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VIBRATION TESTS OF 29-STOREY FRAME TYPE BUILDING

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SUMMARY

This paper presents a method of full-scale vibration tests on a 29-storey frame type building and the results. Before completion, forced and free vibration tests and microtremor measurements were carried out, and after completion the latter test was conducted. Dynamic properties were determined from these results and compared with the analytical results. Comparison of computed natural frequencies and mode shapes with experimental results showed good agreement. However, the observed damping factors were smaller than the designed value. In addition to these results, the earthquake response observed in this building and the dynamic analysis are shown.

INTRODUCTION

In 1986, a 29-storey steel frame type building was completed in Tokyo, Japan. In order to confirm the safety against earthquakes and evaluate the validity of analysis, vibration tests of this building were carried out twice. For nine months before completion, three kinds of tests, forced vibration tests, free vibration tests and microtremor measurements, were performed. After completion the last one was conducted only. In addition to these experimental results, the results of observed earthquake response and dynamic analysis are described in this paper.

OUTLINE OF BUILDING AND SOIL CONDITION

The building investigated in this study is used as an office and consists of 4 basements, 29 storeys including 2 penthouse storeys. In plan, it is nearly square (i.e. 36.6x36.0 meters) and about 109 meters high. The structural system uses steel frames from the third storey upward, and steel and reinforced concrete composite frames and shear walls below the third floor level. The penthouse is a cantilever type and relatively large. A typical vertical section in the N-S direction and a floor plan of the building are shown in Figs. 1(1) and (2).

This building was constructed in Shinjuku, Tokyo. A typical soil profile obtained from one of the boreholes is shown in Fig. 2. The site is underlain by a layer of gravel known as Tokyo Grave Bed, which is about 24 meters deep. The building is supported on a mat foundation.

VIBRATION TESTS

PROGRAM-----Vibration tests were carried out twice. First nine months before completion, forced vibration tests, free vibration tests and microtremor measurements were conducted in both the translational N-S and E-W directions and the torsional direction. The building was structurally completed and only the finishing work remained. At this time the building was subjected to full dead loads but little live loads. Therefore, the ratio of load condition was about 90% of designed load. Secondly, microtremor measurements were conducted just after completion. Lower floor offices were furnished, but the load condition was almost the same at the time of the forced vibration tests.

MEASUREMENTS-----The experimental apparatus employed were two vibration generators, sixteen structural response transducers and amplifiers associated with integral circuits, a digital data recorder, a FFT real-time spectrum analyzer, and two oscillographs.

Forced vibrations were produced by two rotating eccentric weight vibration generators. These generators are called BCS-B-75 in Japan, a specification of which is shown in Table 1. The vibration generators were mounted at two points on a diagonal line of the 27th floor, as indicated in Fig. 1(2). In the case of translational tests, two vibration generators were synchronized, and in the case of torsional tests, they were operated 180 degrees out-of-phase. Forced vibration tests were conducted within the frequency range from 0.2 to 4 Hz. The value of unbalanced moment was 75 kilogram-meters. Free vibrations were produced by rapid halt of vibration or five men's power. The excited frequency was a fundamental frequency only.

The transducers were moving-coil-pickups, whose natural frequency and critical damping factor were 0.2 Hz and 0.67, respectively. The locations of pickups are shown in Figs. 1(1) through (4). Output signals of amplifiers were digitalized at 100 samples per second and recorded on a digital data recorder.

RESULTS-----Displacement responses obtained by forced vibration tests were scattered during the tests because of strong and random wind, so that the influence of the wind was omitted from the records by the digital band-pass filter. Natural frequencies and damping factors were found from resonance curves. The damping factors were derived with the bandwidth method (i.e. half power method). The records of free vibration tests and microtremor measurements were analyzed with a method of the fast Fourier transform.

The frequency-response curves obtained by translational N-S and E-W, and torsional excitations are presented in Figs. 3(1), (2) and (3), respectively. Five resonance frequencies could be recognized from the N-S resonance curve. The fourth resonance was caused by the local vibration mode of the penthouse. As well as the N-S resonance curve, five resonances in the E-W direction, and three resonances in the torsional direction were found. Transfer functions of microtremors between the 7th floor and the third basemant floor in the N-S and E-W directions are shown in Figs. 4(1) and (2), respectively.

The dynamic properties of the first three translational modes in both the N-S and E-W direction, as well as the first three torsional modes were determined by vibration tests. The results of the forced vibration tests, free vibration tests and microtremor measurement showed very good agreement. And the natural frequencies in the translational N-S and E-W direction for each mode were similar. Furthermore, it was concluded that the vibration of the building are predominantly of the shear type, because the ratios of the natural frequencies were close to the corresponding ratio of a shear beam(i.e. 1:3:5). The damping factors were about 1% of critical damping in all modes. These results are summarized and compared with the analytical results below.

EARTHQUAKE OBSERVATION

Observation of earthquakes has been carried out in this building since completion. The specifications of the observation system are shown in Table 2. Sensor units were installed in the third basement, and 16th and 27th floor. Seismic waves are recorded by the digital cassette recorder. The building was subjected to the Chibaken-Toho-Oki earthquake with a magnitude of 6.7 on December 17 of 1987. Maximum acceleration at the third basement for the N-S motion, was 28.8 gal and for the E-W motion, 25.8 gal. In contrast, the response at the 27th floor was observed at a maximum value of 43.2 gal for N-S and 44.4 gal for E-W. Strong-motion accelerograms recorded on the 27th and the third basement floors in the N-S direction are shown in Fig. 5. The Fourier spectra ratios between the two locations are displayed in Fig. 6.

DYNAMIC ANALYSIS OF BUILDING

An analytical model of this building was the 26-mass system. The steel and reinforced concrete composite structure from the second storey downward was heavier and more rigid than the steel structure from the third storey upward, so that it was fixed at the third floor level. The weight and rigidity were obtained from the structural calculation. The dynamic characteristics of the system were determined. The natural frequencies in the N-S, E-W and torsional directions are summarized in Table 3 with the experimental results. And the vertical mode shapes in the N-S and E-W directions are shown in Fig. 7.

An earthquake motion observed at the third basement in the N-S direction on Dec. 17 of 1987 was applied to the basic model. However, the analytical natural frequencies were different from the observed ones. Therefore, the weight and the rigidity in the model were adjusted until the fundamental frequency and mode shape matched the corresponding experimental value. The damping factor of the fundamental mode was 1.0% (the value was determined experimentally by forced vibration test.). The time response of relative displacement at the 27th floor in the modified model is displayed in Fig. 8, and compared with the observed one.

Natural frequencies of the model formulated by structural calculation were lower than the corresponding experimental ones, because the weight and the rigidity were not evaluated accurately. Then, in the N-S direction the weight and the rigidity were derived from the fundamental frequency. Changes of the weight and the rigidity are shown in Fig. 9, where the designed values are 1. Fundamental frequencies obtained experimentally at both the time of forced vibration tests and the completion were almost the same, because the weight and the rigidity of the building did not change during the period. However, at the time of the earthquake the building was subjected to much live load, so that natural frequencies were lower than ones obtained by vibration tests. The observed rigidity is larger than the designed value since partition walls increase the rigidity. The rigidity at the earthquake was smaller than that at vibration tests. The reason may be that the influence of the rocking vibration was large or the influence of the partition walls was small because of a large response caused by the earthquake. Mode shapes of the basic model showed good agreement with the experimental ones. This proves that the distribution of the weight and the rigidity were set correctly.

Damping factors obtained from vibration tests were about 1% of critical damping in all modes. Furthermore, the response of the dynamic analysis on the model with 1% damping factor was close to the observed one. However, the designed damping factor was 2% of critical damping, which is customarily applied to steel buildings in Japan, and the real damping factor was smaller than the designed one.

CONCLUSION

The dynamic properties of the building were determined accurately by the vibration tests. From the results, natural frequencies for the translational N-S and E-W directions for each particular modes were very similar. The response of the building was the shear type, because the ratios of the first three modes in the N-S, E-W, and torsional modes, were close to the corresponding ratio of a shear beam. A comparison of these dynamic characteristics with analytical properties showed very good agreement. However, the observed damping factors were smaller than the designed damping factors.

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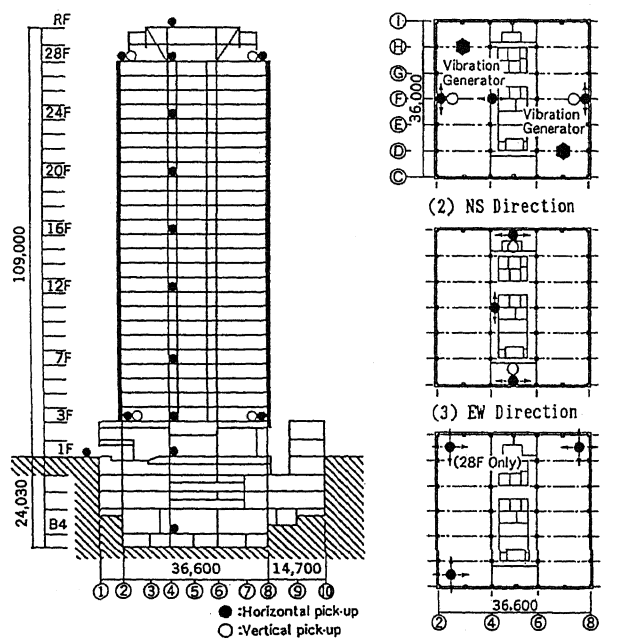
REFERENCES

1. Midorikawa, M., Kitagawa, Y., "U.S.-JAPAN COOPERATIVE RESEARCH ON R/C FULL-SCALE BUILDING TEST, PART 4 DYNAMIC CHARACTERISTICS OF THE BUILDING", 8th WCEE, San Francisco, July 21-28, 1984, VOL. VI, pp.619-626
2. Stephen, R. M., Wilson, E. L., Stander, N., "DYNAMIC PROPERTIES OF A THIRTY STORY CONDOMINIUM TOWER BUILDING", UCB/EERC-85/03, Earthquake Engineering Research Center, University of California, Berkeley, Apr. 1985



(1) Section in NS Direction

(2) NS Direction

(3) EW Direction

(4) Torsional Direction

Fig. 1 Outline of Building and Location of Pick-ups

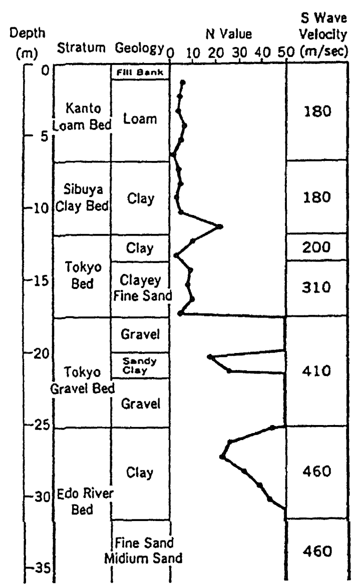
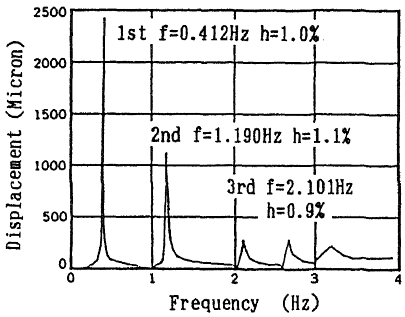


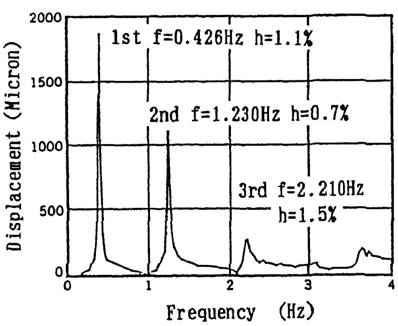
Fig. 2 Soil Profile

Table 1 Specification of Vibration Generators

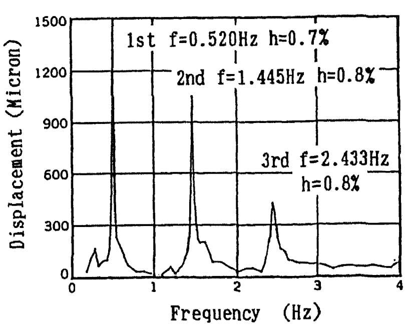
Items	Specifications
Type	BCS-B-75
Unbalanced Moment	1～75 Kg·m
Maximum Generated Force	10 tonf
Frequency Range	0.2～15 Hz
Direction	Two Horizontal



(1) NS Direction

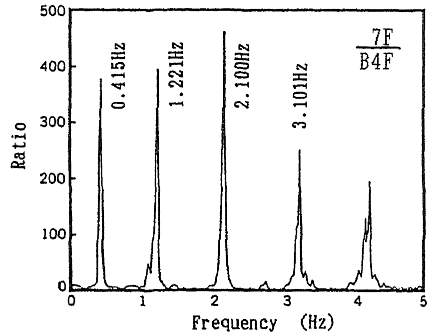


(2) EW Direction

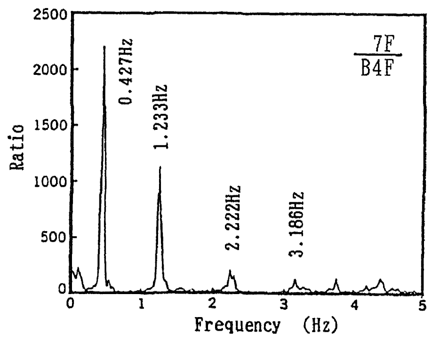


(3) Torsional Direction

Fig. 3 Resonance Curves



(1) NS Direction



(2) EW Direction

Fig. 4 Transfer Function of Microtremores at Time of Forced Vibration Tests

Table 2 Specification of Observation System

Items	Specifications
Sensor	Servo-type Accelerometer 3 components (X,Y,Z)
Signal Conditioner	0.03～1000 gal 0.1～30 Hz
A/D Converter	16 bits 100 Hz (20～200 Hz) 16 channels
Delaying Time Memory	10 sec. (100 Hz)
Recorder	Digital 3M-type, 7700 BPI
Time Code Generator	Clock Adjusted by Radio Time Signal Every One Hour

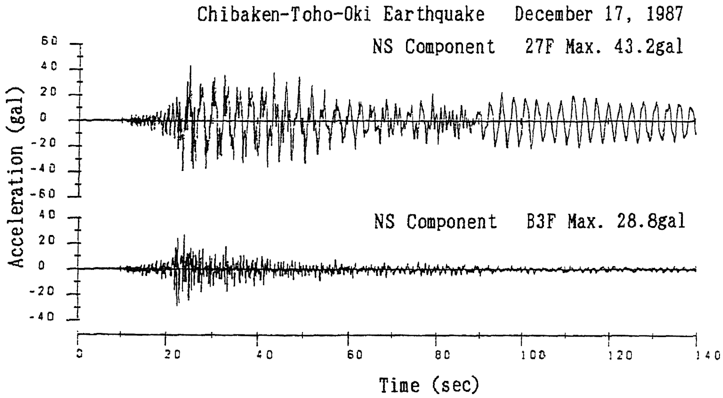


Fig. 5 Accelerograms on 3rd Basement and 27th floor

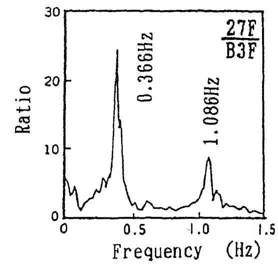


Fig. 6 Fourier Spectrum Ratio of Earthquake Motions in NS Direction

Table 3 Comparison among Natural Frequencies

(in Hz)

Mode	Forced Vibration Tests			After Completion	Earthquake	Design
	Forced Vibration Tests	Free Vibratio Tests	Microtremor Measurements	Microtremor Measurements	Earthquake Observation	
N-S						
1st	0.412	0.415	0.415	0.415	0.366	0.34
2nd	1.190	-	1.221	1.245	1.086	1.01
3rd	2.101	-	2.100	2.185	-	1.79
4th	-	-	3.101	-	-	2.50
E-W						
1st	0.426	0.427	0.427	0.427	0.366	0.35
2nd	1.230	-	1.233	1.306	1.099	1.04
3rd	2.210	-	2.222	2.283	-	1.82
4th			3.186	-	-	2.63
Torsional						
1st	0.520	-	0.525	0.537	-	0.50
2nd	1.445	-	1.480	-	-	1.47
3rd	2.433	-	-	-	-	2.00

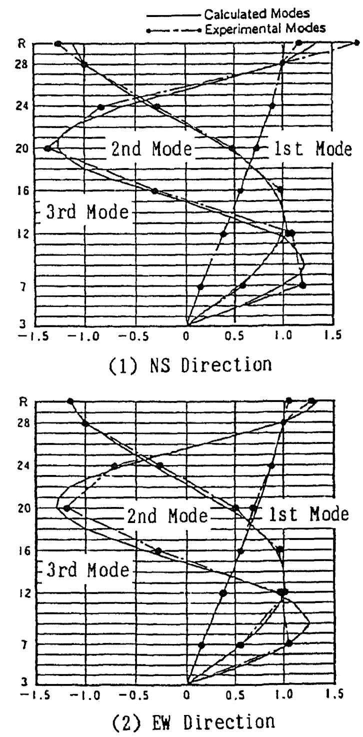


Fig. 7 Mode Shapes

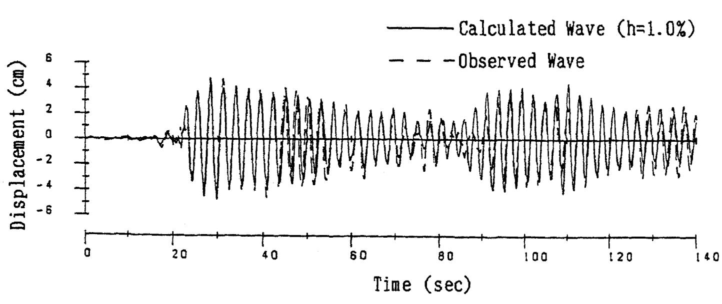


Fig. 8 Comparison between Observed and Calculated Response

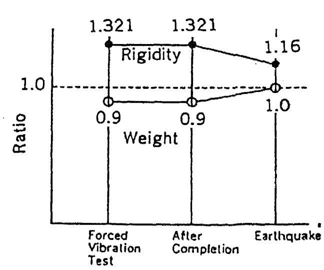


Fig. 9 Change of Weight and Rigidity