EARTHQUAKE EFFECTS ON STEEL TOWER STRUCTURES ATOP BUILDING

by
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SYNOPSIS:

This research was carried out utilizing the opportunity of studying the effects of '68 Off-Tokachi Earthquake on the steel tower for micro-wave antennas erected on the roof of buildings by NTT (Nippon Telegraph and Telephone Public Corp.). This report gives the results of the full-scale measurements and earthquake response analysis carried out on an actually existing steel tower, as well as the results of vibration test performed by using Steel Tower and Building Models.

1. INTRODUCTION

In Japan, seismic calculation for the structural design of steel towers is made according to the Building Standards Law and AIJ Structural Standards (Architectural Institute of Japan). This is a static calculation method of seismic forces using the so called lateral seismic coefficient of 0.3 or more. In the combination of design loads of tower stipulated by the present Building Standards Law, the seismic force is generally smaller as compared to the wind force. However, when the tower has heavy objects at the upper part or in case of uneven distribution of rigidities, or when the tower erected on buildings are affected by the building structure during earthquake, etc., the seismic force is expected to exceed the wind force.

A large number of steel towers for microwave antennas have been erected all over the country; and most of these towers are atop-building towers as shown in Photo.1. In the areas affected by '68 Off-Tokachi Earthquake, detailed investigation of the steel towers showed slight effects such as deformation of structural members, as shown in Fig.1. It is confirmed through this investigation that severe vibration of steel towers on roof occures during an earthquake due to quasi-resonance phenomenon. Fig.2 shows data on structural design of an actual steel tower for microwave antennas. It is evident from this figure that, the major condition affecting the steel tower is not the ununiformity of the load distribution over the vertical direction, but the tower-height and the number of antenna platforms. The distribution of shearing force coefficient converted from the wind froce remains in the range of 0.7-1.2 in the vertical direction of tower.

In the present research by means of the full-scale measurements, model vibration test and seismic response analysis, it was made clear that the steel tower atop building may be affected by the vibrational characteristics of the building and consequently generate high seismic forces at the time of an earthquake.

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2. FULL SCALE MEASUREMENTS AND THEORETICAL ANALYSIS

2.1 FULL SCALE MEASUREMENTS

(1) Free-vibration tests of actual steel towers in 'man-excited' manner were carried out by measuring the resultant tower displacement. The fundamental period(T) of tower was presented by the experimental formula $T=0.015 \times H$ (sec. H: tower height m), as shown in Fig. 3. The measured logarithmic-decrement indicated the damping to be approximately 1% of the critical damping as shown in Fig.4.

(2) Forced-vibration test of actual coupled system of tower (69m) and building (31m) was carried out by running a vibrator. The resonant period and displacement curve obtained from this test are shown in Table 1 and Fig.5. The fundamental resonant period and damping coefficient of the steel tower are similar to the results obtained from free-vibration test. The higher period of steel tower is close to the fundamental or secondary vibration period of the building and the resonance curve of steel tower is sharp as compared to the building. The fundamental vibration curve obtained from microtremor measurements of the steel tower as given in Fig. 6 shows that the displacement tends to increase in the vertical direction of tower, and this tendency is remarkable at the position of each platform.

2.2 THEORETICAL ANALYSIS

The natural periods and vibration mode shapes (excitation function) as eigenvalue problem for two vibration systems mentioned below and as shown in Table 2 and 3 respectively, were calculated by substituting these with multi-mass dynamic system as shown in Fig. 7.

(1) In case of tower alone

The calculated fundamental periods of towers are found to conform well to the measured periods, and as shown in Fig. 8, the second mode frequency and the third mode frequency of towers are respectively in the range of 3.5-4.5 and 6.5-7.5 times the fundamental frequency. An example of vibration mode shape obtained by plotting the excitation function of tower is shown in Fig. 9, which shows that the fundamental mode shape indicates unusual shape of which the lowest mass position is of a different phase. Its cause is considered to be due to the tower-frame-work with large inclination of plane of the truss.

(2) In case of coupled system of Tower-Building

The tower and building are substituted with a double-mass dynamic system and computation for excitation function of tower was carried out taking the period ratio (Tower/Building) and the mass ratio (Tower/ Building) as parameters and the calculated result is shown in Fig. 10. The excitation function of tower indicates its maximum value when the individual natural periods of tower and the building become identical. When the period ratio assumes constant, the excitation function tends to increase with the lessening of mass ratio and this tendency is remarkable in the resonant range.

Natural frequency and excitation function when tower and building were substituted with multi-mass dynamic system, were calculated by following Equation (1) and (1)'.

$$[M]\{\ddot{X}\} + [S]\{x\} = 0 \qquad \text{-----------} \qquad (1)$$

$$[S]^{-1} = [F] = \begin{array}{c} \left[\begin{array}{cccc|cccc} \frac{1}{K_1} & \frac{1}{K_1} & --- & --- & --- & --- & --- & \frac{1}{K_1} \\ \frac{1}{K_1} & \frac{1}{K_1}+\frac{1}{K_2} & --- & --- & --- & --- & --- & \frac{1}{K_1}+\frac{1}{K_2} \\ --- & --- & --- & --- & --- & --- & --- & --- \\ --- & --- & --- & \sum_1^i \frac{1}{K_i} & --- & --- & --- & \sum_1^i \frac{1}{K_i} \\ \hline --- & --- & --- & --- & \sum_1^i \frac{1}{K_i}+f_{11} & \sum_1^i \frac{1}{K_i}+f_{12} & --- & \sum_1^i \frac{1}{K_i}+f_{1n} \\ --- & --- & --- & --- & \sum_1^i \frac{1}{K_i}+f_{21} & \sum_1^i \frac{1}{K_i}+f_{22} & --- & \sum_1^i \frac{1}{K_i}+f_{2n} \\ --- & --- & --- & --- & --- & --- & --- & --- \\ \frac{1}{K_1} & \frac{1}{K_1}+\frac{1}{K_2} & --- & \sum_1^i \frac{1}{K_i} & \sum_1^i \frac{1}{K_i}+f_{n1} & \sum_1^i \frac{1}{K_i}+f_{n2} & --- & \sum_1^i \frac{1}{K_i}+f_{nn} \end{array}\right] \end{array} \begin{array}{l} \text{building} \\ \\ \\ \\ \\ \text{tower} \end{array} \qquad \text{---} \; (1)'$$

where, $[M]$: Mass matrix $\qquad$ $[X]$: Relative displacement

$[S]$: Stiffness matrix $\qquad$ $[F]$: Flexibility matrix

K_{1-i} : Spring factor for 1-i th story of building

f_{nn} : Elements of flexibility matrix of tower

Fig.11 shows an example of calculated natural frequencies of a coupled system of Tower-Building. It is evident from this figure that natural frequency of fundamental and higher mode to be approximately coincident with those of either tower or building of coupled system. The excitation functions of coupled system of Tower-Building are plotted as shown in Fig.12. It becomes clear that when the individual fundamental frequencies of tower and building come closer to each other, the fundamental and secondary excitation functions of tower become remarkably larger as compared to other frequency ratios.

3. VIBRATIONAL TESTS OF MODELS USING SHAKING TABLE

3.1 VIBRATIONAL TEST OF 1/70 SCALED MODEL

Tower and building models used in this test are of the dimensions shown in Fig.13, where the tower can be detached from the building and replaced with towers of variable height and mass. Free-vibration test and forced vibration test using a shaking table wherein these were subjected to horizontal sinusoidal wave excitations, were carried out for the Tower (M), Building (B) and the combined Tower-Building model.

(1) Period and Mode of Vibration

Fig.14 shows the vibration period of a coupled system(B+M), wherein the natural vibration period of the building (B) is kept constant and that of the tower (M) was varied. The fundamental period of a coupled system tends to come close to shorter period of the tower or the building, and this tendency is markedly seen with the increase in their period ratio. A similar tendency is also observed in higher mode frequencies as shown in Fig.11.

Fig.15 shows the relation of resonant period and displacement mode obtained from the tests. When the vibration period of the coupled system(B+M) is close to the natural vibration period in case of the tower(M) or the building(B) alone, it shows the tendency that the tower or the building alone resonates.

The relation of vibration period and excitation factor (β_i) shows in Fig.17 that the excitation factor for fundamental vibration mode in case of tower alone is 1.3 and that for secondary mode is 0.3. The excitation factor of a coupled system has the tendency to become larger as the period ratio of tower and building becomes smaller. These values, as compared to the values obtained in case of tower alone, are far greater.

(2) Damping Coefficient

When the damping coefficients of tower alone and of the coupled system are expressed as related to fundamental vibration period as is shown in Fig.16, it becomes evident that damping coefficient in case of tower alone is not vitally affected by period, while in case of the coupled system the damping coefficient tends to increase with the lengthening of fundamental vibration period. Moreover, the period ratio also increases relatively along with the lengthening of fundamental period in the coupled system, and consequently it is observed that the damping coefficient of the coupled system varies with the changes of period ratio.

3.2 VIBRATION TEST OF 1/20 SCALED MODEL

Vibration tests were carried out using 1/20 scaled models of tower, building and the coupled system as shown in Photo.2. The tower model has platforms of steel plate(thickness:9-19mm) and trussed panel sections of steel square bars. The building model is a three-storied building equivalent to six-story height in actual building, where steel plates(thickness:16-19mm) are used as each floor slab and steel square bars(30×30mm) are used as column members. The mass ratio of model of the tower to the building is made to be 1 : 3.

The model was excited with horizontal sinusoidal wave in frequency range of 6-50cps, of which the acceleration of shaking table was kept at a constant level of 0.1g. For each step of excitation frequency, the acceleration and strain were measured at each trussed panel section.

(1) Resonant Frequency and Vibration Mode Shape

The resonant frequencies and vibration mode shapes of the tower, building and the coupled system obtained from the test as shown in Table 4, Fig.18 and Fig.19, are found to conform well to the analytical results. From these table and figures, it is evident that the tendencies on vibrational characteristics of each model are similar to results obtained from full-scale measurement and theoretical analysis on actual towers atop buildings, then results of this model test could be considered to conform to that of an actual steel tower.

(2) Resonant Curve and Damping Coefficient

The measured coupled system model accelarations at various frequencies are plotted as magnification of table acceleration in Fig.20. On this resonant curve, the acceleration-magnifications of the Tower-top-section at each resonant frequency show almost no difference. The damping coefficients based on band-width method, almost coincide with the hypothetical values assumed as to make the measured curve idential with the theoretical curve. Both of these values tend to become smaller along with the rise of frequencies from lower mode to higher modes.

(3) Axial Force and Bending Moment

Axial forces and bending moments on each structual member of the tower and building based on strain measurements, are found to conform well to the analytical results by applying a shearing force obtained from the measured acceleration, as shown in Fig.21 and Table 5. Axial force at each structural member of the tower in case of model of the coupled system is of a larger value at the fundamental resonant frequency than at the time of other resonant frequencies, and also the axial force transferred from the tower to the building is maximum at the time of fundamental resonant frequency. The bending moments of each structural member of the building for fundamental frequency are almost equivalent to those for the secondary mode, and the bending moment at the time of third mode becomes conspicuously small.

4. EARTHQUAKE RESPONSE ANALYSES

Earthquake response analyses were conducted by substituting a tower alone and the coupled Tower-Building system mentioned earlier with a multi-mass dynamic system of linear hysteretic characteristics, using EL CENTRO Earthquake Record (1940 NS 0.33g) and TAFT Earthquake Record (1952 EW) magnified upto 0.33g. In the response analyses the damping coefficients of fundamental vibration mode of the tower and the building are assumed to be internal viscous damping type of 1 % and 5 % respectively.

4.1 IN CASE OF TOWER ALONE

(1) As shown in Fig.22(a), the story displacement tends to increase in the vertical direction of the tower, the value of which is within 3cm (approximately 1/170 of the distance separating the stories) at the position of platforms for almost all towers. The maximum relative displacement of the tower, as shown in Fig.22(b), corresponds to 1/230-1/350 of the total tower height.

(2) The base shear coefficient(tCb), as shown in Fig.22(c), lies within the range of 0.6-1.2 and is close to the value of shear coefficient converted from the wind force for structural design. Distribution of shear coefficients is represented by an inverted trapezoidal shape, where the shearing force coefficient(tCt) of the tower top section is within 1.2-2.0 times the base shear coefficient.

4.2 IN CASE OF COUPLED SYSTEM OF TOWER-BUILDING

(1) The response values in case of the coupled Tower-Building system, vary with the period ratio and mass ratio of the tower to the building, as shown in Fig.23(a), (b) and Fig. 24(a), (b), and reach the maximum when the fundamental vibration periods of the tower and building become close to each other. In this case, the response acceleration becomes larger as the fundamental vibration period of the coupled Tower-Building becomes shorter, while as the period becomes longer the response displacement tends to grow larger. As the mass of tower becomes smaller compared to that of the building, the response value tends to grow larger.

(2) All the response values of the tower in the coupled Tower-Building system are affected by the vibration characteristics of the building and consequently become remarkably larger than the response values in case of Tower alone system; and as shown in Fig.25 and Fig.26 wherein response values are indicated as a multiple of the base shear coefficient, these values reach proportions of 3.5-7.0 times the base shear coefficient when the period ratio is 1.0, and even when the period ratio is 2.0, the response values are in the range of 1.3-2.0 times the factor. The ratio of base shear coefficient to that of the tower top section is affected by the vibration characteristics of the building thus reaching upto 1.3-2.2.

5. CONCLUSION

The steel tower erected on the roof of a building is likely to show resonant phenomenon with the building during earthquake over a wide range of frequency. Besides, it becomes clear that the seismic forces acting on the tower at the time of a strong earthquake might be exceed the design wind force, since the damping of tower is very small. In the present paper, it has been clarified that, it is feasible to practically calculate the seismic forces acting on the tower from the base shear coefficient which varies with the vibrational characteristics of the coupled system of Tower-Building, and the shear coefficient which is distributed in the vertical tower direction in an inverted trapezoidal shape.

Furthermore, no particular problems would arise even if a general method of analysis wherein the acceleration generates at each section of the tower during a strong earthquake, is multiplied by the tower mass as a static load, in order to analyze the stress.

ACKNOWLEDGEMENTS:

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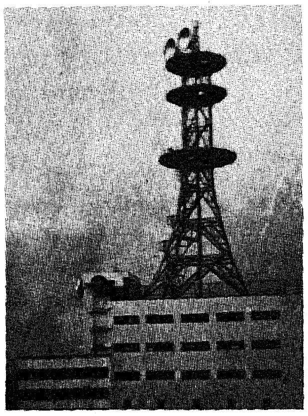


Photo. 1 Profile of steel tower for microwave antennas

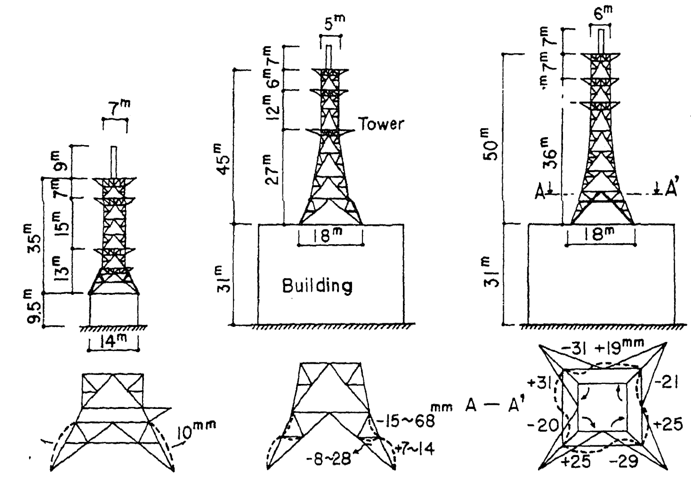


Fig. 1 Damage examples of steel towers in the Tokachioki Earthquake. (1968)

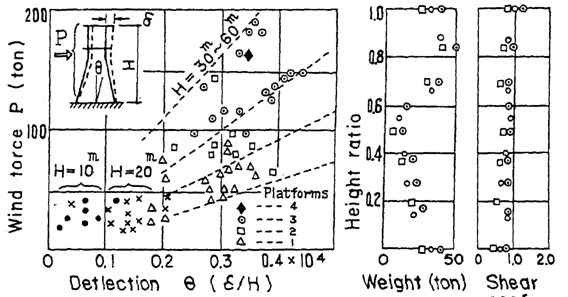


Fig. 2 Wind force. Deflection. Weight · and Shear coet of Tower

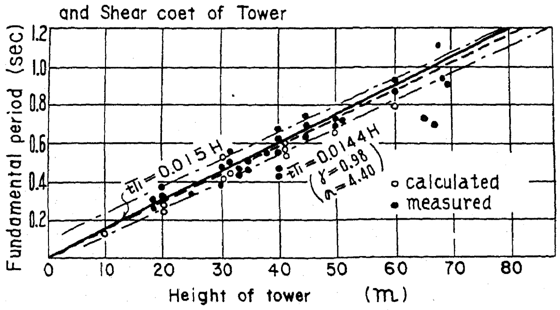


Fig. 3 Fundamental period and Height of tower

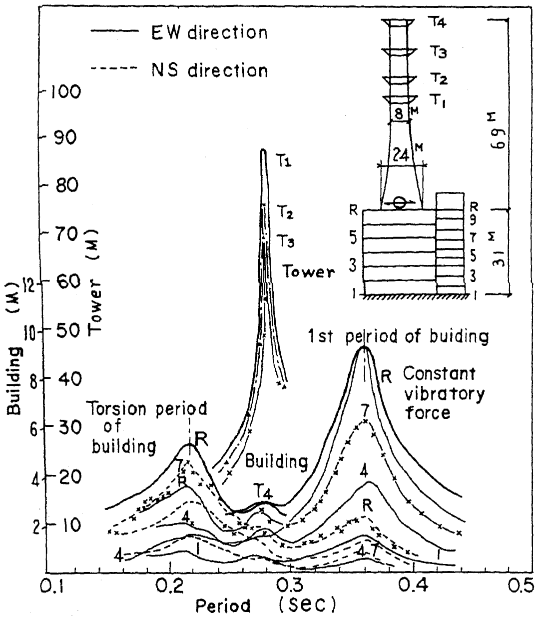


Fig. 5 Resonant displacement curve of tower and building

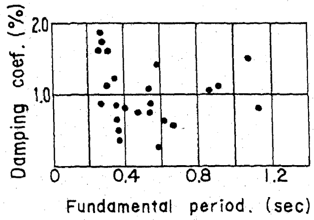


Fig. 4 Damping coefficient of tower

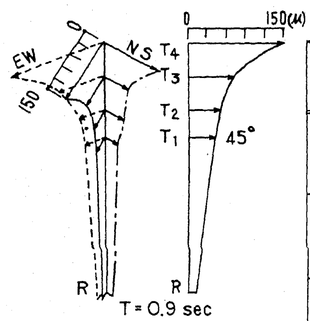


Fig. 6 Fundamental vibration mode of displacement

Table. 1 Resuls of Forced-vibration test

	Resonant period (sec)	Rocking (%)	Damping coefficient
Building N.S	0.35 (1st) 0.217 (distortion)	26.6	0.054
Building EW	0.36 (1st) 0.217 (distortion)	26.0	0.034
Tower	0.90 (1st) 0.218 (2nd) 0.45 (distortion)	———	0.010

Table. 2 Vibration models of tower

NO.	Tower Name	Tower Height (m)	Base Stance (m)	Number of Platform
1	Izumi	10	7	1
2	Awa	20	7	1
3	Fujigaya	20	12	2
4	Matuzaka	30	12	1
5	Tuyama	30	12	2
6	Sizuoka	30	12	3
7	Iida	40	12	1
8	Kamaisi	40	12	2
9	Naoetu	40	12	3
10	Tokusima	50	18	3
11	Funabasi	60	18	3

Table. 3 Vibration models of "Tower - Building"

No. of tower	1	3	4	5	9	10	11
	10m-1PL	20m-2PL	30m-1PL	30m-2PL	40m-3PL	50m-3PL	60m-3PL
Tower. 1st period	0.133	0.216	0.403	0.456	0.521	0.594	0.782
Bldg. stories	2	3	2,3,4,5,6,6	6	6	6	6
Bldg.NO. of mass	2	3	2, 3, 6,	3	3	3	3
Period ratio	1.0,2.0	0.7~2.6 (5steps)	1.0 ~ 3.37 (6 steps)	1.0,2.0	1.0	1.0	1.0
Mass ratio	0.020 0.005	0.005	0.005~0.025 (6 steps)	0.005	0.005	0.005	0.005

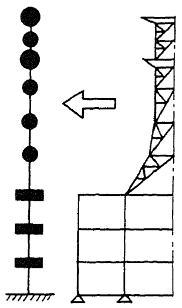


Fig.7 Model of multi-mass system

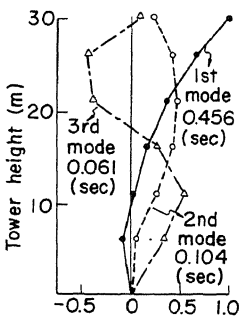


Fig.9 Excitation function of tower

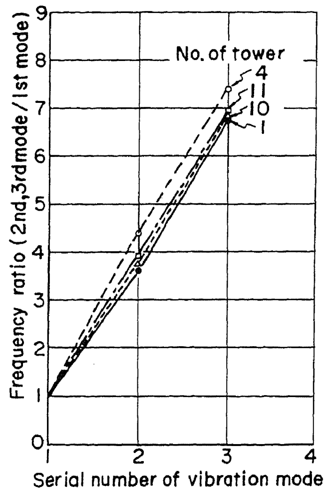


Fig.8 Frequency ratio of tower

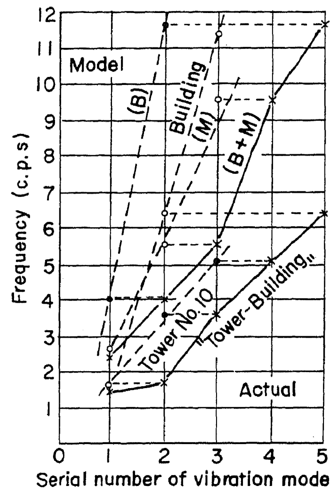


Fig.11 Frequency of "Tower-Building"

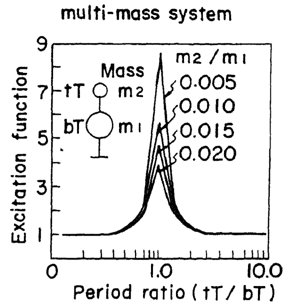


Fig.10

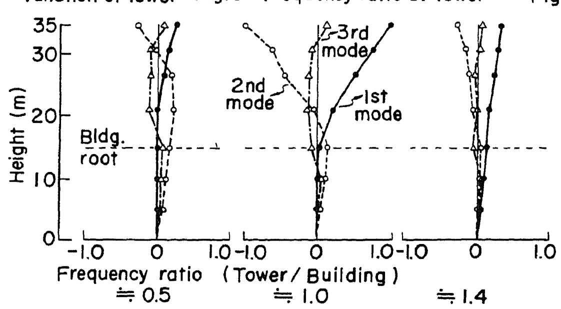


Fig. 12 Excitation function of "20m Tower-Building"

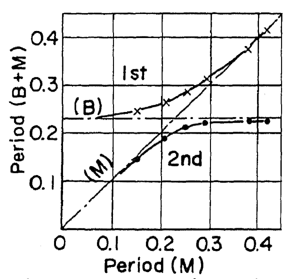


Fig. 14 Period of (B+M)

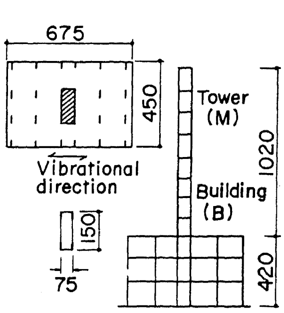


Fig. 13 Dimensions of model for tests

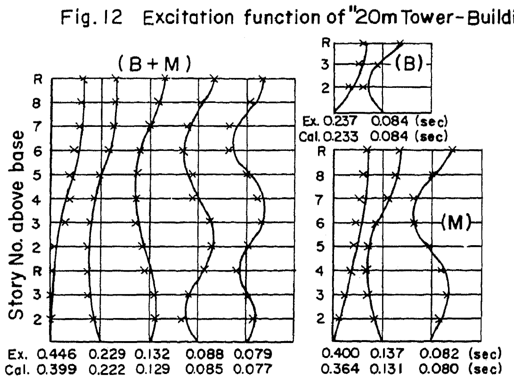


Fig. 15 Resonant period and displacement mode

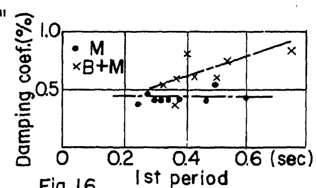


Fig.16

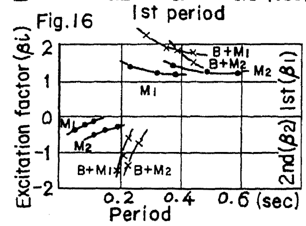


Fig.17 Excitation factor of (B+M)

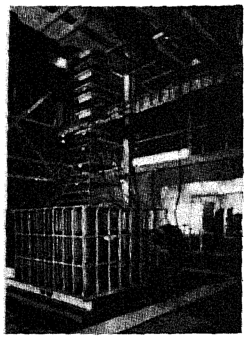


Photo.2 Model of "Tower-Building" for test

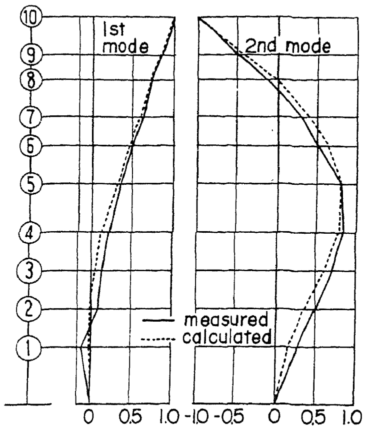


Fig.18 Vibration mode of tower

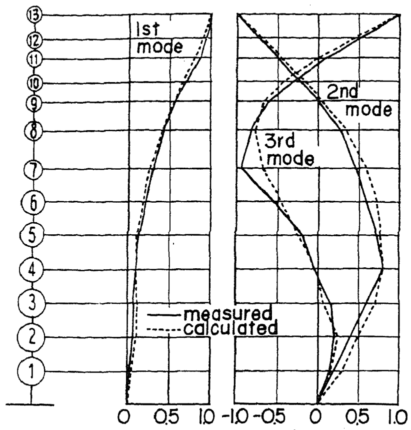


Fig.19 Vibration mode of "Tower-Building"

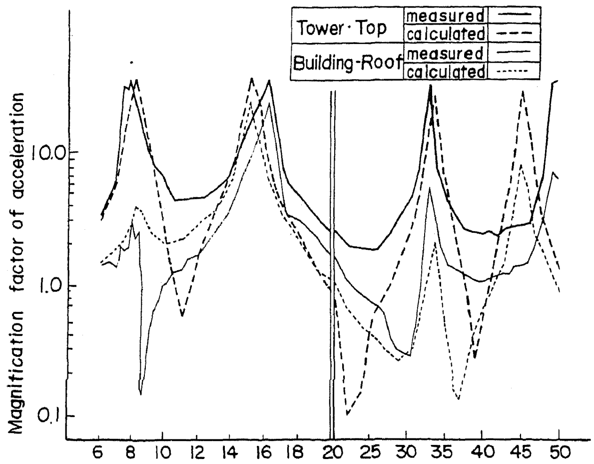


Fig.20 Resonant acceleration curve

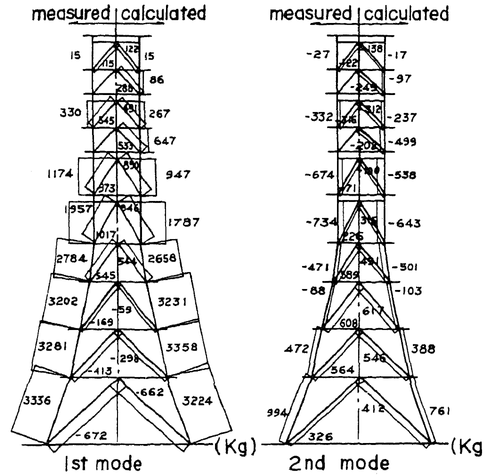


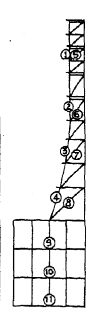
Fig.21 Axial forces of structural members

Table.4 Resonant frequency and damping coefficient

Serial NO. of mode			1st mode	2nd mode	3rd mode
Tower	Frequency	measured	8.71	30.50	—
		calculated	8.90	32.20	—
	Damping coef.	measured	2.00	0.97	—
		assumed	1.50	0.80	—
Building	Frequency	measured	17.10	—	—
		calculated	16.50	48.60	—
	Damping coef.	measured	2.80	—	—
		assumed	3.00	—	—
"Tower-Building"	Frequency	measured	8.06	16.10	33.30
		calculated	8.33	15.20	33.80
	Damping coef.	measured	3.80	2.30	1.20
		assumed	3.00	2.00	1.00

Frequency:(c/s) Damping.coef.: (%)

Table.5 Axial forces and Bending moments of structural members

Model	NO.	N or M	1st mode measured	1st mode calculated	2nd mode measured	2nd mode calculated	3rd mode measured	3rd mode calculated
	1	N	381	348	- 240	- 230	180	180
	2	N	2393	2393	1090	-1150	380	460
	3	N	4084	4377	-1320	-1560	- 10	90
	4	N	4274	4413	- 670	- 890	- 540	- 450
	5	N	572	699	- 320	- 390	190	230
	6	N	1156	1290	- 300	- 360	- 200	- 150
	7	N	- 132	48	360	450	- 370	- 420
	8	N	- 571	- 873	530	810	120	180
	9	N	2712	-3580	- 220	- 350	- 440	- 520
		N	-1356	-1787	-1060	- 980	128	125
	10	N	2474	-3326	- 160	- 330	- 340	- 490
		M	-1237	-1705	-1770	-1514	- 170	- 330
	11	N	2348	-3206	- 190	- 310	- 240	- 470
		M	-1174	-1710	2070	-2018	- 390	- 730

N: Axial force (kg) M. Bending moment (kg·cm)

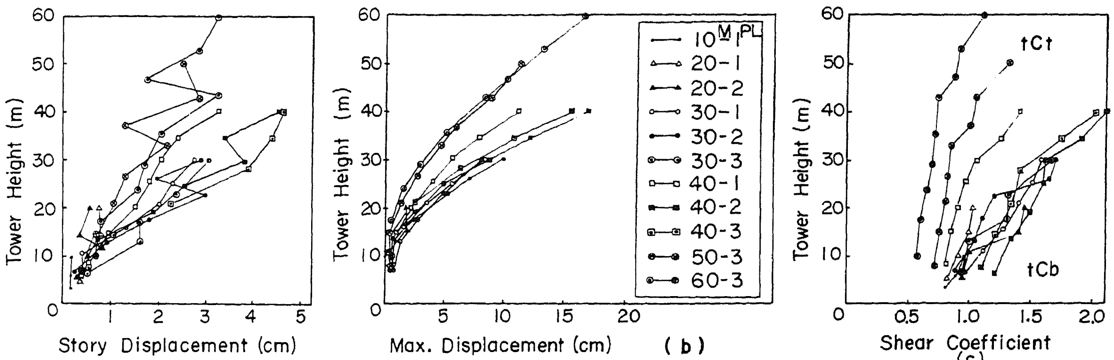


Fig.22 Earthquake Response of "10_60m Tower"

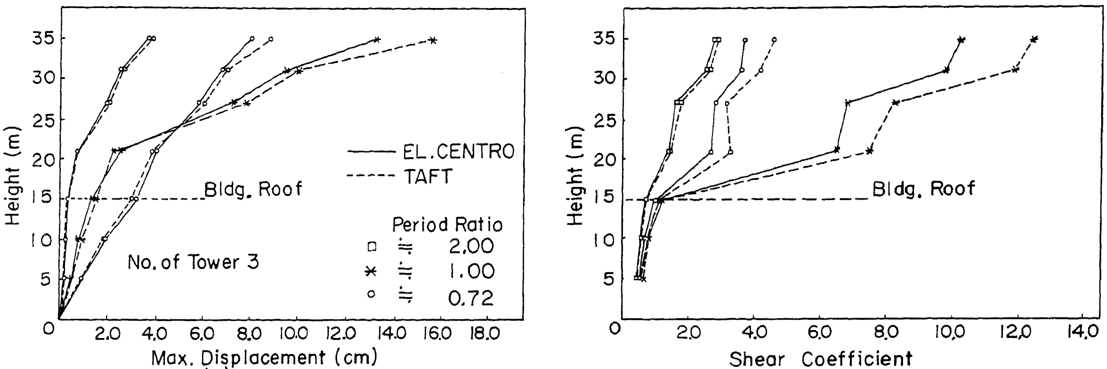


Fig.23 Earthquake Response of "20m Tower-Building"

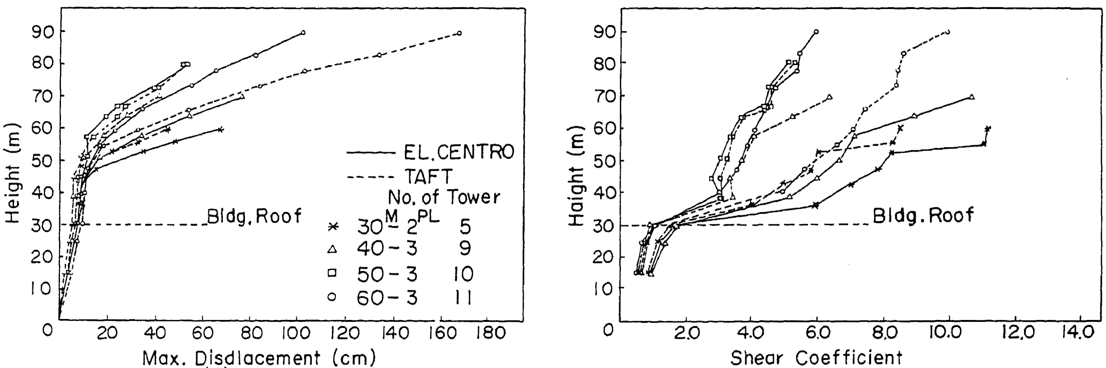


Fig.24 Earthquake Response of "30-60m Tower-Building"

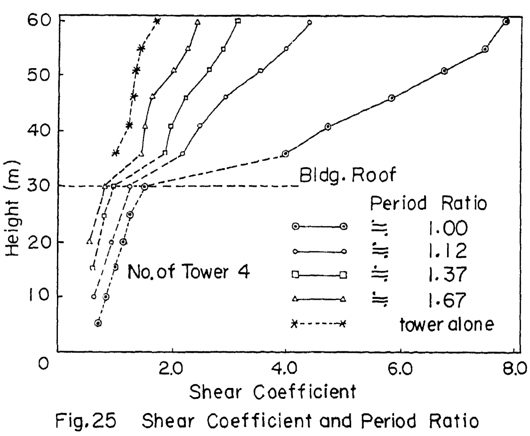


Fig.25 Shear Coefficient and Period Ratio

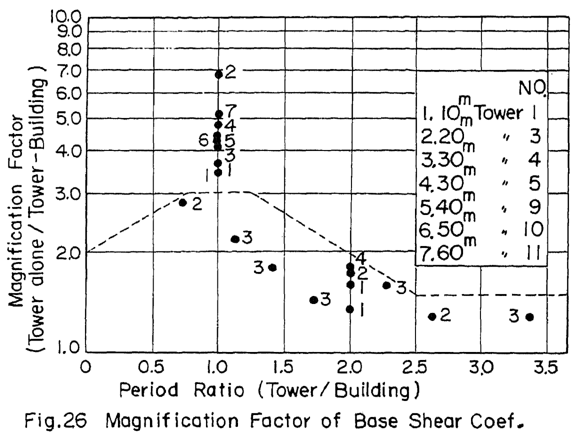


Fig.26 Magnification Factor of Base Shear Coef.