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Quake-response modelling on an actual building and its resultant consideration compared with measurements

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ABSTRACT: The author treats a problem of vibratory modelling for earthquake-response of an actual 19 storied pure frame-structure with its height of about 65 meters above ground and a massive rigid body embedded 15 meters underground, which is a large parallel-epiped concrete-box containing two basements and soil mass within. As to its surrounding situation it is encircled near by buildings with five to ten stories constructed by reinforced concrete. On the modelling method in this case, the author follows the theory based on the formula derived by Dr. H. Tajimi (Ref. 1). It deals with dynamic analysis of a multi-mass structural system with a rigid body embedded in an elastic stratum inclusive of the effect of radiation damping between soil and rigid body interaction. In order to simplify the case, when applying the embedded parallelepiped body for Tajimi's theory, it is approximately replaced as an equivalent circular cylindrical body. On the other hand, for gaining the actual earthquake-response behavior an automatic measuring system controlled by two microcomputers has been installed (Ref. 2). In this place the simulated structural response-waves vibrated by the actual waves acquired at the level of 59 meters underground are compared with the measured waves at the level of GL-24m, GL-12m and GL+60m. Finally the author concludes the modelling after Tajimi's theory will be able to give a comprehensible approximation when compared with the actual behavior as a whole, if the parameters related would be decided properly to the structural system.

1. INTRODUCTION.

The general features of the structural model treated here are as shown in Fig. 1. The whole system of the structure is a 19 storied pure frame with a parallelepiped rigid concrete box underground. Some details are explained as constituted by a concrete box with 2 basement-stories inside enclosed by outer walls of thickness 0.8m with the plan sizes of 26.75mx49.15m and the depth of 15. meters, 4 storied steel reinforced concrete frames with the plan sizes of 26.75mx46.70m and the height of 3.75 meters, and 15 storied pure steel frame with the plan sizes of 16.30mx37.8m and average height of 3.4 meters inclusive of 2 storied pent-houses as shown in Fig. 2. As for the surface layer of soil, around the site region the geophysical and geological characteristics are thought to be classified as having 150 to 450 meters per second of S-wave and 300 to 900 meters per second of P-wave for its deposited sand strata constituted almost by fine and medium size particles with a thin hard Tokyo gravel layer as shown in Fig. 3. The fundamental characteristic values on its 1st natural periods caluculated of the upper naked structure itself at the fixed state of its base of the 1st story without embedded body are 1.32 seconds for the transversal direction (NS) and 1.20 for the

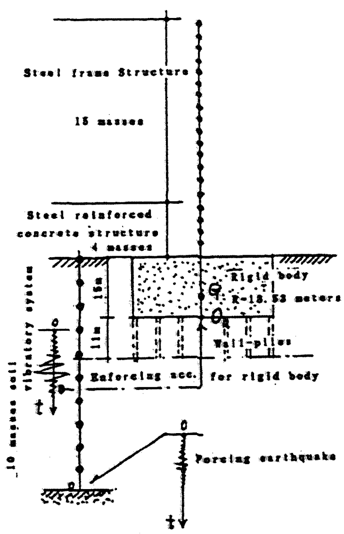


Figure 1. Configuration of structural system

longitudinal (EW), but as their observational values through microseismograph are 1.13 seconds and 1.00 inclusive of rocking effects of the rigid body respectively, showing clearly the influence by additional stiffnesses of finish materials such as inner curtain wall panels and outer precast concrete walls with glass-window etc. (Table 1). On the

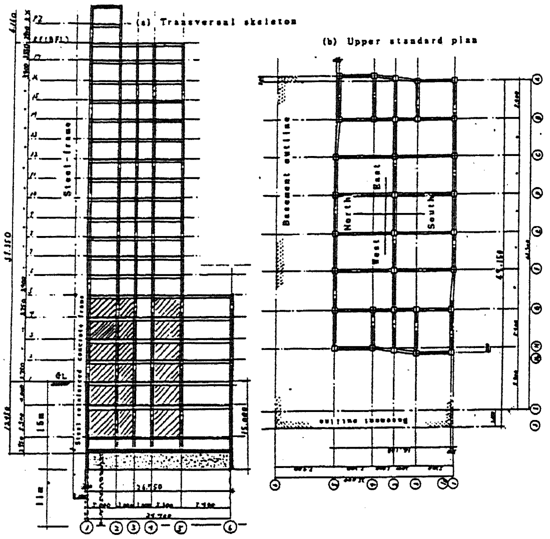


Figure 2. Structural dimensions

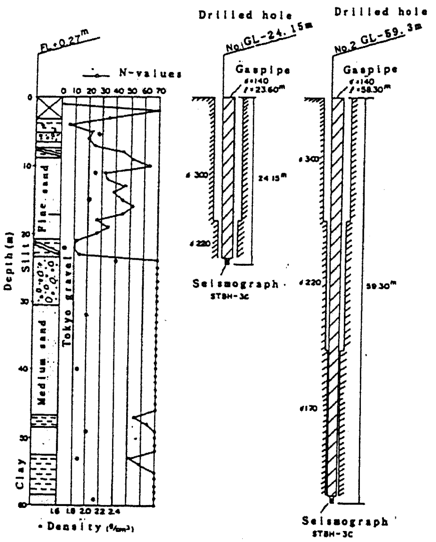


Figure 3. Soil strata by boring test and seismograph-holes

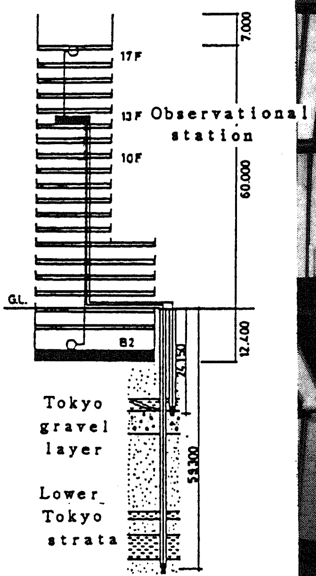
other hand, for the purpose of comparison of results by the theoretical analyses with those by the real behavior, an automatic data acquisition system for seismographs has been established for these years as shown in Fig.4. In the system, 4 sets of seismographs with 3 components through total 12 channels are set

Table 1. Characteristic periods of the upper structure without rocking

Direction	Fundamental periods of upper frame without rocking					With rocking
	1st period	2nd period	3rd period	4th period	5th period	Measured 1st
N-S	1.3166 sec 0.8790	0.4434 sec 0.2956	0.2597 sec 0.1731	0.1880 sec 0.1253	0.1538 sec 0.1015	1.130 sec
E-W	1.1941 sec 0.7940	0.3178 sec 0.2114	0.2302 sec 0.1535	0.1873 sec 0.1248	0.1421 sec 0.0950	1.000 sec

Note: Numbers of period in upper line are those for the naked frame, and those in lower line are for the frame with stiffnesses multiplied by 2.25 times possibly added through inner and outer finishes.

at the place of GL-59.meters, -24.m, -12.5m on the 2nd basement and GL+60.m of the 18th floor being controlled of their synchronization and data-acquisition by two microcomputers (Photo.1). In this paper the quake-responded results on waves simulated under the conditions of parametric changes as to dampimg ratios of frame and soil, poisson's ratio and velocities of S- and P-waves of soils are compared with the actually acquired waves.



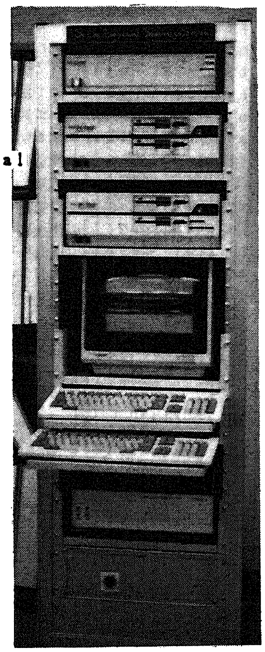


Figure 4. Installation of seismographs

Photo 1. Vibratory measuring system

2 OUTLINE OF THE MODELLING ON THE STRUCTURAL SYSTEM.

For the vibratory modelling of this joint system of upper framework with the underground concrete box structure, Dr.H.Tajimi's theory has been adopted and solved by numerical computation system developed by the author et al. using the method of frequency response analysis by Fourier Transformation. Tajimi's analytical formula is summarized as a multi-mass vibratory system with a circular cylindrical rigid body embedded in a surface soil layer on bed rock applying the 3-D elasticity, which effects radiational and dynamic side-pressure dampings to the structural system through interaction between rocking motion of the rigid body and quaking motion of the surface soil by the earthquake given to the base. The outline of the theory with some part added for computing is briefed refering to Fig. 5-6 as follows;

1. fundamental motion equations on this problem are written by 3-D elasticity refering to Fig. 5,

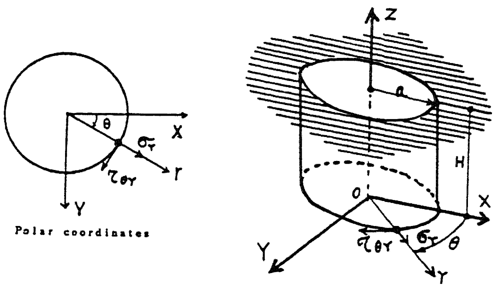
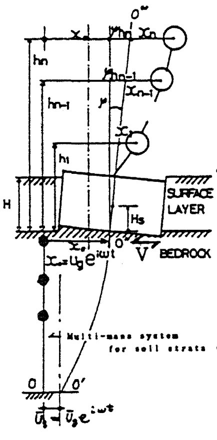


Figure 5. Embedded body in Tajimi's theory



$$\frac{\lambda+2\mu}{\rho}=C_L^2 \quad \frac{\mu}{\rho}=C_T^2 \quad \frac{\mu+i\mu'\omega}{\rho}=C_T^2\left(1+i\frac{\mu'}{\mu}\omega\right)$$

$$C_T\frac{\pi}{2H}=\omega_g \quad \sqrt{n^2\left(1+i\frac{\mu'}{\mu}\omega\right)-\left(\frac{\omega}{\omega_g}\right)^2}=\xi_n^2 \quad \frac{\mu'}{\mu}=\frac{2h_g}{\omega^g}$$

- Surface layer
  - λ, μ :Lame's constants
  - L_T, L_L :Velocities of S- & P-waves
  - ρ :Mass density
  - μ' :Resistg. force per unit velocity
  - ω_g :1st angular velocity
  - h_g :Damping constant
  - K , K :Modified Bessel functions
  - $\ddot{u}$:Enforcg. acc. for soil vibr. system
  - $\ddot{x}_o$:Enforcing acc. for rigid body
  - φ :Rotational displ. of rigid body
  - V :Shearg. force at rigid body base
  - x_i :Horizon. displ. from the O-O axis
- Bottom layer
  - L , L :Velocities of S- & P-waves
  - ρ_b :Mass density
  - ν_b :Poisson's ratio
  - k :Rotat. spring for rigid body base

Figure 6. Multi-masses system after Tajimi's theory

Vibratory equations on 3-D elastic solid

$$(\lambda+2\mu)\frac{\partial}{\partial r}\left[\frac{1}{r}\frac{\partial}{\partial r}(ru_r)+\frac{1}{r}\frac{\partial u_\theta}{\partial\theta}\right]-\mu\frac{1}{r}\frac{\partial}{\partial\theta}\left[\frac{1}{r}\frac{\partial}{\partial r}(ru_\theta)-\frac{1}{r}\frac{\partial u_r}{\partial\theta}\right]$$
$$=\left(\rho\frac{\partial^2}{\partial t^2}-\mu\frac{\partial^2}{\partial z^2}-\mu'\frac{\partial^3}{\partial t\,\partial z^2}\right)u_r-\rho u_g\cos\omega^2 e^{i\omega t} \qquad (1)$$
$$(\lambda+2\mu)\frac{1}{r}\frac{\partial}{\partial\theta}\left[\frac{1}{r}\frac{\partial}{\partial r}(ru_r)+\frac{1}{r}\frac{\partial u_\theta}{\partial\theta}\right]+\mu\frac{\partial}{\partial r}\left[\frac{1}{r}\frac{\partial}{\partial r}(ru_\theta)-\frac{1}{r}\frac{\partial u_r}{\partial\theta}\right]$$
$$=\left(\rho\frac{\partial^2}{\partial t^2}-\mu\frac{\partial^2}{\partial z^2}-\mu'\frac{\partial^3}{\partial t\,\partial z^2}\right)u_\theta+\rho u_g\sin\theta\,\omega^2 e^{i\omega t}$$

2. solutions for u_r, u_θ are assumed as some series by modified Bessel functions with unknown coefficients A_n, B_n to be decided by the condition of equality on displadement of rigid body and soil at the same point on r=a,

Assumed solutions using modified Bessel functions

$$u_r=\sum_{n=1,3,5\cdots}^{\infty}\left\{-A_n\left[\frac{1}{r}K_1\left(\frac{\xi_n\omega_g r}{C_L}\right)+\frac{\xi_n\omega_g}{C_L}K_0\left(\frac{\xi_n\omega_g r}{C_L}\right)\right]+\frac{B_n}{r}K_1\left(\frac{\xi_n\omega_g r}{C_T}\right)\right.$$
$$\left.+\frac{4}{n\pi}\frac{u_g}{\xi_n^2}\left(\frac{\omega}{\omega_g}\right)^2\right\}\cos\theta\sin\frac{n\pi z}{2H}e^{i\omega t} \qquad (2)$$

$$u_\theta=\sum_{n=1,3,5\cdots}^{\infty}\left\{-\frac{A_n}{r}K_1\left(\frac{\xi_n\omega_g r}{C_T}\right)+B_n\left[\frac{1}{r}K_1\left(\frac{\xi_n\omega_g r}{C_T}\right)+\frac{\xi_n\omega_g}{C_T}K_0\left(\frac{\xi_n\omega_g r}{C_T}\right)\right]\right.$$
$$\left.-\frac{4}{n\pi}\frac{u_g}{\xi_n^2}\left(\frac{\omega}{\omega_g}\right)^2\right\}\sin\theta\sin\frac{n\pi z}{2H}e^{i\omega t}$$

A , B & gained under given displt. conditions

$$A_n=-\frac{a}{\chi_n}\left[2K_1\left(\frac{\xi_n\omega_g a}{C_T}\right)+\frac{\xi_n\omega_g a}{C_T}K_0\left(\frac{\xi_n\omega_g a}{C_T}\right)\right]\left[\frac{8\varphi_0 H}{\pi^2}\frac{(-1)^{\frac{n-1}{2}}}{n^2}-\frac{4}{n\pi}\frac{u_g}{\xi_n^2}\left(\frac{\omega}{\omega_g}\right)^2\right]$$
$$B_n=-\frac{a}{\chi_n}\left[2K_1\left(\frac{\xi_n\omega_g a}{C_L}\right)+\frac{\xi_n\omega_g a}{C_L}K_0\left(\frac{\xi_n\omega_g a}{C_L}\right)\right]\left[\frac{8\varphi_0 H}{\pi^2}\frac{(-1)^{\frac{n-1}{2}}}{n^2}-\frac{4}{n\pi}\frac{u_g}{\xi_n^2}\left(\frac{\omega}{\omega_g}\right)^2\right] \qquad (3)$$
$$\chi_n=\left[K_1\left(\frac{\xi_n\omega_g a}{C_L}\right)+\frac{\xi_n\omega_g a}{C_L}K_0\left(\frac{\xi_n\omega_g a}{C_L}\right)\right]\left[K_1\left(\frac{\xi_n\omega_g a}{C_T}\right)+\frac{\xi_n\omega_g a}{C_T}K_0\left(\frac{\xi_n\omega_g a}{C_T}\right)\right]$$
$$-K_1\left(\frac{\xi_n\omega_g a}{C_L}\right)K_1\left(\frac{\xi_n\omega_g a}{C_T}\right)$$

3. using the components of $\sigma_r, \tau_{r\theta}$ to the vibratory direction, rocking moment M on the rotational axis for the base is formulated by integration,

Stresses of $\sigma_r, \tau_{r\theta}$

$$\sigma_r=\lambda\left[\frac{1}{r}\frac{\partial}{\partial r}(ru_r)+\frac{1}{r}\frac{\partial u_\theta}{\partial\theta}\right]+2\mu\frac{\partial u_r}{\partial r} \qquad \tau_{r\theta}=\mu\left(\frac{\partial u_\theta}{\partial r}-\frac{u_\theta}{r}+\frac{1}{r}\frac{\partial u_r}{\partial\theta}\right)$$

Rocking moment by interactive soil pressures

$$M=\int_0^H\left[\int_0^{2\pi}(\sigma_r|_{r=a}\cos\theta-\tau_{r\theta}|_{r=a}\sin\theta)a\,d\theta\right]z\,dz$$
$$=-\sum_{n=1,3,5\cdots}^{\infty}a^2\rho\pi\left(\frac{2H}{n\pi}\right)^2(-1)^{\frac{n-1}{2}}\xi_n^2\omega_g^2\Omega_n\left[\frac{8\varphi_0 H}{\pi^2}\frac{(-1)^{\frac{n-1}{2}}}{n^2}-\frac{4}{n\pi}\frac{u_g}{\xi_n^2}\left(\frac{\omega}{\omega_g}\right)^2\right]e^{i\omega t}$$

$$\Omega_n=\frac{1}{\chi_n}\left[4K_1\left(\frac{\xi_n\omega_g a}{C_L}\right)K_1\left(\frac{\xi_n\omega_g a}{C_T}\right)+\frac{\xi_n\omega_g a}{C_T}K_1\left(\frac{\xi_n\omega_g a}{C_L}\right)K_0\left(\frac{\xi_n\omega_g a}{C_T}\right)\right. \qquad (4)$$
$$\left.+\frac{\xi_n\omega_g a}{C_L}K_1\left(\frac{\xi_n\omega_g a}{C_T}\right)K_0\left(\frac{\xi_n\omega_g a}{C_L}\right)\right]$$

4. refering to the notes in Fig. 6 and introducing usual matrices [m] (mass matrix), [C] (damping coeff. matrix) & [K] (restoring spring matrix) necessary to the upper structural vibration, the shear force V at the base and rotational spring k_R by the bed rock, the following simultaneous equations with variable ω given sequentially are established and solved of x_i and φ as complex numbers due to ω resulting in Transfer Functions $x_i(\omega)$ & $\varphi(\omega)$ or $\{H(\omega)\}$ in bound form, and

Simultaneous equations for the whole struc. response

$$\begin{cases}[m]\{\ddot{X}\}+[C]\{\dot{X}\}+[K]\{X\}=-[m]\{\ddot{X}_0+\ddot{\varphi}h\}, \\ I\ddot{\varphi}+k_R\varphi=-m_R H_s\ddot{X}_0+M+VH\end{cases} \qquad k_R=\frac{\pi}{2(1-\nu_b)}\mu_b a^3 \qquad (5)$$

Variables expressed in the complex form

$$X_i=\delta_i e^{i\omega t},\ X_0=u_g e^{i\omega t},\ \varphi=\varphi_0 e^{i\omega t},\ V=V_0 e^{i\omega t}$$

Responded vector of $\{X_i(\omega)\}, \{\dot{X}_i(\omega)\}, \{\ddot{X}_i(\omega)\}$

$$\begin{cases}\{\ddot{X}(\omega)\}=\{H(\omega)\}\ddot{X}_0(\omega) \\ \{\dot{X}(\omega)\}=\{H(\omega)\}\dot{X}_0(\omega) \\ \{X(\omega)\}=\{H(\omega)\}X_0(\omega)\end{cases} \qquad (6)$$

Fourier transform of $X_0(t)$

$$\begin{cases}\ddot{X}_0(\omega)=\int_0^T\ddot{X}_0(t)e^{-i\omega t}dt \\ \dot{X}_0(\omega)=\int_0^T\dot{X}_0(t)e^{-i\omega t}dt \\ X_0(\omega)=\int_0^T X(t)e^{-i\omega t}dt\end{cases} \qquad (7)$$

5. finally the above transfer functions of $x_i(\omega)$ and $\varphi(\omega)$ are processed by Fourier Inverse Transformation being multiplied by enforcing earthquakes in the form of Fourier Transformation.

$$\begin{cases}\{\ddot{X}(t)\}=\frac{1}{2\pi}\int_{-\infty}^{\infty}\{H(\omega)\}\ddot{X}_0(\omega)e^{i\omega t}d\omega \\ \{\dot{X}(t)\}=\frac{1}{2\pi}\int_{-\infty}^{\infty}\{H(\omega)\}\dot{X}_0(\omega)e^{i\omega t}d\omega \\ \{X(t)\}=\frac{1}{2\pi}\int_{-\infty}^{\infty}\{H(\omega)\}X_0(\omega)e^{i\omega t}d\omega\end{cases} \qquad (8)$$

On the other hand this time, the enforcing earthquakes to the bottom of the rigid body have been calculated through another conventional multi-mass vibratory system of shearing type for soil strata using a thin parallelepiped soil with unit sectional area shown in *Fig. 7.*

Mass-point (Depth)	Soil	N-val.	Mass (t. s/cm)	k (stiff: t/cm)
0m – 3.40m	surface	6	0.0027755	1.9169
3.40m – 7.45m	sandy silt	18	0.0060816	4.8274
7.45m – 15.00m	fine sand	44	0.0094693	6.3503
15.00m – 20.70m	fine sand	36	0.0108162	6.8926
20.70m – 23.65m	clay&silt	11	0.0070611	4.3241
23.65m – 30.55m	gravel	463	0.0087632	107.4579
30.55m – 38.65m	fine sand	301	0.0129673	40.3009
38.65m – 46.75m	fine sand	124	0.0132240	16.6739
46.75m – 52.40m	fine sand	143	0.0112244	27.5512
52.40m – 60.10m	fine sand	136	0.0108979	17.5011

Figure 7. Multi-masses system for soil strata

3 ADOPTED DATA AND PARAMETRIC COMBINATION FOR CALCULATION.

On the process of comparison with observation, the author has tried to take the method of parametric study because of several existing unknown factors. Firstly, structural data needed have been decided as written in Table 1

Table 2. Combinated list of parameters

	C_T M/S	C_L M/S	C_{TB} M/S	C_{LB} M/S	ρ T/MS²/CM	ρ_b T/MS²/CM	h_s Surface layer damping constant	ν_b Bottom layer Poisson's ratio	Damp. coeff. of upper str. HA
(1)	1000	2000	3500	7000	0.168E-3	0.180E-3	0.05	0.35	0.008
(2)	150	300	1150	2400	0.168E-3	0.180E-3	0.07	0.38	0.016
(3)	300	600	1150	2400	0.168E-3	0.180E-3	0.09	0.37	0.014
(4)	450	900	1150	2400	0.168E-3	0.180E-3	0.12	0.36	0.012
(5)	375	750	1150	2400	0.168E-3	0.180E-3	0.10	0.37	0.013
(6)	400	800	1150	2400	0.168E-3	0.180E-3	0.10	0.36	0.012

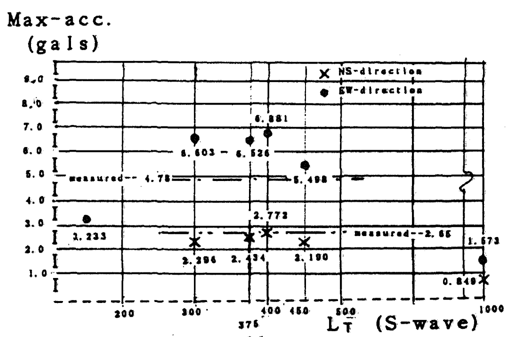
Direct	H M	RAD M	HS M	FMAS TS²/CM	FIM TMS²/CM
NS	15	18.53	5.574	20.57	0.277E+4
EW	15	18.53	5.574	20.57	0.486E+4

H ・・・ Height of rig. body
Hs ・・ Height of grav. centre
FIM ・ Mass of rig. body
RAD ・・ Radius of rig. body
FMAS ・ Rot. mass of rig. body

and 2. Secondly, mentioning the observation, the system has not yet operated completely except only one case of very small quake-magnitude until now, and so in this paper the earthquake data applied are about 1 gal as max-acceleration-values, which suggests the state of all mechanical properties of the constituent marerials must remain in the incipient elastic region. Thirdly, the important among the unknown factors is the additional rate of stiffnesses by inner and outer finishes to the original naked upper frame's at the time of weak earthquake, of which in this case the value has been taken as 125% through taking into considerations the ratio of the 1st periods between analysis and observation by micro-seismograph. Fourthly, another factors much concerned are damping ratios for both upper structure and soils, poisson's ratios of soils, velocities of S- and P-waves for surface and bottom soils. The follwing is the table on combinated list of the above factors thought for the purpose of search for adequate parameters to let analytical waves meet near the observed (Table. 2).

4 CONSIDERATIONS ON COMPARISON OF RESULTS.

Some considerations are described comparing mutually results obtained through analyses and real observation to verify the suitableness of modelling. As mentioned in 2 ADOPTED DATA ····, several cases of combinated parameters have been applied for computing analyses to both directions of NS and EW. The comparison of responded waves within soil-strata, which were calculated for a model of 10 mass vibratory system in shearing type under the forced conditions by earthquakes acquired at GL-59m, with the measured ones at GL-24m, -12m has been done. Waves by both theory and

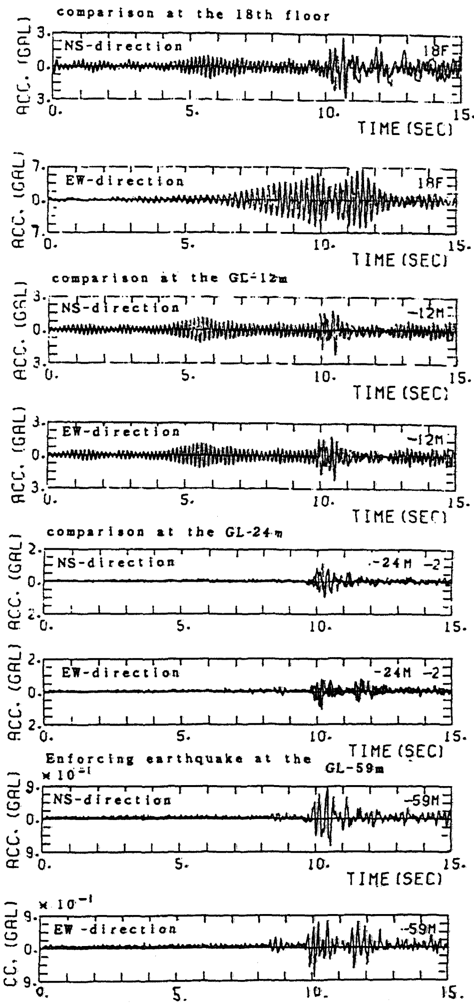


For the 18th fl. (m/sec)

	(a) Max-acc. distrib. for NS		(b) Max-acc. distrib. for EW	
	ACC(GAL)	(SEC)	ACC(GAL)	(SEC)
20	4.736		15.194	
19	3.623	10.79	10.230	11.61
18	2.772	10.79	6.526	11.60
17	1.799	10.78	4.920	11.59
16	0.934	10.96	3.215	11.57
15	1.987	10.71	2.830	11.34
14	2.962	10.70	3.647	11.53
13	3.333	10.69	4.755	11.51
12	3.283	10.68	5.748	11.51
11	3.270	10.66	6.450	11.50
10	3.412	10.65	6.885	11.50
9	3.633	10.63	7.053	11.49
8	3.806	10.62	7.050	11.49
7	3.879	10.61	6.916	11.48
6	4.061	10.59	6.684	11.48
5	4.241	10.58	7.303	11.47
4	3.810	10.58	6.248	11.47
3	3.329	10.58	5.114	11.46
2	2.815	10.58	3.992	11.46
	0.000	7.52	0.000	8.30

Figure 8. Curves on responded max-accelerations vs S-wave values

measuring show near values of the maximum accelerations mutually with some differences of their shapes for both directions (Fig. 9). Then the above theoretical quake-waves for NS and EW at the GL-15 meters are applied for the above analytical model and the values after numerical simulation are compared with those of observation. As for the comparison at the place of the 18th floor, the parametric combination of Case 6 for NS and Case 5 for EW have been decided most suitable after scrutiny of the max-accelerations responded (Fig. 8). The measured max-accel. values of 2.61 gals (NS) and 4.69 gals (EW) in Case 6 and 5 are nearly approximated relatively with some difference in wave-shapes (Fig. 9). This time concerning the velocities of S- and P-waves for the surface layer and its under strata, the values



Explanatory notes: —— bold line---calculated
—— thin line---measured

Figure 9. Comparison of responded waves for the 18th floor, GL-12m & -24m

could not be decided so easily only from the values of penetrating test performed, though from general thinking they should be necessarily decided of their values as precisely as possible by some other proper soil-tests such as pressio-meter test etc..

5 CONCLUSION.

As the conclusive remarks, the author observes the following through this parametric study on the structural system. This time the enforcing earthquakes given to the base of rigid body is yet very small in size of acceleration level and further short of duration time, and therefore more or less some difference is seen from the total view on max-accelerations and waveshapes, especially of the 18th roof-floor in relation even to the Case 6 & 5, which is the nearest between theory and observation.

This is thought to be concerned to the following reasons;

1. though the velocities of about 400 & 800 m/s for S- & P-waves seem to be adequate for this case, they are necessitated to be decided by proper testing method,
2. the state of vibration this time is thought in the completely initial elastic region because of the response for a weak enforcing quake given,
3. the mechanical properties of materials contain as a whole several uncertain and difficult factors unable to decide so easily as related to the state on nonlinear process towards collapse of the structural system, and
4. folowing the above items 2. and 3., the much more other cases of study are necessitated for the most appropriate parameter identification.

Finally the author concludes it will be recognized through much more future investigation that the modelling after Tajimi's theory will provide a very good analytical method to explain the behavior of this sort of structural quake-respose system.

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