VIBRATION TESTS ON REINFORCED CONCRETE TOWERS

FOR MICROWAVE TELECOMMUNICATION

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SYNOPSIS

Two reinforced concrete towers for microwave telecommunication were subjected to vibration tests. One of these towers has a cylindrical cross section and stands 80m above ground. The other has a rectangular cross section and stands 80m above ground. The dynamic characteristics of the towers determined by these tests are compared to those predicted by lumped mass models and finite element models. The models that should be used in analysis for design of these towers are discussed.

INTRODUCTION

Microwave telecommunication towers are one of the most important facilities in a telecommunication network. About 1,500 of these towers have been constructed so far in Japan, mostly latticed steel towers using structural angle steel. Their dynamic properties have been investigated and were reported in the 5th WCEE[1].

Recently, reinforced concrete was employed to construct microwave telecommunication towers, and three reinforced concrete towers were constructed between 1975 and 1978.

These towers were designed to be elastic for earthquake acceleration almost 0.25g (g; acceleration of gravity). In addition to that, aseismic performance of the towers was confirmed as satisfactory by elasto-plastic response analysis for earthquake acceleration almost 0.4g.

However, the accuracy of the results in large measure depends upon the analysis models of the towers. In order to investigate the model for dynamic analysis of these towers in regard their elastic range, vibration tests were conducted on two full-scale towers, one with a cylindrical cross section and the other with a rectangular cross section. Although dynamic properties of cylindrical cross section towers have already been reported, for instance, for tall chimneys, microwave telecommunication towers are different from reinforced concrete chimneys in that these towers have several platforms on the upper part to mount heavy microwave antennas and are designed to be stiffer in order to limit tower displacement and ensure continuous telecommunication function. Very few reports on dynamic properties of rectangular cross section towers have been published.

DESCRIPTION OF TOWERS

Tarumae tower (called "Tower A" in this paper) is located in Hokkaido. The other is Ishizaki tower (called "Tower B" in this paper), located in Aomori prefecture. Both towers were constructed with reinforced concrete cast in place using slip form method.

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Tower A has a cylindrical cross section and stands 80m above ground. Fig. 1 shows a section of the tower. The diameter of the tower decreases linearly from 7m to 5m from the base to 49m above ground, and in the upper part it stays constant at 5m. The plate thickness of the wall is 0.95m at the base, 0.47m at 49m above ground and 0.4m at the top, and it decreases linearly in each part with increasing height. The tower has four circular platforms for mounting antennas in the upper part. These platforms are cantilever reinforced concrete slabs. The foundation system is composed of a rectangular footing, 24m x 24m, and piles, about 140 in number. The one-story building envelopes the tower, but is structurally separated at the roof.

Tower B has a rectangular cross section and stands 89m above ground. Fig. 2 shows a transversal section of the tower. The dimensions of the tower are 16m x 21m at the base and 7m x 21m constant in the upper part from a height of 45m above ground. The longitudinal walls rise along the circumference of a circle with a radius of 220m from the base to a height of 45m above ground, and they are straight in the upper part. The transversal walls rise straight from the base to the top. The plate thickness of the walls is 0.5m at the base and 0.3m in the upper part from a height of 45m above ground. Besides these peripheral walls, the tower has two interior reinforced concrete walls 0.3m in thickness, one of which rises parallel to the transversal walls from the base to PL-3, nearly at the center of the longitudinal span. The other rises parallel to the longitudinal walls from PL-3 to PL-1, nearly at the center of the transversal span. The tower has two reinforced concrete slabs for machine rooms in the lower part, and three reinforced concrete slabs for antennas in the upper part. The peripheral and interior walls are connected with these slabs. The foundation is a square footing, 30m x 35m.

EXPERIMENTAL PROGRAM

Free vibration tests were conducted in the N-S direction for both towers, and forced vibration tests were conducted in the N-S direction for Tower A and in the N-S and E-W directions for Tower B.

Free vibrations were produced by jet reaction force of a rocket mounted on PL-1. The reaction forces were measured by a load-cell.

Forced vibrations were produced by a rotating-mass vibration generator mounted on the 1st floor for Tower A, and mounted on PL-2 for Tower B. The force of excitation used in forced vibration tests may be computed by the equation Force $= \frac{WR}{g} \cdot (2\pi f)^2$ kg/shaker (g; acceleration of gravity, f; exciting frequency in Hz). The values for WR are listed in Table 1 for Tower A, and in Table 2 for Tower B.

Transducers of the moving-coil velocity type were used to measure the vibrations of the towers during these tests. The signals from all transducers were integrated and recorded as displacement. For the translational motions, five transducers were located at points of interest along the height of the towers. For the transversal motions of Tower A, a transducer was located on PL-1 perpendicular to the excited direction. For the torsional motions of Tower B, two transducers were located near both ends of PL-1. In addition, vertical motions were measured at both ends of the 1st floor of each tower to determine the base rotation.

The locations of the rocket, vibration generator and transducers are shown in Fig. 1 and Fig. 2.

MATHEMATICAL MODEL

Two types of mathematical models were used.

One is a lumped mass model, in only a single plane, and the stiffness between masses was estimated considering the tower as a tapered beam. The purpose of this study is to assess the ability of this lumped mass model to predict the dynamic characteristics of the tower, especially for Tower A.

The other is a finite element model derived by considering the tower as a structure composed of shell elements. The dynamic characteristics of the tower as a three dimensional shell structure (called "the 3-D shell model" in this paper) were computed for both towers. In addition to these characteristics, the N-S and E-W modes of vibration of Tower B were computed separately by Guyan reduction method (called "the Reduction model" in this paper) in order to compare the mathematical results to those obtained from the vibration tests.

Fig. 3 and Fig. 4 show the mathematical models of the towers. Each model was assumed to be fixed at the base. The Young's modulus of concrete is 256 t/cm^2 for Tower A and 269 t/cm^2 for Tower B. These values were determined by the tests on the concrete used in constructing the towers.

DYNA, a dynamic analysis program of lumped mass models, and STRAP, a general purpose structural analysis program using the finite element method, developed at Musashino Electrical Communication Laboratory in NTT, were used to compute the natural frequencies and mode shapes.

TEST AND ANALYTICAL RESULTS OF TOWER A

The motions of PL-1, and the reaction force of the rocket are shown in Fig. 5. The natural frequency and damping coefficient of the 1st mode obtained from the free vibration records are 0.66 Hz and 0.65 percent. The vibration mode shape was obtained from the ratios of displacement of each measured point. The translation and rotation of the basement slab contributed 1 percent and 3 percent, respectively, to the translation of PL-1. Consequently, soil-structural interaction was almost negligible in regard to the behavior of the tower. The displacement in the transversal direction at PL-1 was almost 25 percent of that in the excited direction.

The frequency resonance curves in the region of the 2nd mode are shown in Fig. 6.

The natural frequencies and damping coefficients obtained from the experimental data along with the analytical results are listed in Table 3. Also, the mode shapes are shown in Fig. 7. In this figure, the computed mode shapes are those obtained from the lumped mass model.

The mode shapes computed from the 3-D shell model are shown in Fig. 8. The 3rd mode of the 3-D shell model is the 1st torsional mode of the tower.

The natural frequencies and mode shapes computed from the lumped mass model are reasonably good estimates of these measured for the actual structure, in the first two translational modes.

TEST AND ANALYTICAL RESULTS OF TOWER B

The motions of PL-1 in the N-S direction and the reaction force of the rocket are shown in Fig. 9. The natural frequency and damping coefficient of the 1st mode in the N-S direction, obtained from the free vibration records, are 1.16 Hz and 1.65 percent. The translation and rotation of the basement slab contributed 1 percent and 6 percent, respectively, to the translation of PL-1.

The frequency resonance curves in the N-S direction are shown in Fig. 10. The natural frequency, damping coefficient and mode shape of the 1st mode, obtained from the resonance curves, agreed with those obtained from the free vibration tests.

The frequency resonance curves in the region of the 1st mode in the E-W direction are shown in Fig. 11. The higher modes for the E-W direction were not apparent.

The natural frequencies and damping coefficients obtained from the experimental data along with the analytical results are listed in Table 4. Also, the mode shapes are shown in Fig. 12. In this figure, the computed mode shapes are those obtained from the Reduction model. The computed mode shape, compared to that measured at 4.30 Hz, was obtained from the simple summation of the 2nd and 3rd modes in the N-S direction, considering modal interference.

In comparing the measured and computed natural frequencies of the first two modes, the values computed from the finite element models are reasonably good estimates of those measured for the actual structure. However, this is not so for the lumped mass model, especially in the E-W direction. In the higher modes, the natural frequencies computed from the 3-D shell model are somewhat better estimates than those computed from the Reduction models.

The mode shapes computed from the 3-D shell model are shown in Fig. 13. In viewing these figures, the mode shape, obtained from the above mentioned simple summation of the 2nd and 3rd modes in the N-S direction, is not observed. The 4th mode shape in this figure is mainly torsional. The vibrational characteristics of Tower B are more complicated than those expected from the test results. The individual motions of each wall are observed in these mode shapes.

CONCLUDING REMARKS

The comparison of the measured and computed vibrational characteristics of the towers indicates that the lumped mass model is sufficient for Tower A, but insufficient for Tower B to estimate the dynamic properties. The finite element model of three dimensional shell elements should be used for Tower B. The translation and rotation of the basement slab of both towers were small enough to be negligible. The assumption that each model is fixed at the base can be accepted in analysis for design.

The damping coefficients determined from the tests varied from about 0.7 percent to 2.3 percent. The damping values for aseismic design should be not more than 2 percent for lower modes, though it might be possible to evaluate those larger than the test values, considering larger amplitude during a big earthquake.

REFERENCES

1. Kimura, E. and Konno, T., "Earthquake Effects on Steel Tower Structure atop Building", Proc. of 5WCEE, Rome, 1973.
2. Osaki, Y., "The Use of Jet Reaction for Dynamic Tests of Buildings" Trans. AIJ, No. 142, 1967.

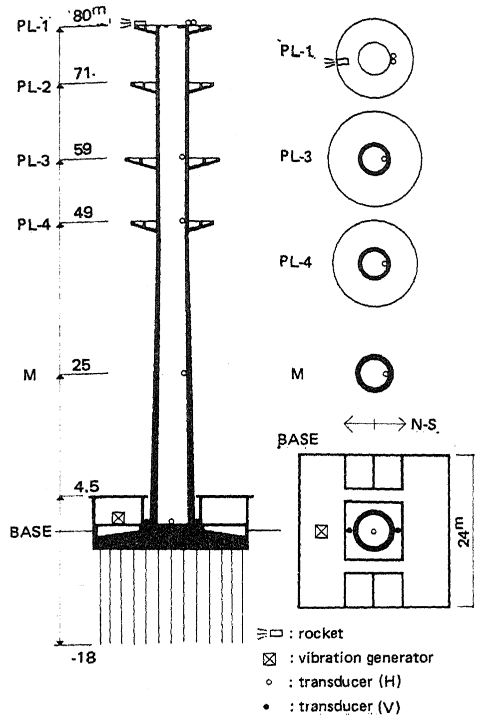

Fig. 1 Section and Plan of Tower A

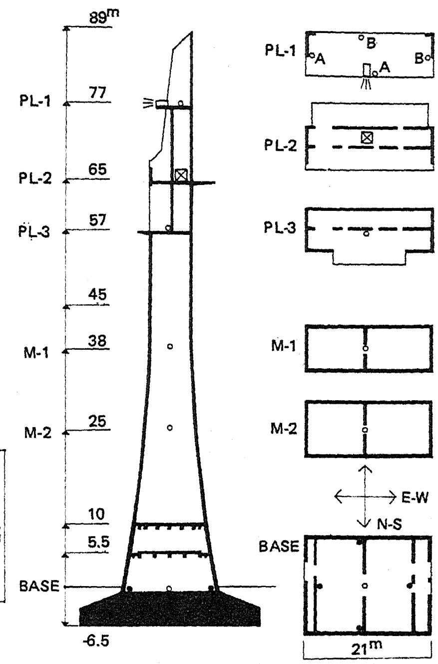

Fig. 2 Section and Plan of Tower B

Table 1 WR for Tower A

f (Hz)	1 - 4	4 - 6	6 - 10
WR (kg·m)	60	20	10

Table 2 WR for Tower B

f (Hz)		1 - 3	2 - 5	3 - 8
WR	N - S	30	20	10
(kg·m)	N - W	20		-

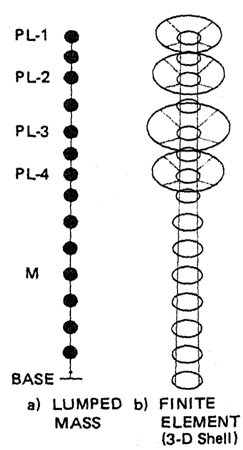

Fig. 3 Models of Tower A

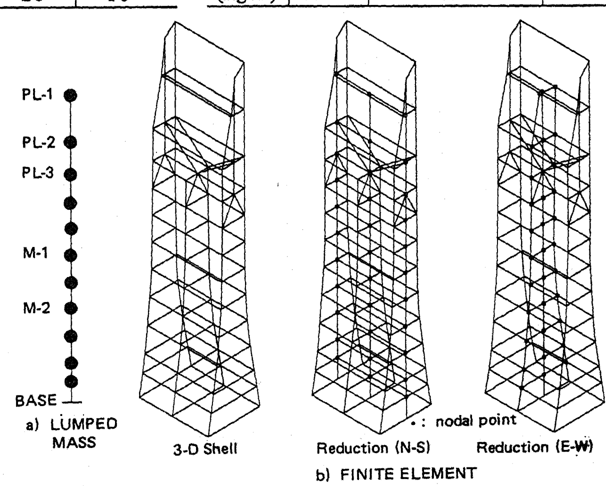

Fig. 4 Models of Tower B

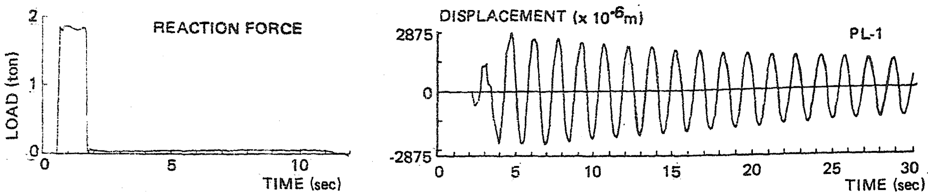

Fig. 5 Records of Free Vibration Test of Tower A

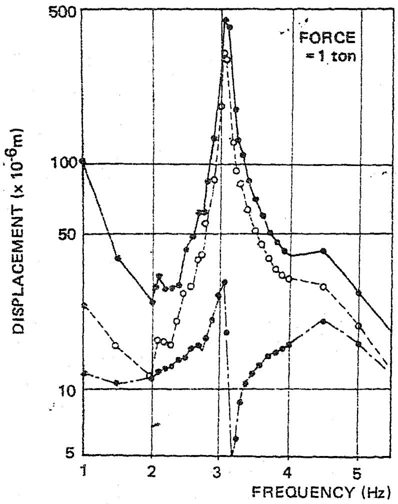

Fig. 6 Resonance Curves of Tower A

Table 3 Natural Frequencies and Damping Coefficients of Tower A

		1	2	3	4
f (Hz)	measured	0.66	3.05	-	7.60
	lumped	0.65	3 17	-	8.10
	3-D shell	0.65	3.11	5.56	7.67
damping (%)		0.65	1.81	-	-

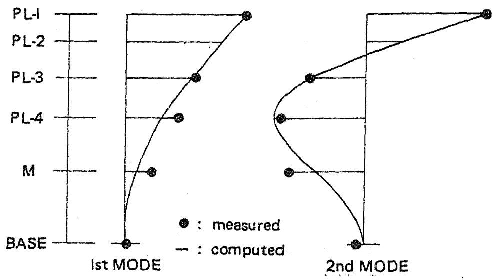

Fig. 7 Mode Shapes of Tower A

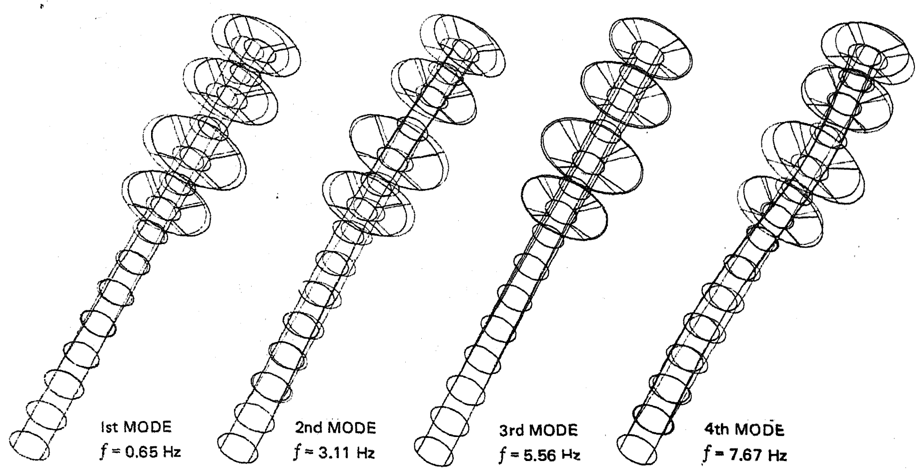

Fig. 8 Mode Shapes Computed from 3-D Shell Model of Tower A

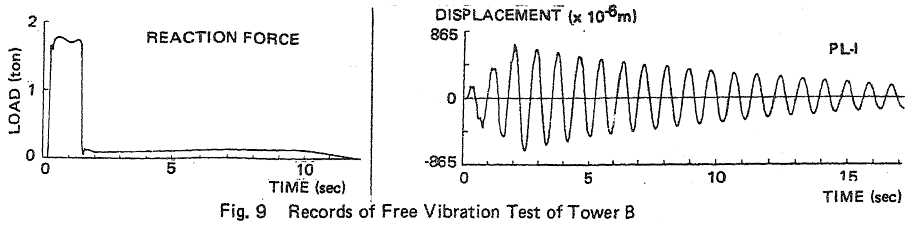

Fig. 9 Records of Free Vibration Test of Tower B

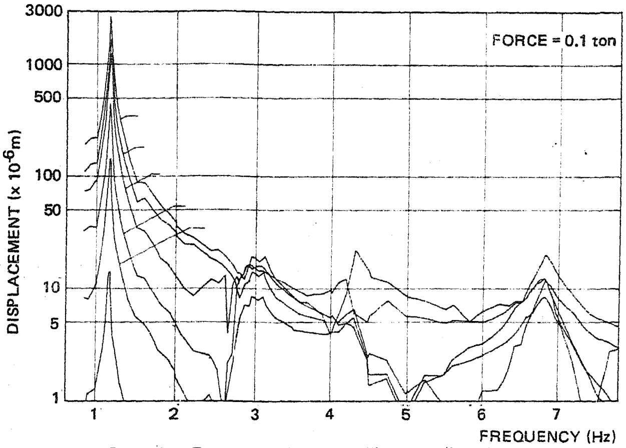

Fig. 10 Resonance Curves of Tower B (N–S)

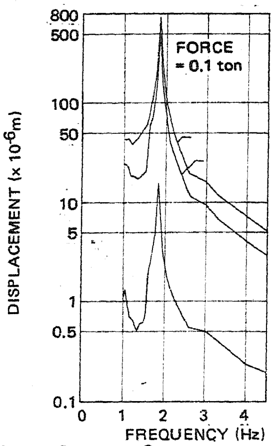

Fig. 11 Response Curves of Tower B (E–W)

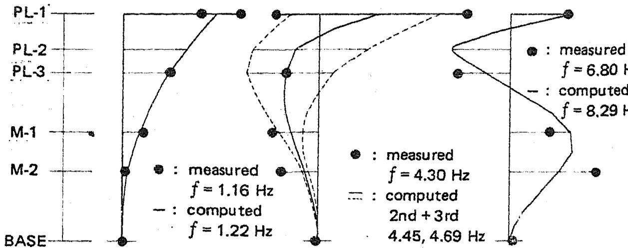

Fig. 12 (a) Mode Shapes of Tower B (N–S)

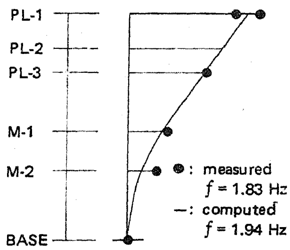

Fig. 12 (b) Mode Shapes of Tower B (E–W)

Table 4 Natural Frequencies and Damping Coefficients of Tower B

			1	2	3	4	5	6	7	8	9
f (Hz)	N - S	measured	1.16		3.03	4.30		6.80			
		lumped	1.34				5.26				
		Reduction	1.22		4.45	4.69				8.00	8.29
	E - W	measured		1.83							
		lumped		2.39							
		Reduction		1.94		4.89				7.91	
	3-D shell		1.21	1.94	3.56	4.65	5.36	7.20	7.43	7.96	8.85
damping coefficient (%)			1.75	1.41				2.28			

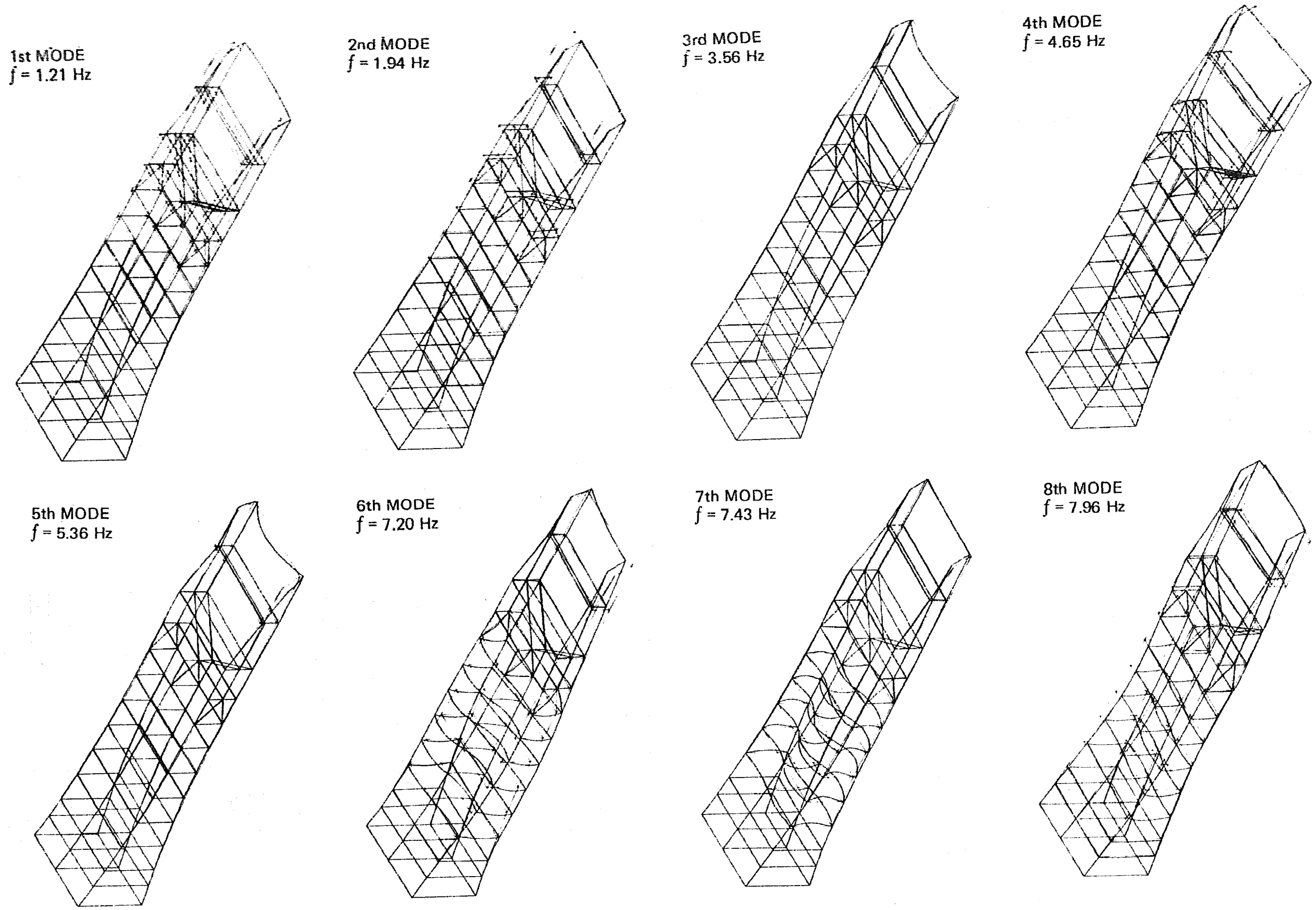

Fig. 13 Mode Shapes Computed from 3-D Shell Model of Tower B