H. Kishida, K. Matsushita and I. Sakamoto

ABSTRACT

The forced vibration tests and the observation of earthquakes about different types of structure were carried out to study the soil-structure interaction problem during the earthquake. The results of these field tests indicated that the behaviour of the structure during earthquakes was much affected by the supporting system of the foundation and that the computed behaviour of the engineering structures showed a fairly good agreement with the observed ones in the proposed earthquake records on the basis of the record of the ground surface.

Three concrete footings, which were on the ground, supported on a steel pile of 30m in length and embedded in the ground respectively, were located at Matsushiro town. The dimensions were 2.00m in width and length, and 1.25m in height, respectively.

An elevator tower of steel structure was located at Mito City. The upper structure is 80m in height, and the plan of the standard floor is 7m in width and 7.5m in length, respectively. The sub-structure is of reinforced concrete and is 5m in depth. The mat type foundation is 1.5m thick.

The detailed vibratory characteristics of the footings and the tower were obtained from the results of forced vibration tests. Three acceleration records of earthquakes were obtained on the footings and on the ground surface at Matsushiro. Several earthquake records were obtained by strong motion accelerographs at the 22nd floor and at the 1st floor of the tower and on the ground surface 50m apart from the tower at Mito.

The results based on these field tests were as follows:

(1) in the forced vibration tests the embedded footing showed the biggest damping ratio of about 0.50. The damping ratio of 0.133 of the footing with a pile was bigger than that of 0.115 of the footing on the ground. This difference is supposed to be mainly due to the hysteretic and energy dissipation characteristics of the soil.

(2) from the special analysis of the earthquake records and from the response analysis with a rocking model, the input acceleration was estimated to be 80% of ground acceleration in the case of concrete footing on the ground and to be 50% of ground acceleration in the case of elevator tower except near the natural period of the tower. This phenomenon can be explained by the theory of wave reflection.

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H. Kishida[I], K. Matsushita[II] and I. Sakamoto[III]

Synopsis

The forced vibration tests of the elevator tower and of three different types of model concrete footings were carried out and the behaviour of these structures during the earthquakes were observed. The results showed that the damping characteristics of the ground and wave propagation between the soils and the footings are important factors in the problem of soil-structure interaction.

Glossary of Terms

A, B and C	= Names of earthquakes at Matsushiro
D.C.	= Direct-Current type strong motion accelerograph
Fs, Fp and Fe	= Names of concrete footings at Matsushiro and their motions
GS	= Embedded concrete block (earthquake motion of ground surface)
h(force) and h(free)	= Equivalent viscous damping ratio obtained from forced vibration test results and from free vibration ones, respectively
h_1 and h_2	= Damping ratio of first and second mode of vibration
K_h and K_r	= Horizontal and rotational dynamic K value calculated from forced vibration test results, respectively
P_h and P_v	= Horizontal and vertical subgrade reaction coefficient, respectively
S_a (h = 0.05)	= Acceleration response spectrum (damping ratio = 5%)
S_v (h = 0)	= Undamped velocity response spectrum

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T_{b+t} = Coupled period of translational and torsional vibration

T_b = Natural period of translational vibration

T_t = Natural period of torsional vibration

T_1 and T_2 = Natural period of first and second mode of vibration

UG = Earthquake motion of underground

1. Introduction

The difference between the motions of the ground and the foundation of the structure is the most important factor in the soil-structure interaction problem. The types of foundations and change of wave propagation between the soils and the footings are the main factors which affect this problem.

The vibratory characteristics of engineering structures are usually determined by forced vibration or free vibration tests and the behaviour of the structure during the earthquakes is estimated on the basis of the assumed model. The records of the earthquakes are measured mostly on the ground surface or inside the structure, and the effect of the presence of the structure on the ground vibration is usually ignored. Housner (1957) indicated that the length of the structure had an important effect on its behaviour during the earthquakes. Zeevaert (1964) observed the earthquake inside and outside a building and he found that the difference of the motion between these two places was caused by the amplification of the vibration of the subsoil. The observations of the earthquakes in or on the ground and inside the building were carried out by Ohsawa (1966) and he assumed the lumped mass system including the ground and the building.

No previous research work of soil-structure interaction has been carried out to compare the forced vibration test results with the observed value during the earthquakes. The authors carried out the forced vibration tests of the elevator tower and the tests of three different types of model footings and observed the behaviour of these engineering structures. The results showed: (1) The damping characteristics of the ground which results from the hysteretic nature of the soil affects greatly the embedded foundation; and (2) Wave propagation between the soils and the footings can be explained by the wave reflection phenomena. The authors also showed that the computed behaviour of the engineering structures showed a fairly good agreement with the observed ones in the proposed earthquake records on the basis of the record of the ground surface.

2. Field Vibration Tests and Soil Conditions

2-1. Model footing tests at Matsushiro

During Matsushiro earthquake swarm (1966-7), forced vibration tests of three different types of concrete footings were carried out to obtain the fundamental data on the vibratory characteristics of the concrete footings. The soil conditions of the experiment site are shown in Fig. 1. The alluvial soft clay stratum with thickness of about 30 meters lies below the ground level, in which thin loose sand layers are located. Under the alluvium strata, the bed rock layer is situated. The ground water level is 0.5 meters below the surface.

The dimensions of the three model concrete footings are 2.00 meters in width and length, 1.25 meters in height, respectively Each of the footings was supported by the three different types of foundation system. The details of the each footing are as follows:

Fs: Footing on the ground.

Fp: Footing supported on a pile: the pile is made of steel (318 mm in diameter and 6.0 mm in thickness) and is a bearing pile, the tip of which reaches the bed rock layer.

Fe: Footing embedded in the ground.

The earthquake records of the ground surface itself were measured by an accelerograph. The accelerograph was placed on a small concrete block (GS), but the effect of interaction between the small concrete block and the ground is thought to be negligible. The locations of footings and the block are shown in Fig. 2. The geometrical constants and weight of these footings are shown in Table 1.

In the forced vibration tests, an eccentric mass type vibration generator was placed on the footing, the eccentric moment of the generator was 170 kg-cm. The amplitude of the motions of the concrete footings during the vibration tests was measured by two magnet type vibration gauges, and the total displacement of the concrete footings was calculated from the test results The resonance curves were obtained by increasing and decreasing the rotation of the vibration generator, the rotation speed of the motor was kept constant for each separate frequency. The amplitude of the horizontal component was measured at the top of the footings. In the state of resonance, amplitudes of the horizontal and vertical components at the top and at the side of the footings were measured respectively, and the shape of the first mode of the rocking vibration was estimated from these data. Free vibration tests were also varried out, in which a shock was applied to the top of the footing in the horizontal direction.

The resonance curves are shown in Fig. 3 and the shapes of rocking vibration are shown in Fig. 4. Frequencies of resonance and damping ratios of these footings on the basis of forced and free vibration test results are shown in Table 2. In Table 2, h(force) means the equivalent viscous damping ratio which is calculated by the so-called $1/\sqrt{2}$ method of the resonance curve, and h(free) means the viscous damping ratio which is obtained from the logarithmic decrement of the results of free vibration test result.

The natural frequency of Fs which was obtained from forced vibration tests and free vibration tests approximately agrees with that of Fp, and, consequently the stiffness of a pile hardly affects the vibratory characteristics of the footing. The damping ratio of Fp is larger than that of Fs, and the reason of this phenomenon is considered to be as follows: (1) The damping ratio of Fp consists of dissipation of energy into the ground and of hysteretic characteristics of the soil which results from the volumetric change due to the displacement of the soil, (2) The contact surface between the soil and Fp which is closely related with energy dissipation is larger than that of Fs, and Fp causes larger volumetric change due to the displacement of the soil than Fs.

The forced vibration test results of the embedded concrete footing (Fe) show no peak in the resonance curve as can be seen in Fig. 3. It means that Fe has high natural frequency and large damping. The value of the damping ratio of about 0.5 was obtained by the relationship between the vibrating force and the maximum amplitude at resonance condition. However, the value of damping ratio on the basis of free vibration test results was about 0.27. The difference between these two values depends mainly on hysteretic characteristics of the soil which is affected by the amplitude of the concrete footing during vibration, because the damping characteristics of energy dissipation is not affected by the amplitude of the concrete footing during vibration.

On the basis of the assumption of Fig. 5, dynamical spring constants (K) and subgrade coefficients (P) of the soil were obtained from the forced vibration test results. The values are shown in Table 3. The ratio of the horizontal subgrade coefficient to the vertical one (P_h/P_V) of Fp is smaller than that of Fs. It means that the presence of a pile strengthens the vertical component of the subgrade reaction. The values of Fe were not obtained, because the distribution of reaction on the sides of the embedded concrete footing was not measured.

2-2. Elevator tower tests at Mito

The forced vibration tests of the elevator tower were carried out to obtain the fundamental data on the vibratory characteristics of the elevator tower, which is a steel structure located at Mito City. The section and the plan of the tower are shown in Fig. 6. The upper structure is 81 meters high and consists of 22 stories. The plan of the standard floor is 7.0 meters in width, 7.5 meters in length, respectively. Three elevator machines are placed at 22nd floor. The walls are made of precast concrete plates. The substructure is of reinforced concrete and is 5.00 meters deep. The foundation which is of the mat type is 1.50 meter thick and 21.00 meters in width and length, respectively. The foundation rests on the gravel layer. The soil conditions are also shown in Fig. 7. The microtremors at this site had been measured and the predominant period was about 0.2 sec.

The forced vibration tests were carried out by two eccentric mass type vibration generator which were placed on the both sides of the 22nd floor of the tower as can be seen in Fig. 6. The eccentric moment was 1640 kg-cm or 5800 kg-cm. Amplitude of the tower during the vibration tests was measured with two magnet type vibration gauges and the acceleration of the tower during the tests was estimated from the test results.

The resonance curves were obtained by increasing and decreasing the rotation of the vibration generator, the rotation of the motor was kept constant for each separate frequency. The first and second natural period of both translational and torsional mode are included in the obtained resonance curves. The translational mode is the one in reference to the 7 m direction. The natural modes of the tower during the vibration were measured as follows: one vibration gauge was fixed at the 22nd floor and the other was set on floor by floor in the resonance state of each mode, and the ratio of the displacement of all floors to the 22nd floor was obtained. Free vibration tests were carried out by suddenly stopping the motion of the vibration generator in the state of resonance. Some micro-tremors at the 22nd floor were also measured.

The resonance curves and shapes of each natural mode obtained from the forced vibration test results are shown in Fig. 8. The resonance periods of each mode were determined from the resonance curves and the values of the resonance periods are shown in Table 4. In general, the natural period is approximately equal to the resonance period, but in this experiment, these two values do not coincide well due to the eccentric shape of the plan as can be seen in Fig. 6. The reason is as follows: vibrations of pure translational mode and that of torsional mode are thought to be coupled with each other during the forced vibration test, the coupled period of translation and torsion is greater than the period of pure translation, and, consequently, the natural period is shorter than the resonance period. With reference to the first natural period of the pure translational mode, the relation of natural periods between pure translation and torsion is expressed by the following equation on the basis of the assumed simple model in Fig. 9.

$$T_{b+t}^2 = T_b^2 + T_t^2$$

where T_{b+t}: coupled period of translation and torsion

T_b : period of pure translation

T_t : period of torsion

With this equation the first natural period of pure translation is tentatively computed to be 1.55 sec. In the cases of natural periods of higher translational or torsional vibration, matters are more complicated.

Damping ratios for each mode were obtained from the resonance curves of forced vibration and also from the results of free vibration tests. The method of calculation is the same as the one in the tests of Matsushiro. The values of the damping ratios of each mode are shown in Table 4. The structure of the tower is flexible, and there is very little displacement at the first floor, both in horizontal and in vertical component occurred during the vibration tests, consequently, the damping due to energy dissipation into the ground is considered to be very small. The obtained damping is mostly due to the internal damping of the structure itself

3. Behaviour of the Model Footings and the Elevator Tower during Earthquakes

3-1 Behaviour of the model footings during earthquakes

Three accelerographs were placed on the surface of concrete footings, Fs, Fp and Fe to measure the behaviour of the concrete footings One accelerograph (which measured the behaviour of the ground motion during earthquakes) was also placed on the surface of the small concrete block (GS) embedded in the ground. Three small earthquake records which were named A, B and C were obtained. The maximum accelerations on the ground surface are A = 5.2 cm/sec^2, B = 4.4 cm/sec^2 and C = 14.0 cm/sec^2 respectively The epicenters of these three earthquakes were at Matsushiro Town and the magnitude of the earthquakes was somewhat less than IV on the Gutenberg-Richter scale. The intensity scale of the earthquakes on the basis of Japanese Meteorological Agency is about II or I at the experiment site. The distance between the experiment site and the epicenter was about 5 kilometers. One example of the records of the earthquake of December 20, 1966 (B earthquake) is shown in Fig. 10.

3-2. Behaviour of the elevator tower during earthquakes

Behaviour of the elevator tower during earthquakes are measured with three direct-current type strong motion accelerographs, two of which are placed at the 1st floor and at the 22nd floor of the tower, and the other is placed on the concrete block (GS) embedded in the ground surface. The distance between the tower and the concrete block was 50 meters. Several earthquake records were obtained. One typical record which was measured during the earthquake of November 19, 1967, showed maximum acceleration of 45 gal on the ground surface. The epicenter is 70 kilometers apart from the site in ES direction. The magnitude of the earthquake was about 4.3, and the intensity scale at Mito City was about II. The duration of the ground motion was about 20 seconds. The typical accelerograms are shown in Fig. 11.

4. Analysis of Test Results and of Field Observations

4-1. Behaviour of model footings at Matsushiro

Response spectrum curves with zero and non-zero damping were computed for all obtained records of the earthquakes. Fig. 12 shows S_a (h = 0.05) of the ground motion which was measured on the embedded small concrete block. One or two peaks can be found in every curve, and the predominant period of these earthquake records is the value of about 0.18 sec. The first natural period of this soft clay stratum of 30 meters deep in Fig. 1 is estimated to be about 2.0 sec. on the basis of the velocity of shear wave propagation, but no peak for the period of 2.0 sec. was found. The records of Matsushiro earthquakes, the epicenters of which are within about 5 kilometers of the observation site, show rather a kind of shock motion than usual earthquake motion. It means that the waves of shorter period are predominant than those of first natural period of the ground. These waves of short period are presumably due to the original waves which propagated from the bed rock layer to the ground surface.

In the earthquake records of Fs and Fp, the natural periods of these two concrete footings are approximately equal, but the phase angles of these two records are different. The same phenomenon was also found in the earthquake records of Fe and GS. The distance between Fs and Fp is about 20 meters and the distance between Fe and GS is about 30 meters. When the time axis of the earthquake record of GS is shifted by 0.2 second to right direction, the earthquake records of GS and Fe resulted in the same shape. The same relation was also obtained in the earthquake records of Fs and Fp. The value of about 0.2 second approximately coincides with the time lag of the wave propagation between the two places.

The characteristics of the motion of Fs, Fp and Fe during earthquakes were clarified by computing the ratio of S_v (h = 0) of Fs, Fp and Fe to S_v (h = 0) of GS. The computed results are shown in Fig. 13. The period of the peak points in the curves of Fs/GS and Fp/GS are about 0.24 second for all earthquakes in Fig. 13. These periods show a fairly good agreement with the natural periods of about 0.21 second, which were obtained in the forced vibration tests The periods of the peak points in the curve of Fe/GS are not distinctive, this means that the movement of the concrete footing embedded in the ground, Fe, resembles nearly the movement of the ground because of its high frequency and because of large damping.

The behaviour of the model footing, Fs, during earthquakes was calculated by the assumed model in Fig. 14. Spring constants and subgrade coefficients of the soil are the same values in Table 3. The calculated results at the top of the footing showed considerable difference from the observed values as can be seen in Fig. 15. The modifications which decrease the input acceleration without changing the damping ratio were made to coincide with these two values. S_a (h = 0.05) was computed with assumption of 80% of ground acceleration as input acceleration. The curve of S_a approximately equals to the one that was computed with observed acceleration in Fig. 15. The computed result shows the following tendency; the input earthquake wave which is transmitted from the ground beneath the footing to the base of the footing shows smaller value than the one recorded on the ground surface. This decrease of the input earthquake is demonstrated by the change of wave propagation, which occurs at the boundary plane between the soils and the footing.

4-2. Behaviour of elevator tower at Mito

Auto corelation functions ($\phi(\tau)$) and velocity response spectrum curves with zero damping (S_v (h = 0)) were computed for the earthquake records obtained at the top and base of the tower and on the ground. The auto corelation functions are shown in Fig. 16 and the velocity spectrum curves are shown in Fig. 17.

The ground motion record indicates that a regular wave with period of 0.14 second appears clearly in $\phi(\tau)$, however, the predominant period of microtremor at this site is 0.2 second. The difference between these two values results mostly from the pseudo-resonance of the concrete block or from the local effect of the ground.

The record at the 22nd floor is expected to show the characteristics of the tower itself. The computed results of the record at the 22nd floor show that a wave with period of 0.4 sec. appears very regularly in $\phi(\tau)$ and that a peak at 0.4 second also appears in S_v. This period corresponds to the second natural period of translational vibration of the tower. A high peak at 1.5 second in S_v shows a fairly good agreement with the first natural period of translational vibration of the tower. In the cases of auto corelation function and the obtained earthquake record, the wave which has the period of 0.4 second appears during the first 20 seconds of the earthquake and the wave with the period of 1.5 second is not clear at that stage. Following that stage, however, the period of 1.5 second becomes more predominant to the period of 0.4 second. This phenomenon indicates that the elevator tower vibrated in its second mode during the first 20 seconds of the earthquake and that the first mode appeared after 20 seconds.

Another peak which appears at 0.6 second corresponds to the first natural period of torsional vibration of the tower.

The earthquake record measured at the first floor, which approximately represents the motion of the basement, shows only one peak of 0.6 second in the curve of S_V. The periods of the first and second translational vibration of the tower do not indicate any peak in the curve of S_V at the first floor. The reason is as follows: the location of the strong motion accelerograph does not coincide with the center of the rotation, and therefore, the torsional motion of the tower affects the accelerograph record of the horizontal components.

Fig. 17 also shows the velocity response spectrum on the basis of the assumed accelerogram at the top of the gravel layer (UG) which was computed on the basis of the theory of multiple reflection. Velocity of the shear wave propagation in the surface layer was assumed to be 40 m/s from the standard penetration test results, the depth of the layer is 4.0 meters, and impedance ratio between the surface layer and the ground layer is assumed to be 0.1.

The S_V curves at the top of the gravel layer is similar to that of one half of the ground surface record overall periods longer than 2.0 sec., but the S_V curve at the first floor shows the similar tendency to that at the top of the gravel layer or to one half of that at ground surface. The difference between the S_V curves of the first floor and other two curves are large at periods shorter than 1.0 second. These results indicate the following: (1) in the long period range, longer than 2.0 seconds, the behaviour of the tower is estimated on the basis of the assumed lumped-mass system with the input earthquake record which is one half of the record on the ground surface,. (2) in the short period range, smaller than 10 seconds, the behaviour of the tower causes the big difference between the observed and estimated values, which presumably depends on the coupling effect between the structure and the ground.

5. Conclusion

(1) The embedded footing and the footing with a pile give the larger values of the damping ratio than the footing which rests on the ground.

(2) The time lag due to the propagation of the earthquake waves was observed between model footings at Matsushiro.

(3) The second mode of the translational vibration of the tower appeared in the record obtained at the 22nd floor during the first stage of the earthquake and the first mode appeared during the last stage of the earthquake.

(4) The first mode of the torsional vibration of the tower affected the movement of the first floor of the tower.

(5) The computed behaviours of the engineering structures show a fairly good agreement with the observed ones in the following input earthquake records:

(a) as for the footing rested on the ground, eighty percent of the earthquake record was adopted,

(b) as for the elevator tower, fifty percent of the earthquake record was adopted except near the natural periods of the tower.

Acknowledgement

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Tab. 1 Mechanical constants of footings

Base area	4.00 m^2
Weight	12.0 ton
Mass	1.28 $ton \cdot sec^2/m$
Moment of inertia	0.638 $ton \cdot sec^2 . m$
Height of center of gravity	0.67 m

Tab. 2 Frequencies and damping ratios of footings

	Fs	Fp	Fe
F (force)	4.8 c/s	4.7	-
F (free)	5.2	5.0	7.3
h (force)	0.115	0.133	-
h (free)	0.12	0.15	0.27

Tab. 3 Dynamic K values and subgrade reaction coefficients

	Fs	Fp
Horizontal dynamic K value K_h (10^4 kg/cm)	1.75	1.56
Rotational dynamic K value K_r (10^8 kg.cm)	1.74	2.33
Horizontal subgrade reaction coefficient P_h kg/cm^3	0.44	0.39
Vertical subgrade reaction coefficient P_v kg/cm^3	1.31	1.75
The ratio of P_h to P_v P_h/P_v	0.33	0.23

Tab. 4 Resonance periods and damping ratios of elvator tower

	Translational	Torsional
T_1	1.72 sec.	0.75
T_2	0.39	0.24
h_1	0.0065	0.013
h_2	0.019	0.028

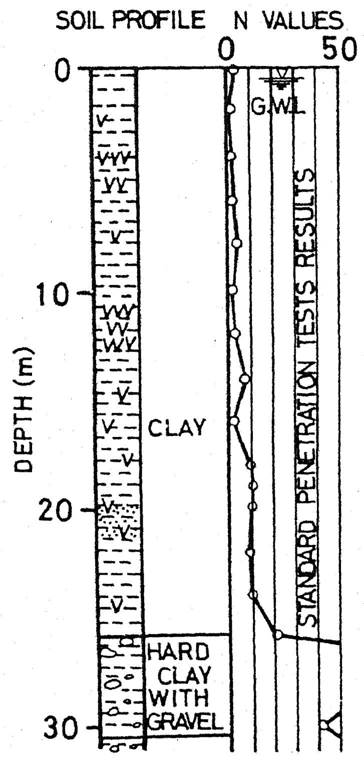

FIG.1 SOIL PROFILE AT MATSUSHIRO

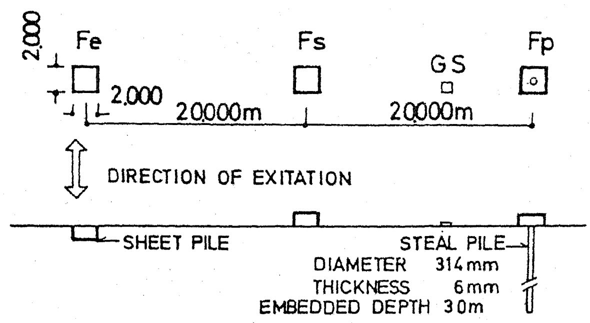

FIG.2 LOCATION OF FOOTINGS

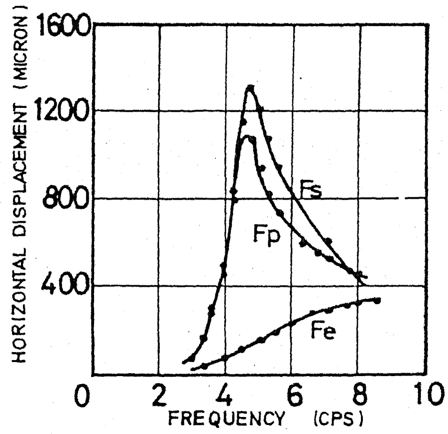

FIG.3 RESONANCE CURVES OF FIRST ROCKING MODE

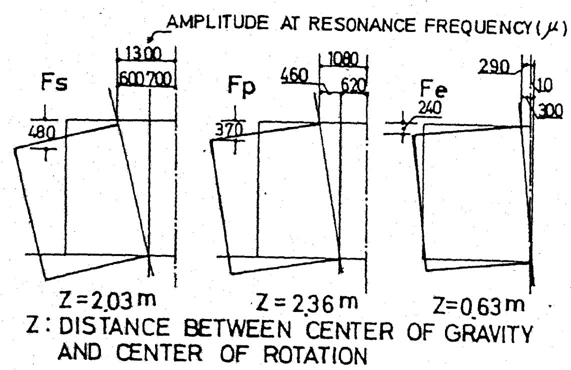

FIG. 4 SHAPES OF FIRST ROCKING MODE

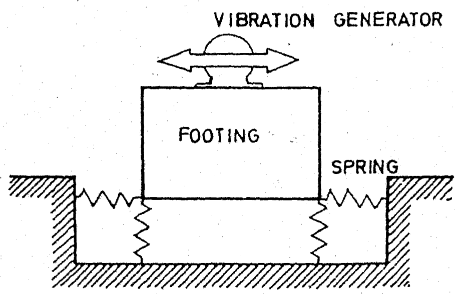

FIG.5 ROCKING MODEL FOR COMPUTING DYNAMIC K VALUE

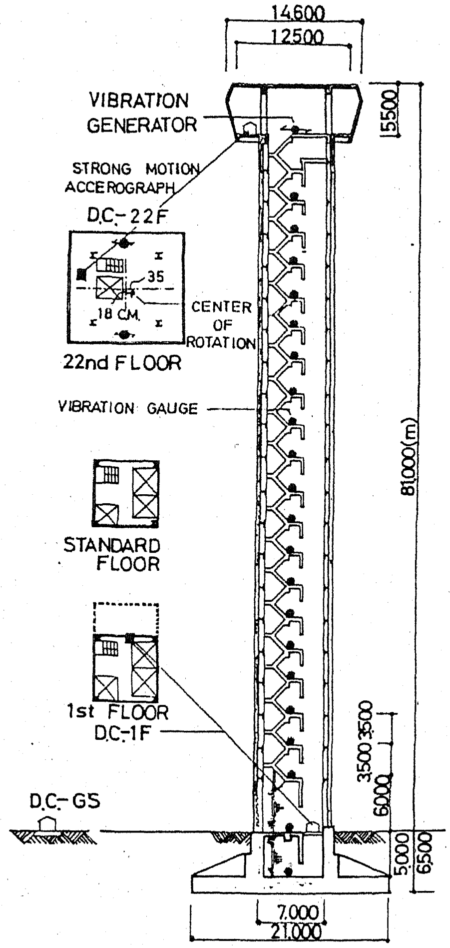

FIG. 6 SECTION AND PLAN OF ELEVATOR TOWER

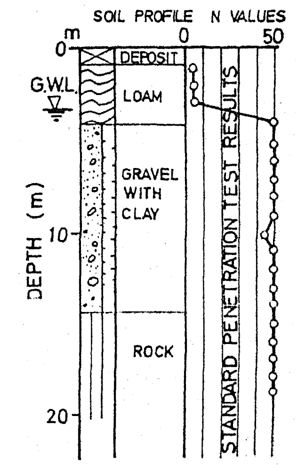

FIG. 7 SOIL PROFILE AT MITO

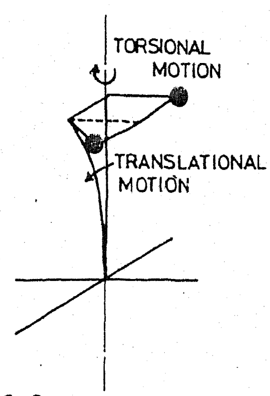

FIG. 9 MODEL FOR COMPUTING Tb

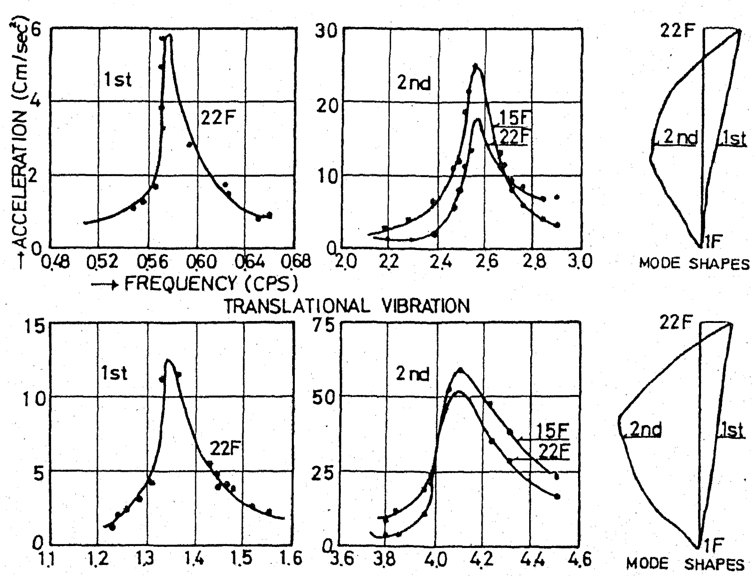

FIG. 8 RESONANCE CURVES AND SHAPES OF VIBRATION

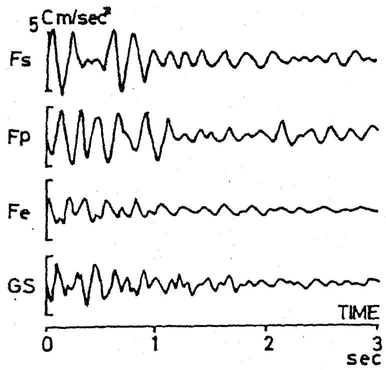

FIG.10 EARTHQUAKE RECORDS
(B AT MATSUSHIRO)

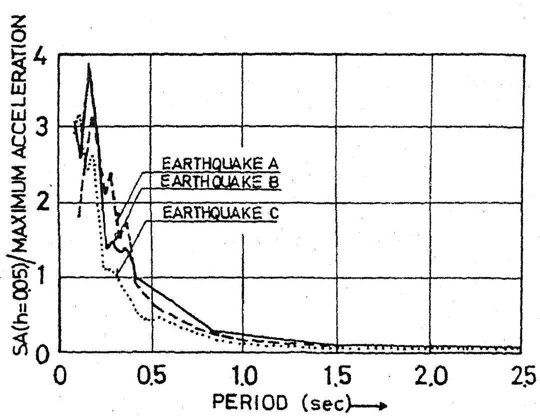

FIG.12 NORMALIZED ACCELERATION SPECTRUM OF GS
(MATSUSHIRO)

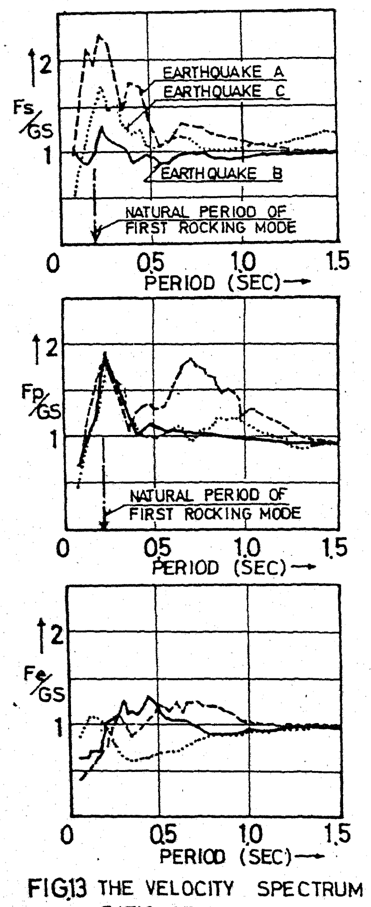

FIG.13 THE VELOCITY SPECTRUM RATIO OF FOOTING TO G-
-ROUND SURFACE

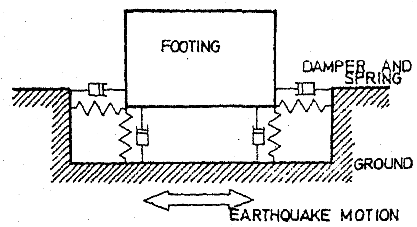

FIG.14 ROCKING MODEL FOR COMPUTING
RESPONSE TO EARTHQUAKE

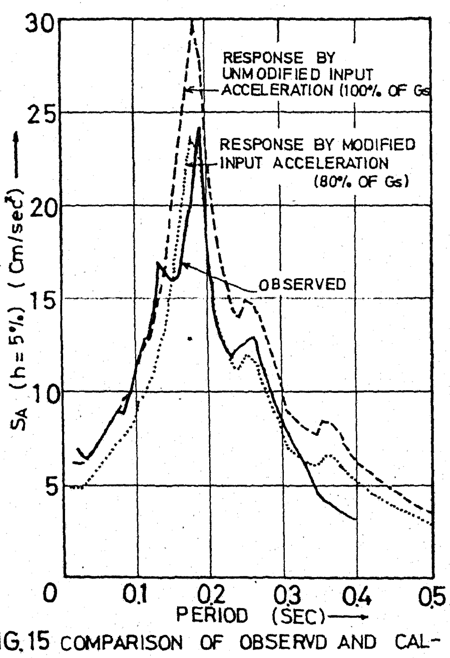

FIG.15 COMPARISON OF OBSERVD AND CAL-
-CULATED ACCELERATION SPECTRUM

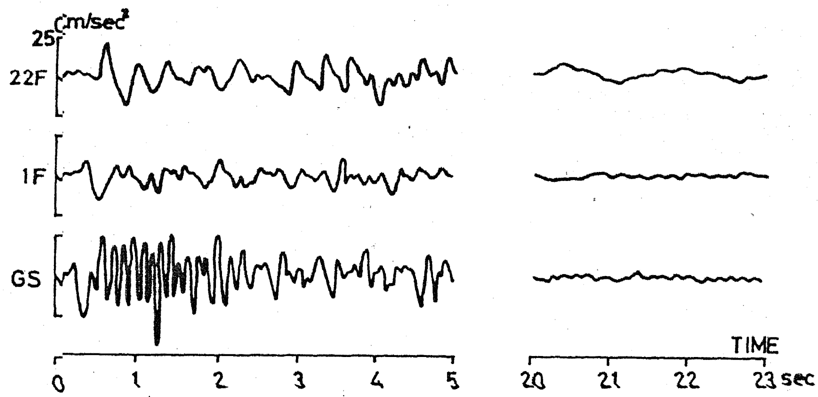

FIG.11 EARTHQUAKE RECORDS (AT MITO)

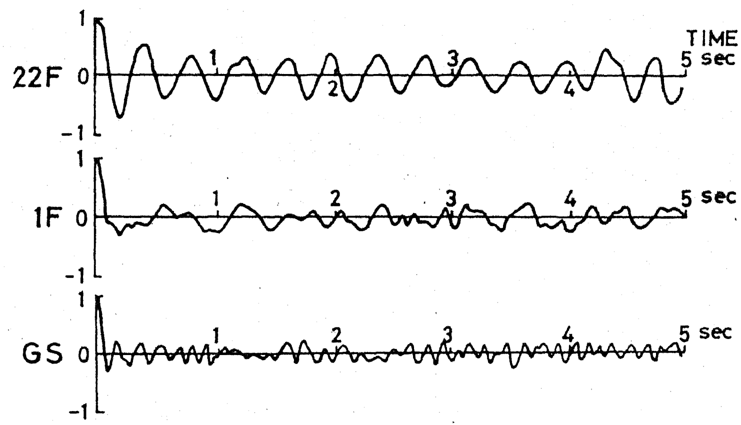

FIG. 16 AUTO-CORRELATION FUNCTIONS OF EARTHQUAKE RECORDS (MITO)

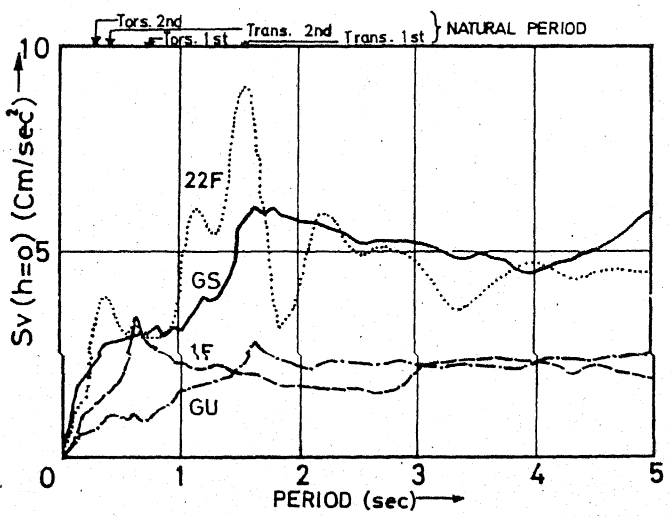

FIG. 17 COMPARISON OF VELOCITY SPECTRA AT VARIOUS POINTS