THE VIBRATIONAL ANALYSIS OF THE TOWER BUILDING

(THE VIBRATION TEST AND EARTHQUAKE RESPONSE OF SAN-AI BUILDING)

by
Issao Funahashi* and Katsuhiro Kinoshita**

INTRODUCTION

Today we cannot avoid analyzing the dynamic behavior of a structure for design of tall building. In general, in the case of investigation the vibrational characteristics of structure, it is used to treat a structure as "shear mass system". However, to bending element having the greater part of the displacement of structure, it will be proper to analyze a structure as on the basis of bending system. It is desirable to apply the analytical method of bending type as continuous beam structure. In this report bending mass system is adopted to substitute for continuous beam structure. And the vibrational characteristics of a structure is analysed. Original system is displaced by bending mass system and replaced by equivalent few bending mass system for easying analysis and the capacity of analogue computor.

On the above consideration, the vibrational behavior of a structure to earthquake was responded in case of actual towery building in this report. The vibrational characteristics of an actual structure was measured by the forced vibration test and verified by the results of numerical analysis, the earthquake response as bending mass system was investigated. Therefore, the data of the earthquake response of the structure at the designing time was not obtained, but after completion of the building.

DESCRIPTION OF THE BUILDING

The actual building to investigate the earthquake behavior is San-ai building in Tokyo. This building can be regarded as the towery building by an external appearance and the frame of structure and situated momumentally and shinning brightly show-window itself in center of Tokyo.

The building is nine story and has three story underground. The structure of a building is composite type (steel reinforced concrete) having 43 meter of height as shown in Fig 8. The building consists of main and sub-towers, the former of cylindric type has cylindrical core included stairs, elevators and duct space to fulfil the structural and equipmental requirements around which the P.C. slab of doughnut type in each story is coming out. The sub-tower (stair case) is connected to main tower with bridge but is separated structurally by expansion joints.

---

* Chief Research Engineer   Takenaka Building Research Institute

** Research Engineer   Takenaka Building Research Institute

Now the frame is designed by taking 125% of the Japanese code valve for the seismic coefficient and base shear coefficient results in 0.28. General data of building, geological section, plan, section, P.C. slab and the story distribution of designed seismic coefficient are as shown Table 1 . and Fig 1 ~8.

FORCED VIBRATION TEST AND VIBRATIONAL CHARACTERISTICS OF THE STRUCTURE

The forced vibration was imparted to the building by means of the vibrator located on R-th roof floor. The vibrator is essentially consisted of three wheels, each having an eccentric mass. The eccentric moment of the vibrator used in the range of 1176 kg.cm. Usually the vibrator is used to set the building into forced vibration with small amplitudes. The natural periods, amplitudes of each story, damping and rocking motion of the building were determined from resonance curve which were measured by electro-magnetic seismograph. The relation curve of natural period and its amplitude and the deflection curve of the building in each resonanced periods are as shown in Fig 9, 10 .

Measured natural periods were recognized first period $T_1 = 0.57$ sec, second period $T_2 = 0.15$ sec and torsional period $T_t = 0.27$ sec caused by eccentric forces of vibrator's location. First period $T_1 = 0.57$ sec was estimated to be of a higher ratio of 20 - 30 % than general building about nine story which was also built by composite type structure, but the value was not so longer for its slender appearance.

Considering the first vibrational mode, which is similar to the bending type of cantilever from the rigid ground. Percent of critical damping h was about 0.07. The vertical amplitudes of rocking on base were not measured accurately but was less.

ANALYSIS OF BENDING MASS SYSTEM

The original structure was replaced by twelve bending mass system and analytically investigated the natural period and vibrational mode. In order to analyze the bending mass system, the stiffness of story K , substituted for bending stiffness EI, which is calculated by moment of inertia of cylindrical core. We could obtain the vibrational characteristics from the deflection of lateral froce caused by acting 1 g of each story mass m_i. We assume that structure at ground floor level is rigid, because of depth of the underground stories and higher stiffness of below ground level than above.

On the result of natural period's calculation, we could get two cases $T_1 = 0.562$ sec and $T_1 = 0.586$ sec, the former case was only bending type having no regard to opening of cylinder and the latter case was both bending and shearing type in consideration with the opening of cylinder and stiffness of seismic wall at lower fourth story. In comparison of measured first natural period $T_1 = 0.57$ sec and calculated period, elements of displacement of the structure were found occupying almost bending displacement.

Consequently the analysis was proceeded on the definition of bending system only bending displacement of cylindric core ignoring shearing displacement. Bending and shearing vibrational mode in consideration with the opening of cylinder and only bending vibrational mode are as a shown in Table 2 .

To treat a structure as the bending mass system, it is assumed that:

1) The structure is considered to be a cantileber, rigid in ground and it is only bending displacement.

2) The cylindric core is sectioned by the masses of the floor slabs are concentrated at equidistant floor levels.

3) The bending stiffness EIi is constant between each mass m_i.

4) To analyze this bending mass system which is rigid in ground, we assume the continuous beam which is supported at each mass and unit load is forced at the respective supports.

5) Influence number of bending displacement d_{ij} of each support is caused by unit load and influence number of supported reaction R_{ij} is caused by unit displacement are calculated.

In the case of San-ai building, the original structure was replaced to twelve bending mass system which had mass concentrated at each floor lebel and bending stiffness of story as shown in Fig 15 . We calculated the vibrational properties by considering the influence of steel section, concrete Young's modulus $E_c = 2.1 \times 10^5$ kg/cm^2 and the ratio of modulus of elasticity to steel $E_s/E_c = 10$.

Bending displacements of each mass are calculated by M_i/EI_i . Relative bending displacement of n-th story $\delta_{B \cdot n}$ are obtained by rotative displacement $\delta_{\theta n}$ of rotative angle θ_n at the ground story and bending displacement $\delta_{M \cdot n}$ only of n-th story.

$$\delta_{B \cdot n} = \delta_{\theta \cdot n} + \delta_{M \cdot n} \qquad (1)$$

now
$$\delta_{\theta \cdot n} = \theta_n \cdot h_n \qquad (2)$$

Then rotative angle is obtained by Mohr's Theorem.

$$\theta_n = \frac{1}{EI_i} \int_1^{n-1} M_i \cdot dh = \sum_{i=1}^{n-1} \frac{M_i}{EI_i} h_i \qquad (3)$$

Equation (3) means the sum of the area of M_i/EI_i upto n-1 story.

$$\delta_{mn} = \frac{1}{EI_n} \int_0^{h_n} M_n \cdot dh \cdot x_n = \frac{M_n}{EI_n} \cdot h_n \cdot x_n \qquad (4)$$

Now, x_n is distance from center of gravity of $\frac{M_n}{EI_n}$ area as shown in Fig 11 .

From equations (2), (3) and (4)

$$\delta_{Bn} = \left(\sum_{i=1}^{n-1} \frac{M_i \cdot h_i}{EI_i}\right) h_n + \frac{M_n \cdot h_n}{EI_n} \cdot x_n \tag{5}$$

We assume that a trapezoid bending moment distribution of each story regards a rectangle having same area and then $x_n = \frac{h_n}{2}$. This assumption will not give the error for practical analysis. Thus we have new equation (6).

$$\delta_{Bn} = \left(\sum_{i=1}^{n-1} \frac{M_i h_i}{EI_i}\right) h_n + \frac{1}{2} \cdot \frac{M_n h_n^2}{EI_n} \tag{6}$$

We get equation (7) by putting story stiffness $k = I/h$ and standard stiffness K_o.

$$\delta_{Bn} = \left(\sum_{i=1}^{n-1} \frac{M_i}{k_i} + \frac{1}{2} \cdot \frac{M_n}{k_n}\right) \frac{h_n}{EK_o} \tag{7}$$

We could calculate methodically used table as shown in Table 6. in the case of San-ai building.

Then influence number of bending displacement d_{ij} of each support is calculated. Namely d_{ij} is the bending displacement of the j-th mass when the i-th is forced by unit load. In the case of replacement to equivalent a few bending mass system from original system, equation (7) is calculated at given mass.

For given example, we showed the case of which the twelve bending mass of original system was replaced to three bending mass system. It was three bending mass system in which masses gathered at twelveth, eighth and forth stories. Results of influence number of bending displacement d_{ij} are shown in Table 4, 6. by three equivalent bending mass system as well as original bending twelve mass system. By means of d_{ij}, we can obtain influence number of reaction at the respective supports R_{ij} when unit displacement of the i-th story causes.

According to the relation of R_{ij} and relative story displacement δ_{ij}, the unit load of j-th story and the others stories are given by the following equation (8).

$$\sum R_{ij} \cdot \delta_{ij} = 1. \qquad \sum R_{ij} \cdot \delta_{ij} = 0 \tag{8}$$

For instance, in the case of upper-most story for unit displacement, in three bending mass system, the following simultaneous equations are expressed.

$$\left.\begin{aligned} R_{33} \cdot \delta_{33} + R_{33} \cdot \delta_{23} + R_{31} \cdot \delta_{13} &= 1 \\ R_{33} \cdot \delta_{32} + R_{32} \cdot \delta_{22} + R_{31} \cdot \delta_{12} &= 0 \\ R_{33} \cdot \delta_{31} + R_{32} \cdot \delta_{21} + R_{31} \cdot \delta_{11} &= 0 \end{aligned}\right\} \tag{9}$$

Calculated results of influence numbers R_{ij} of three supported reactions are as shown in Table 5.

By these deflection and influence number, the vibrational equations having n-th mass are given by the following simultaneous second order differential equations.

$$m_i \ddot{y}_i + \sum_{j=1}^{n} C_{ij} \cdot \dot{\delta}_i + \sum_{j=1}^{n} R_{ij} \cdot \delta_i = - m_i \ddot{y}_0 \qquad (10)$$

Now C_{ij} is viscous damping constant for i-th mass by unit displacement of j-th story. The definition of C_{ij} is

$$C_{ij} = \frac{2h_1}{\omega_1} \cdot R_{ij} \qquad (11)$$

Where ω_1 is the circular frequency of the first mode, h_1 is percentage of critical damping. Nomenclature $\ddot{y}_0$ is acceleration of the ground with respect to a fixed datum. By equation (10), we can analyze the earthquake response of vibration system having m_i, C_{ij} and R_{ij} to earthquake acceleration $\ddot{y}_0 (t)$. Now the vibrational characteristics will be considered by C_{ij} and $\ddot{y}_0$ equal to zero.

$$m_i \ddot{y}_i + \sum_{j=1}^{n} R_{ij} \cdot \delta_i = 0 \qquad (12)$$

By the theory of determinants following relation is obtained.

$$\begin{vmatrix} (R_{11} - R_{21}) - m_1 \omega^2 & R_{21} - R_{31} & R_{31} \\ R_{12} - R_{22} & (R_{22} - R_{32}) - m_2 \omega^2 & R_{32} \\ R_{13} - R_{23} & R_{23} - R_{33} & R_{33} - m_3 \omega^2 \end{vmatrix} = 0 \qquad (13)$$

By equation (13), the results of natural period, vibrational mode and excitation function βu are shown in Fig 12.

EARTHQUAKE RESPONSE AND ITS CONSIDERATION

The earthquake behavior of structure having these vibrational characteristics was computed by analogue computor - SERAC -. Block diagram of three bending mass system by equation is as shown in Fig 13. The driving earthquake force considered in this analysis was based on the N-S component of the motions in El Centro earthquake of May 18, 1940 and the S69°E component in Taft 1952 and their intensities were 0.33 g. The maximum value of story displacement in each story is obtained directly from these records are shown in Fig 14. The first vibrational mode by response results was nearly similar to that of forced vibration test. Displacement response of El Centro was recorded about one and a half times of Taft earthquake. Then, by one mass response spectrum of El Centro, modal analysis was calculated for story displacement. As the first natural period was comparatively short, accurate values of response spectrum were hard to obtain. Result of modal analysis was about 10% more than that of response, because of linear response.

Now let us investigate the stress of cylindrical core at each story in El Centro earthquake. Displacement of original twelve bending mass system could be assumed to be the response results of three bending mass system. For convenience on account of similarity first vibrational mode, top displacement of three bending mass system was distributed along the first vibrational mode.

Then yield bending moment of each story core section was calculated by acting axial force and bending moment. Due to the fiber stress of cylindric core section, each yield bending moments were obtained by concrete initial crack, steel tensile yield, concrete compressive yield and concrete maximum failure of plastic coefficient f. Table 8. shows values of each bending moment in El Centro response and bending moment of designed seismic coefficient as we described before. The tensile stress of concrete was taken 20 kg/cm^2, 10% of designed compressive stress 210 kg/cm^2 and yield stress of steel 2400 kg/cm^2. When El Centro earthquake drived, the structure would suffer the concrete crack over the value of concrete yield bending moment. At fourth and fifth stories, we could imagine the tensile stress in steel partly got to yield, the value of fiber stress. The reason of steel yield at partial story was due to the ratio of gross steel area to concrete was 2.7% at fourth and fifth stories 8.7% at first and second stories and 5.5% at the third story. But then also, the partial steel yield could not be fatal blow of main structure. Now plastic coefficient f is difined as the ratio of plastic sectional Z_p to elastic sectional modulus Z_e.

$$f = Z_P / Z_e \qquad (14)$$

Plastic sectional modulus Z_p is expressed by the following equation in the case of cylindrical core having thickness t and diameter D of core

$$Z_P = \frac{1}{6} D^3 \left[1 - \left(1 - \frac{2t^3}{D} \right) \right] \qquad (15)$$

Therefore, if final compressive destruction is dicided with concrete, the final bending moment must be the compressive bending moment of concrete multiplied by the value of f. Though, in the plastic behavior of bending mass system, when the bending moment of cylindric section of foundation got to yield value, we had to investigate such non-linear system as the hinge made by the moment of inertia of the basement might be happened and then the compressive failure of concrete is obtained.

As above mentioned, the analysis was considered as the bending mass system. In the case of the bending mass system, maximum bending moment is corresponded with base shear coefficient. But, to the consideration of the earthquake behavior of structure, nothing would be better than base shear coefficient. Now let us see by substituting shear mass system for bending mass system. The story shear force which was corresponded with response displacement. From above shear stiffness of spring replaced to bending stiffness EI was obtained. Of course, the vibrational mode and characteristics of shear mass system coinsided with that of bending mass system was always necessary. Because the analysis of bending mass system took the place of shear mass system having the same vibrational properties. Without damping, bending and shear mass system are expressed in the following

$$m_i \ddot{y}_i + \sum_{j=1}^{n} R_{ij} \cdot \delta_i = - m_i \ddot{y}_0 \qquad (16)$$

$$m_i \ddot{y}_i + \sum_{j=1}^{n} k_i \cdot \delta_i = - m_i \ddot{y}_0 \qquad (17)$$

By equations (16) and (17), the equality of each mass was solved. In case of San-ai building, shear stiffness were obtained as follows: k = 85.32 t-cm, k = 136.78 t-cm, k = 342.38 t/cm. From the results of displacement response, maximum story shear coefficient were gained as follows: q = 2.25, q = 0.561, q = 0.355. In the same case, maximum story shear coefficients caused concrete crack at each story were q = 2.16, q = 0.594 and q = 0.32. Consequently, response shear coefficient q = 0.355 of El Centro earthquake exceeded the designed seismic coefficient q = 0.28 and concrete crack's coefficient at each story q = 0.32. As compared with response base shear coefficient of El Centro q = 0.355 and Taft 0.252, the former was in the plane of expecting tensile stress of steel after concrete crack and the latter might be partly concrete crack, but lower than designed seismic coefficient.

CONCLUSIION

An analytical method of bending mass system was explained in detail giving an example of actual structure San-ai building which had much bending element in displacement of structure. The vibrational characteristics were investigated by actual forced vibration test. And the earthquake behaviors of structure were computed by analogue computor. Then we found the stress of structure by the results of response. We are going to study non-linear response of bending mass system which has partial yield.

ACKNOWLEDGEMENT

The authors wish to thank Prof. H. Umemura, Tokyo University and Mr. A. Shibata, graduated Tokyo University for grateful assistance using analogue computor SERAC. Also, we appreciate Mr. Yano and Kobayashi, Nikken Sekkei Komu with gratitude for co-operation.

BIBLIOGRAPHY

1. I. Funahashi, and K. Konoshita "The Vibrational Analysis of Tower Building" Transaction of the Architectural Institute of Japan, No.89 1963; SERAC Report No.5.

2. R.L. Jennings and N.M. Newmark "Elastic Response of Multi-story Shear Beam Type Structures subjected to Strong Ground Motion" II. WCEE 1960 Tokyo.

NOMENCLATURE

T	:	Natural period
ω	:	Circular frequency
m_i	:	i-th mass
k	:	Story stiffness I/h
I	:	Moment of inertia
E	:	Modulus of elasticity of steel or concrete
δ_{Bi}	:	Relative bending displacement of i-th story
$\dot{\delta}$	:	Relative velocity
h_i	:	Story height of i-th story
M_i	:	Bending moment of i-th story
X_n	:	Distance from center of gravity
d_{ij}	:	Influence number of displacement for i-th story by unit load of j-th story
R_{ij}	:	Influence number of reaction for i-th story by unit displacement of j-th story
y	:	Displacement of the ground relative to a fixed datum
$\ddot{y}_0$	:	Earthquake acceleration
Z	:	Sectional modulus
C_{ij}	:	Viscous damping constant for i-th story by unit displacement of j-th story
h	:	Percentage of critical damping
β	:	Excitation function
δ	:	Base shear coefficient
f	:	Plastic coefficient, the ratio of Zp to Ze

Area :	The Building Area 221.5 m^2
	Total Floor Space 2491.1 m^2
Structure :	
Underground :	Reinforced Concrete (General)
	Composite Type Structure (Core)
Above Ground :	Composite Type Structure (General)
	Prestressed Concrete Slab (9~5 Floors)
Advertisement Tower :	Steel Structure
Construction Period :	May 1961 ~ Dec. 1962
Construction :	Takenaka Koumuten Co., LTD.
Design :	Nikken Sekkei Koumu Co., LTD.

Table 1. General Data of Building

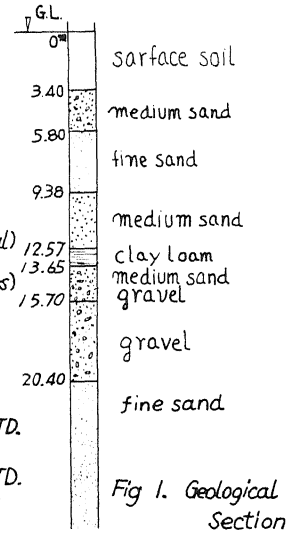


Fig 1. Geological Section

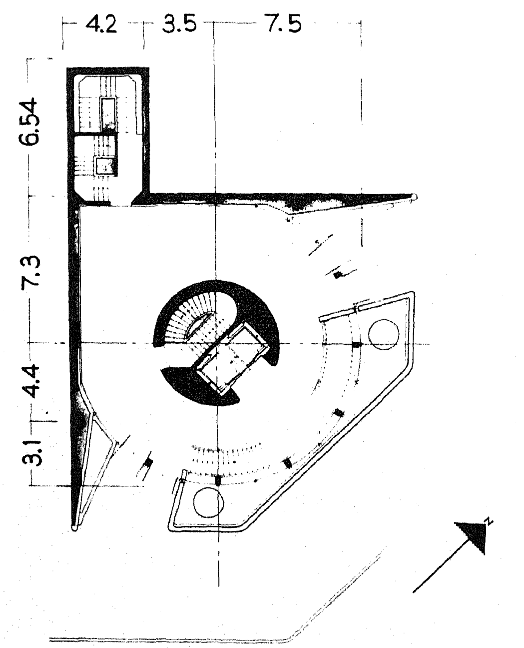


Fig 2. Plan (First Foor)

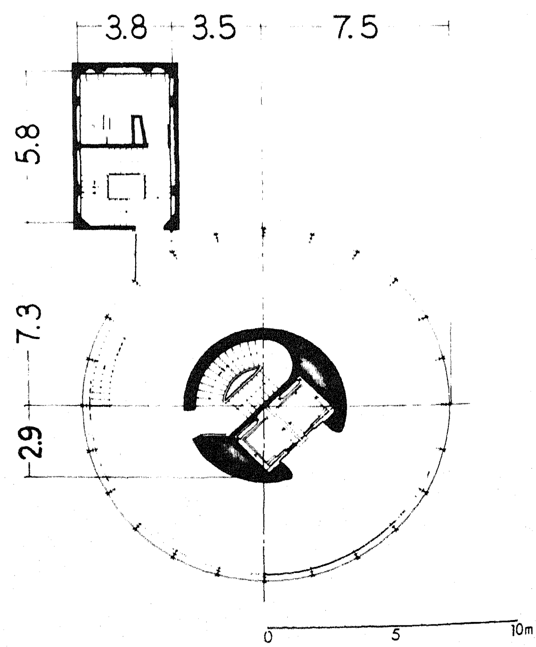


Fig 3. Plan (6, 8 Floor)

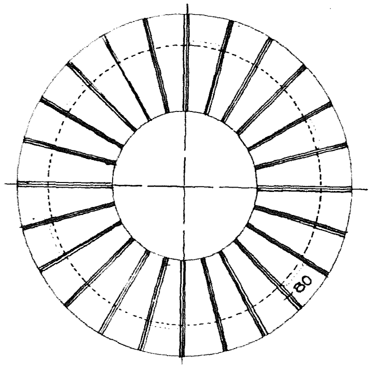


Fig 4. P.C. Slab Plan

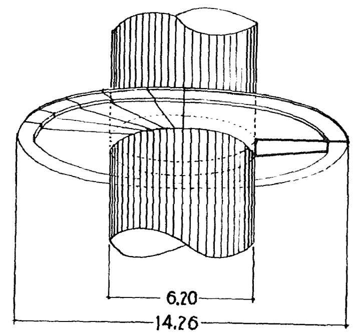


Fig 5. Core and P.C. Slab

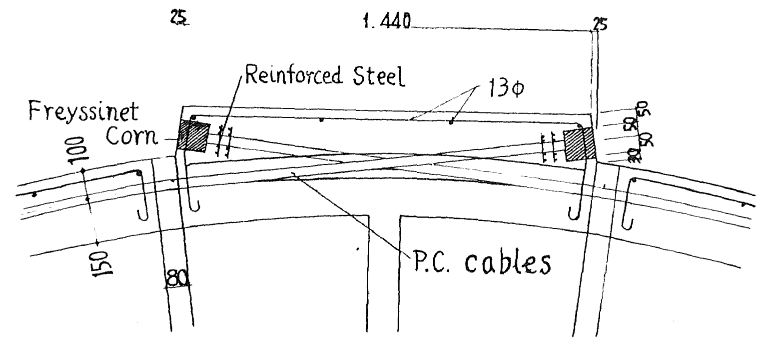


Fig 7. Detail of P.C. Cable Anchored

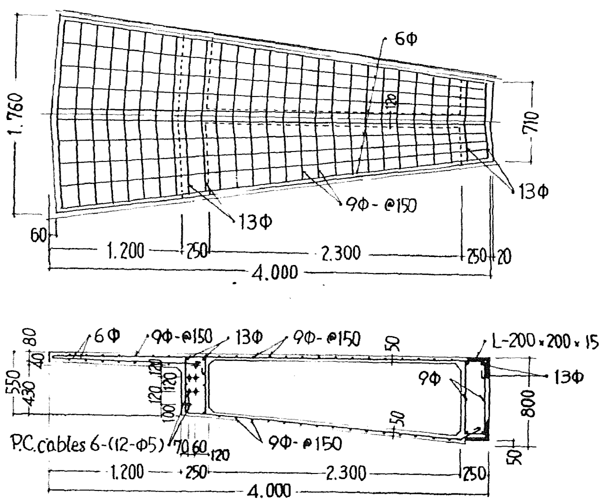
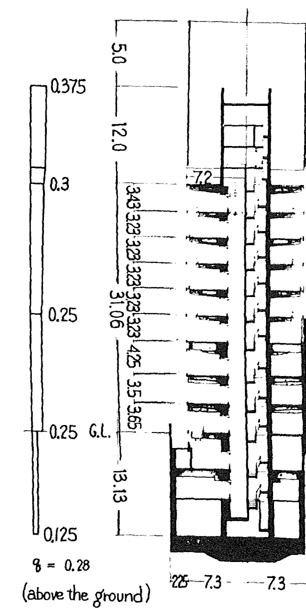


Fig 6. P.C. Slab Details (Plan, Section)



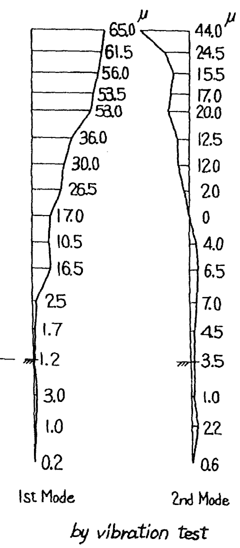


Fig 8. Distribution of Seismic Coefficient

Fig 9. Displacement of Vibrational Mode

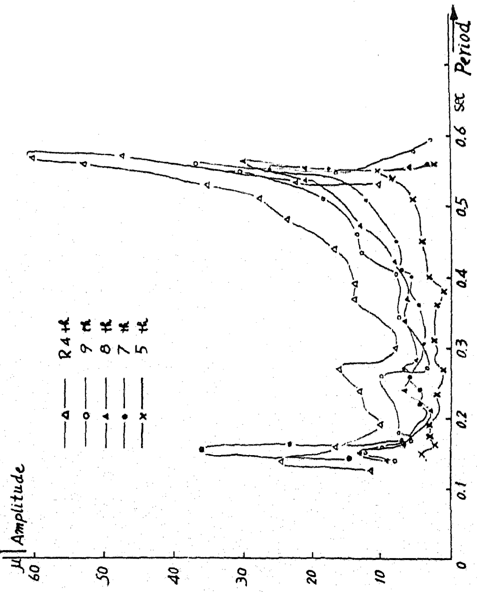


Fig 10. Resonance Curve

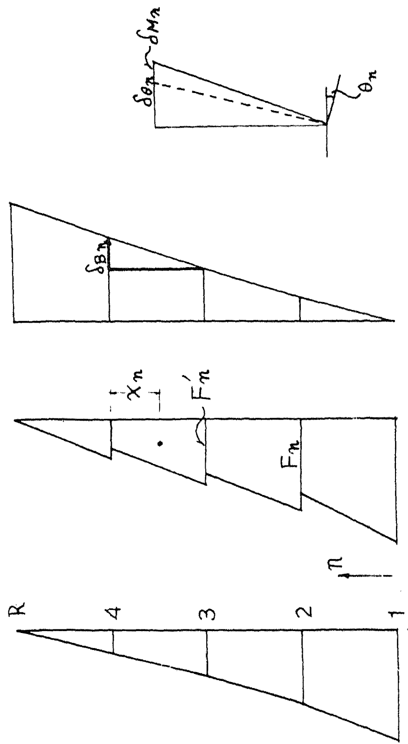
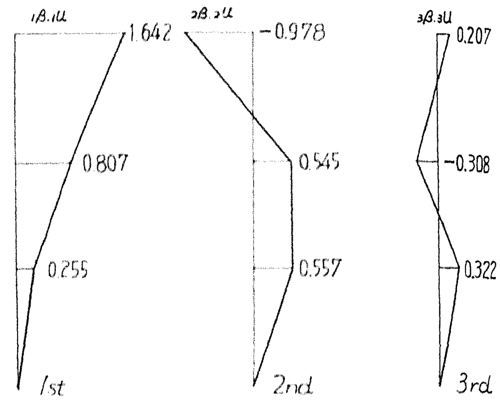


Fig 11. Illustration of Equation of Bending Mass System



$\omega_1 = 10.809$ $\omega_2 = 53.076$ $\omega_3 = 107.979$

$T_1 = 0.581$ $T_2 = 0.118$ $T_3 = 0.058$

$\beta_1 = 1.642$ $\beta_2 = -0.978$ $\beta_3 = 0.207$

Fig 12. Vibrational Characteristics

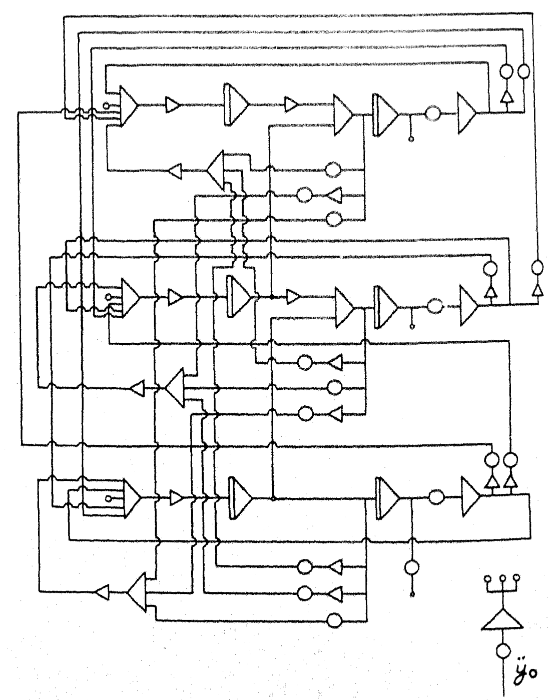


Fig 13. Analogue Block Diagram

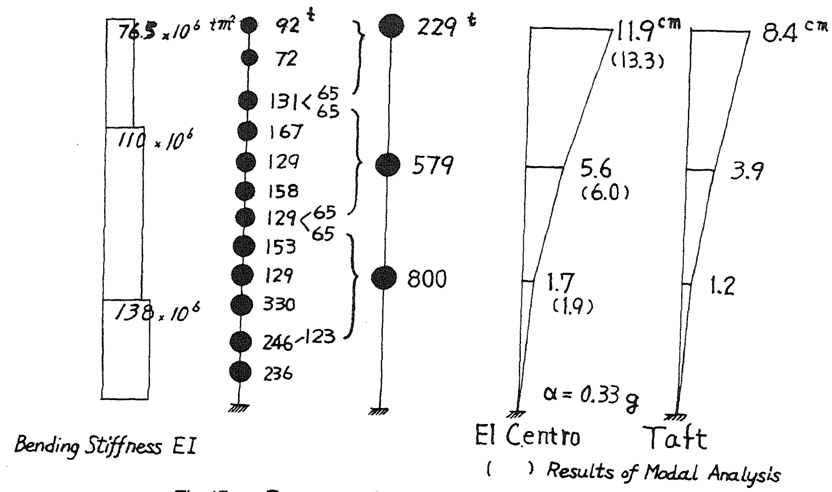


Fig 15. Response Displacement of Earthquake

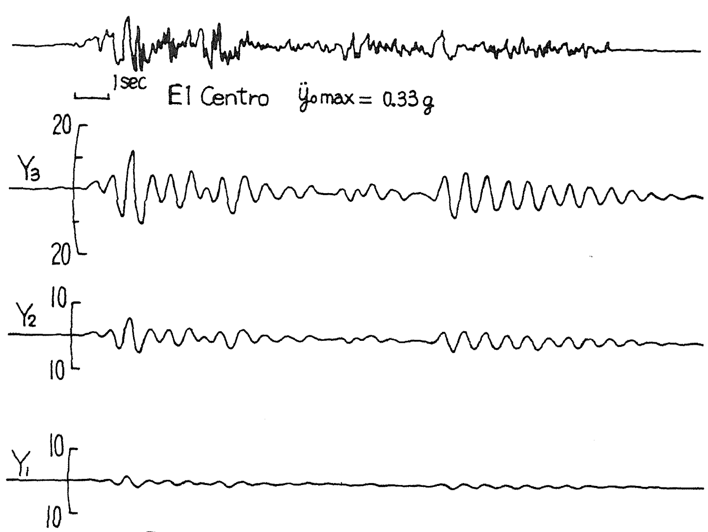


Fig 14. Recorded Response Displacement

Fig 16. Appearance of San-ai Building

	Case 1	Cace 2
12	1.000	1.000
11	0.895	0.896
10	0.725	0.718
9	0.007	0.609
8	0.500	0.510
7	0.404	0.417
6	0.311	0.328
5.	0.230	0.247
4	0.158	0.173
3	0.090	0.107
2	0.040	0.047
1	0.011	0.015

Table 2.

Vibrational mode
Case 1 : only bending displacement
Case 2 : both bending and shearing displacement

	thick t con	thick t steel	I (10^8) con	I (10^8) steel	Σ I	h	K (10^6)	k
12	30.0	0.32	22.8	1.36	36.4	323	11.28	11.28
11	30.0	0.32	22.8	1.36	36.4	520	7.0	7.0
10	30.0	0.32	22.8	1.36	36.4	365	9.97	9.97
9	40.0	1.05	30.95	2.12	52.15	343	15.15	15.15
8	40.0	1.05	30.95	2.12	52.15	323	16.14	16.14
7	40.0	1.05	30.95	2.12	52.15	323	16.14	16.14
6	40.0	1.05	30.95	2.12	52.15	323	16.14	16.14
5	40.0	1.05	30.95	2.12	52.15	323	16.14	16.14
4	40.0	1.05	30.95	2.12	52.15	323	16.14	16.14
3	50.0	4.35	38.5	2.70	65.5	425	15.40	15.40
2	50.0	4.35	38.5	2.70	65.5	350	18.70	18.70
1	50.0	4.35	38.5	2.70	65.5	373	17.55	17.55

Table 3. Caluculation of I, k ($K_0=10^6$)

	1	2	3
1	7.79	18.34	31.01
2	18.34	54.84	102.83
3	31.01	102.83	221.39

Table 4. Influence numbers of α_{ij}

	1	2	3
1	3733.38	-1005.57	-9.75
2	-2505.78	2045.58	-555.12
3	545.37	-979.00	423.87

Table 5. Infulence numbers of R_{ij}

	Q_i	$Q_i h_i$	M_i	k_i	M_i/k_i	$2M_i/k_i$	$4\Delta_B$	$h_i/4EK_0$	δ_B	y_B
12	1	3.23	0 3.23	11.28	0 0.286	0.286	66.834	0.384	26.168	221.390
11	1	5.20	3.23 8.43	7.0	0.461 1.204	1.665	64.883	0.635	41.169	195.222
10	1	3.65	8.43 12.08	9.97	0.846 1.212	2.058	61.160	0.445	27.213	154.053
9	1	3.43	12.08 15.51	15.15	0.787 1.024	1.821	57.281	0.419	24.006	126.840
8	1	3.23	15.51 18.74	16.14	0.961 1.161	2.122	53.338	0.394	21.024	102.834
7	1	3.23	18.74 21.97	16.14	1.161 1.361	2.522	48.694	0.394	19.185	81.810
6	1	3.23	21.97 25.20	16.14	1.361 1.561	2.922	43.250	0.394	17.062	62.625
5	1	3.23	25.20 28.43	16.14	1.561 1.761	3.322	37.006	0.394	14.553	45.563
4	1	3.23	28.43 31.66	16.14	1.761 1.962	3.723	29.961	0.394	11.916	31.010
3	1	4.25	31.66 35.91	15.40	2.056 2.332	4.388	21.850	0.518	11.321	19.184
2	1	3.50	35.91 39.41	18.70	1.920 2.107	4.027	13.435	0.427	5.722	7.863
1	1	3.73	39.41 43.14	17.55	2.246 2.458	4.704	4.704	0.455	2.141	2.141

Table 6. Methodical Calculation Table of δ_{ij} (Unit Load Forced at Uppermost Story)

i\j	1	2	3	4	5	6	7	8	9	10	11	12
1	0.096	0.277	0.496	0.663	0.830	0.998	1.166	1.334	1.513	1.703	1.973	2.143
2	0.277	0.877	1.703	2.332	2.961	3.590	4.220	4.850	5.520	6.222	7.232	7.863
3	0.496	1.703	3.643	5.230	6.815	8.400	9.985	11.570	13.255	15.045	17.599	19.184
4	0.663	2.332	5.230	7.790	10.431	13.069	15.701	18.344	21.106	24.125	28.373	31.010
5	0.830	2.961	6.815	10.431	14.351	18.354	22.355	26.358	30.548	35.125	41.565	45.563
6	0.998	3.590	8.400	13.069	18.354	23.960	29.643	35.329	41.282	47.789	56.946	62.625
7	1.166	4.220	9.985	15.705	22.355	29.643	37.242	44.925	53.087	61.762	74.134	81.810
8	1.334	4.850	11.570	18.344	26.358	35.329	44.925	54.842	65.460	76.750	92.846	102.834
9	1.513	5.520	13.255	21.106	30.548	41.282	53.087	65.460	78.703	93.427	114.027	126.840
10	1.703	6.222	15.045	24.125	35.125	47.789	61.762	76.750	93.427	111.621	137.794	154.053
11	1.973	7.232	17.599	28.373	41.565	56.946	74.134	92.846	114.027	137.794	173.043	195.222
12	2.141	7.863	19.184	31.010	45.563	62.625	81.810	102.834	126.840	154.053	195.222	221.390

Table 7. Influence Numbers of α_{ij} (12 Bending Mass System)

	Responce Moment		Designed Seismic Coeff.	Conc. Crack	Steel Tensile	Conc. Comp	Maximum
	El Centro $\times 10^2$ t.m	Taft $\times 10^2$ tm	Total Moment $\times 10^2$ tm	Yield Moment $\times 10^2$ tm	Yield Moment $\times 10^2$ tm	Moment $\times 10^2$ tm	Moment $\times 10^2$ tm
12	2.65	1.9	1.11	19.6	78.0	209	224
11	10.2	7.9	4.30	18.4	78.5	208	223
10	19.6	14.1	8.33	16.3	73.0	206	221
9	33.1	24.1	14.26	22.5	123.5	293	409
8	49.0	36.3	21.05	20.4	122.8	290	405
7	68.5	50.0	29.26	17.9	121.3	288	402
6	92.4	66.9	38.62	15.8	119.1	286	399
5	120.0	86.4	49.28	13.4	117.5	283	395
4	150.3	107.9	60.98	11.2	116.0	281	392
3	195.9	140.1	79.88	18.9	217.6	420	588
2	241.5	172.3	97.60	26.7	318.0	484	663
1	296.0	212.3	118.69	24.0	315.0	481	659

Table 8. Moment Comparison of Earthquake Response, Designed and Each Yield of Section

THE VIBRATIONAL ANALYSIS OF THE TOWER BUILDING (THE VIBRATION TEST AND EARTHQUAKE RESPONSE OF SAN-AI BUILDING)

BY I. FUNAHASHI AND K. KINOSHITA

QUESTION BY: PROF. K. KUBO - JAPAN

As many people suggest, damping is one of the important factors in dynamic analysis and you determined the damping constant by a dynamic test of the prototype building. I think you determined the damping constant from the resonance curve of vibration tests for the first mode as well as the second mode. Did you find, 1) any difference between the damping constant of the first mode and that of the second mode, and 2) any relation between damping constants and the vibrational amplitude?

REPLY BY: K. KINOSHITA

Yes, I think that the problem of a structure's damping is a very important thing in dynamic analysis. From my vibration test, the damping ratio was decided by the resonance curve, which is shown in Fig. 10, and critical damping of first mode was about 7 per cent.

The damping constant of the second mode did not need to be confirmed in quality, but I think, from the resonance curve, it is approximately equal to the first damping constant. Generally, it is said that the higher the frequency, the bigger the damping constant, but from my test it did not need to be perfectly confirmed.

By means of the resonance curve, a large amplitude at the upper storey appeared to have a high critical damping ratio. Namely, I think that the damping constants of large amplitudes are higher, while those of small amplitudes are lower.