

VIBRATIONS OF BUILDINGS IN JAPAN

PART II

VIBRATION TESTS ON VARIOUS TYPES OF BUILDING STRUCTURES UP TO FAILURE

by

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INTRODUCTION

The authors are members of the "Committee for Seismic Tests of Structures", which was organized in 1948 under the sponsorship of the Ministry of Construction. They designed and constructed a large shaking machine and using this machine made vibration tests (1) on several full scale structures. After the work of this Committee was terminated, the authors have been in charge of the dynamic tests of this nature conducted by the Building Research Institute of the Ministry of Construction.

Up to this time, fourteen structures have been tested. Most of these were constructed of incombustible materials newly invented and adopted as a part of the public housing construction program in post-war Japan. The accelerations recorded at the roof level of test structures ranged from 50 to 100 percent of the value of gravity acceleration. Information was obtained on the vibrational characteristics of these structures during the tests and for some this ranged up to the point of failure.

In this paper, the descriptions are made of the test procedures and results that might be of particular interest to structural engineers in the field of engineering seismology. Detailed information on the overall vibration testing of these structures will be presented in a later publication.

DESCRIPTION OF THE LARGE SHAKING MACHINE AND THE METHODS OF DYNAMIC TESTING

The large shaking machine used in the vibration tests is of the horizontal rotating type having a single arm and is shown in Fig. 1. This machine is capable of producing a centrifugal force of 15 tons at 300 revolutions per minute and 1.5 tons at 60 r.p.m. The design requirements for this type of a machine are low cost, a light weight to facilitate portability, efficient use of an electric variable-speed motor, and as low a plane of rotation as possible. The eccentric moment about the shaft can be increased from 40 kg-meter to 346 kg-meter by attachment of additional lead or steel plates to the tip of the rotating arm.

Most of the test structures were two story structures. The large machine was installed on the roof slab to produce dynamic lateral force

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and a small vibration generator of the ordinary counter-rotating type placed on the second floor slab to produce minute vibrations. A photograph of this setup is shown on Fig. 2.

The vibration of a structure induced by the large machine in the horizontal plane is considered as an elliptical motion. Therefore, three-component strong-motion accelerometers with a calibrated sensitivity of 80-100 gals to 1 millimeter and a natural frequency of 20 c.p.s. were used at each floor to measure simultaneously the vibrations. For the small vibration tests, the ordinary vibration generator produced periodic centrifugal force in one direction and was oriented to the direction desired. The seismometer used in these tests had a pendulum with a period of 2.8 sec. and a magnification of 13. Description of other instruments is omitted due to the limitation of the paper.

VIBRATIONAL RESPONSE AND FAILURE OF TEST STRUCTURES

Test structures: The construction types, dimensions, and other features of the test structures are shown in Table 1.

Resonance curves: The resonance curves obtained for the vibrations of small amplitudes are not presented herein because they present the usual features of linear oscillation. To indicate the vibrational behavior of large amplitudes, a test result obtained for a prestressed concrete framed structure is shown in Fig. 5, where the accelerations are plotted for the forcing periods. In some tests on masonry structures, the vibrational behaviors almost reached resonant conditions but could not progress through resonance because of the lack of motor capacity.

Maximum accelerations observed: Vibration tests using the large machine were made on all structures tested. The eccentric forcing moment was increased gradually. Table 2 shows the vibrational features of the test structures when the maximum accelerations were observed.

Failure of test structures: In the tests on reinforced concrete-block masonry structures, no structural failure was observed except for hairline cracking in mortar joints. A brick masonry structure resisted quite well the lateral force equivalent to Code design standards but suffered failure as the test progressed (see Fig. 3). In the test of a prefabricated reinforced concrete frame structure, wall panels inserted in the frame fell off due to the vibrations, after which bending failure occurred in the column bases and successively in the column tops. In a steel frame structure with concrete-block walls, cracking progressed between the frame and the wall panels and finally only the frame was resisting the exciting force. The failure of a prestressed concrete structure occurred first at the ends of the beams and was followed by failure at the column bases. Fig. 4 shows the structural failure of a "thermocon" structure which was caused in some measure by faulty pouring of the material.

DISCUSSION ON TEST RESULTS

Consideration of vibrational behavior: In the vibrations of concrete masonry structures which have a considerable structural stiffness, it has been recognized that the deformations of the base soil has a great effect on the vibration amplitudes of the structures. For ex-

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ample, in the test on a "MATSUI" type structure, about 30 percent of the displacement observed at the roof level was due to structural deformation. The remaining 70 percent was due to the soil deformation with about 35 percent of this amount due to the swaying motion and 35 percent due to the rocking motion of the structure as a whole. It should be noted that in the large amplitude vibrations, the footings at each side lifted alternately, parting from the base soil and making a so-called "rocking" motion. This phenomenon sometimes caused failure of the middle portions of the foundation beams and the reaction in general caused the base soil to deform to a saddle shape.

On the other hand, framed structures which had a smaller proportional stiffness in relation to the deformability of the base soil than did the masonry structures had predominantly structural deformation. The effect of the base soil was small. An observation obtained on the roof level displacement of a prestressed concrete structure, for example, indicates that only 4.9 percent of the displacement was due to swaying, 5.4 percent was due to rocking and the rest was caused by structural deformation. Thus the vibration of a framed structure might be treated, in general, as that of a multi-degree of freedom system with fixed base support deflecting in shear.

Elongations of natural periods: In Table 3, the natural periods of vibration of the test structures are shown at various stages of vibration. Fig's 6 and 7 indicate how the resonant circular frequencies and amplitudes change according to the progress of structural failure.

Relationship between restoring forces and deformations in resonance: The dynamic lateral force acting upon a building structure can be calculated by measuring the acceleration at each story and the phase difference between the structure and the shaking machine. In resonance, however, it can be assumed that the phase difference is nearly 90 degrees even in a multi-degree of freedom system with ordinary dampings. Thus the relationship between restoring forces and structural distortions in resonance can be obtained from the measurement of accelerations. Fig. 8 shows this relationship obtained for the first story of a light steel framed structure from small vibrations to large vibrations. In this figure, the results of the static loading tests made on the same structure are shown for comparison. It is noted that the dynamic and static test results show good agreement and the structural stiffness decreases with the increase of the deformation.

Theoretical analysis: For the analysis of vibrations of masonry structures, the authors presented a paper (2) wherein the structure was treated as a perfectly rigid body supported by horizontal and vertical springs at the base with the structural deformability amalgamated into the vertical spring coefficient. Furthermore, the vibrational behavior having non-linear characteristics in the larger amplitude range was interpreted by a simplified analysis which assumed the structure as a one degree of freedom system with a fixed rotational axis under the ground level.

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A framed structure which has rather small stiffness can be analyzed as an ordinary multi-degree of freedom system having viscous damping. To explain the behavior in non-linear vibrations with large amplitudes, a method of analysis has been proposed by the authors (3). Experimental results show good agreement with values derived by use of the proposed method.

Coefficients of subgrade reaction of base soils obtained from test results: From the test results obtained for masonry structures, the values of the vertical and horizontal spring constants employed in the analysis given in the reference paper (2) can be calculated. Dividing them by the area, or moment of inertia, of the base, the horizontal (shear) or vertical coefficients of subgrade reaction of the base soils will be obtained, respectively. These values for loam (intermediate density) are presented in the paper (2), and have generally the following ranges:

The coefficients of horizontal subgrade reaction, 1 to 3 kg/cm^3 .

The coefficients of vertical subgrade reaction, 10 to 15 kg/cm^3 .
The coefficient of subgrade reaction is a measure of the ability of the soil to resist dynamic vertical deformation. This value is represented as a spring constant for theoretical treatment.

Equivalent periodic ground motions and dampings: The design lateral force coefficient required in the Japanese Building Code is 0.2 ξ , basically, with some modifications. According to the Code requirement, all of the test structures can be considered to have had the required resistance for lateral force. Larger accelerations than the Code value were produced in the test buildings without failure. However, it should be noted that the accelerations of the equivalent stationary ground motions calculated from the test results have a rather small magnitude and are shown in the Table 4. In the calculations for buildings No. 3, 10 and 11, they are assumed for simplification as two-degrees-of-freedom systems having linear structural stiffness. After finding the structural stiffness and resonance factors for observed modes and periods, the equivalent ground motions are computed to have almost the same amplitudes at each corresponding story level. For the No. 7 building, the calculation was made on the basis of the non-linear vibration theory (2). If the dampings are considered for each story, as proposed in the author's paper (3), story values for a prestressed concrete frame structure as fractions of critical damping, D1 and D2, become:

D1 = 0.041, and D2 = 0.03 in small linear vibrations
or, D1 = 0.057, and D2 = 0.094 in large non-linear vibrations after failure.

The dampings of the vibrations with small amplitudes can also be estimated from the resonance curves experimentally obtained, and are considered to be from 0.03 to 0.05 of critical damping. It is to be noted from the above mentioned analysis, that the increase of the values of the dampings due to failure is not very great and the values assumed for viscous dampings for large non-linear vibrations are considered to be from 0.06 to 0.1 of critical damping.

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CONCLUSIONS

The authors made vibration tests on various types of test building structures erected for this purpose utilizing a large shaking machine on the roof slabs of these structures to exert a dynamic lateral force by means of a rotating eccentric mass. As a result of the large accelerations produced, most of these test structures suffered varying degrees of structural failure. The information on the dynamic behavior up to failure, especially the variations of amplitude, natural period and damping, was obtained in association with the dynamic characteristics of site base soils. By means of these tests, a relationship between Code design forces and the dynamic lateral forces producing failure were established.

Of course, the acceleration patterns of actual strong motion earthquakes are different from those observed in these vibration tests using the large shaking machine. However, if the dynamic relationship between restoring force to the deformation and distortion of the structure can be determined through the elastic and plastic range, a good concept of its aseismic characteristics would be obtainable by investigating the response to measured strong motion earthquake accelerograms.

Building vibration studies of this nature are being continued, if only on an intermittent basis, and it is hoped that further developments may be expected by the time another World Conference is held.

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FIGURE CAPTIONS

- Fig. 1: Large shaking machine used in vibration tests.
- Fig. 2: A vibration test of a "THERMOCON" structure.
- Fig. 3: Failure in a brick structure. (A total failure crack extended to the foundation footing.)
- Fig. 4: Failure of a "THERMOCON" structure.
- Fig. 5: Accelerations of a prestressed concrete frame structure. (Shaking machine on the roof slab.)
- Fig. 6: Amplitudes of vibrations in a brick structure, (at frequency A, hair cracks occurred; at B, the total failure crack shown in Fig.3 appeared.)
- Fig. 7: Amplitudes of vibrations in a prestressed concrete frame structure. (At frequency A, cracks occurred; at B, bending failure occurred at the bases of columns; at C, failure expanded abruptly.)
- Fig. 8: Relationship between restoring force and amplitude for a light steel frame structure with concrete block walls.
- Table 1: Construction types and dimensions of test structures.
- Table 2: Vibrational features of test structures at their maximum accelerations.
- Table 3: Natural periods of test structures.
- Table 4: Equivalent ground motions and dampings calculated from large vibrations test results.

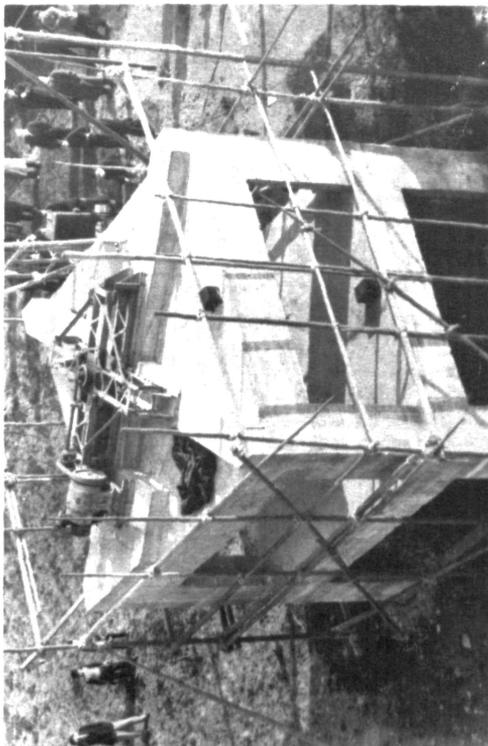


Fig. 2

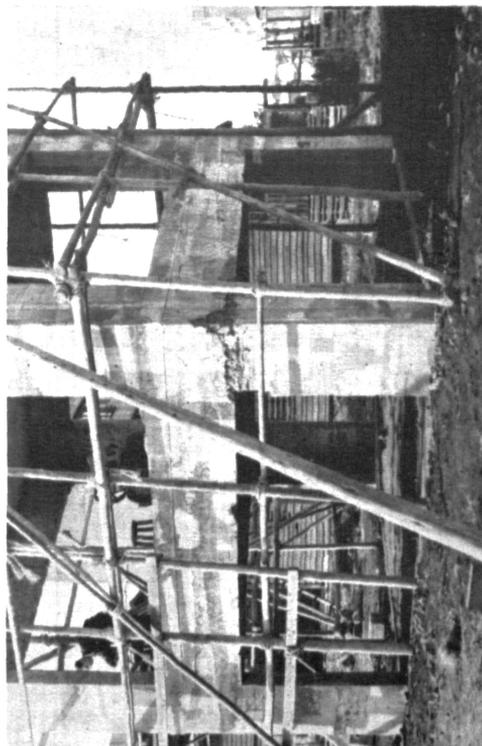


Fig. 4

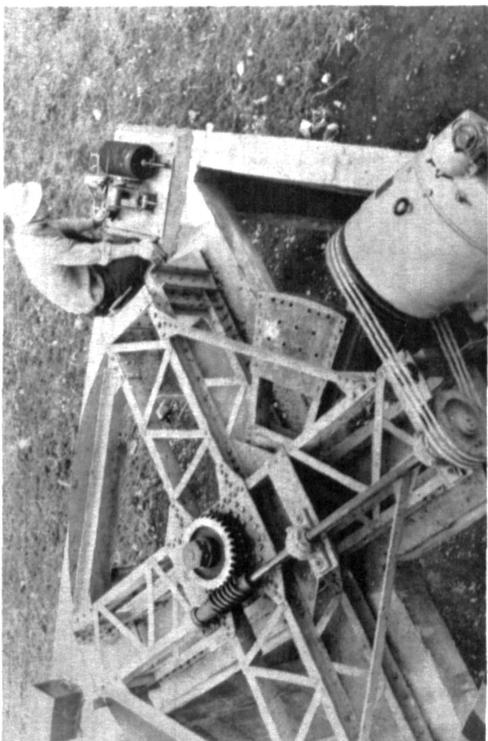


Fig. 1

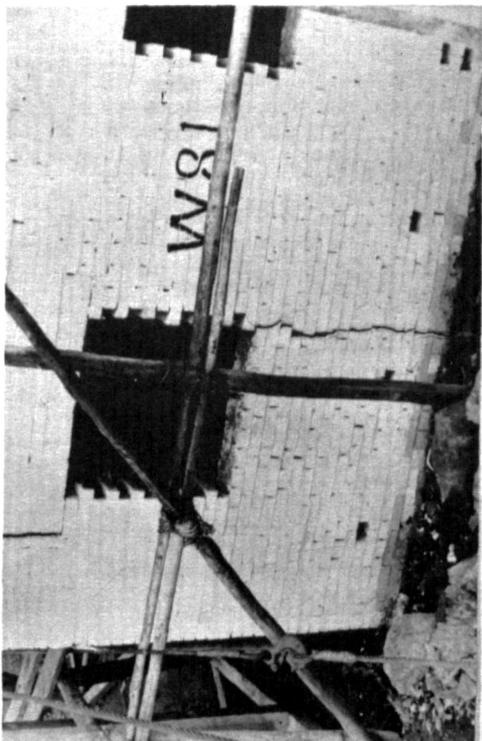


Fig. 3

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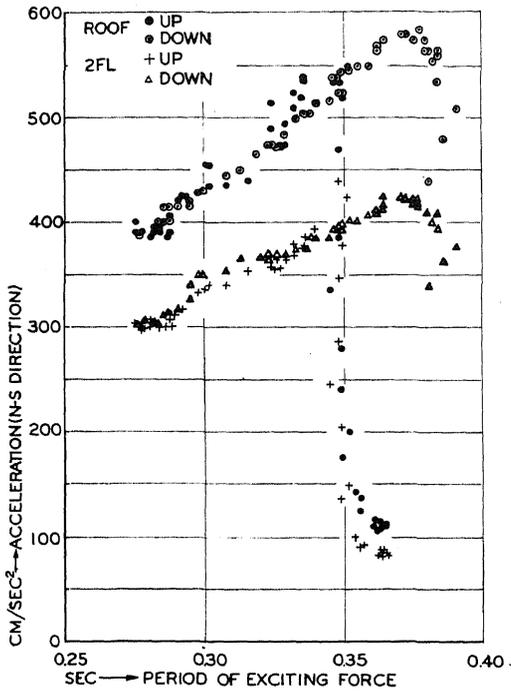


FIG. 5

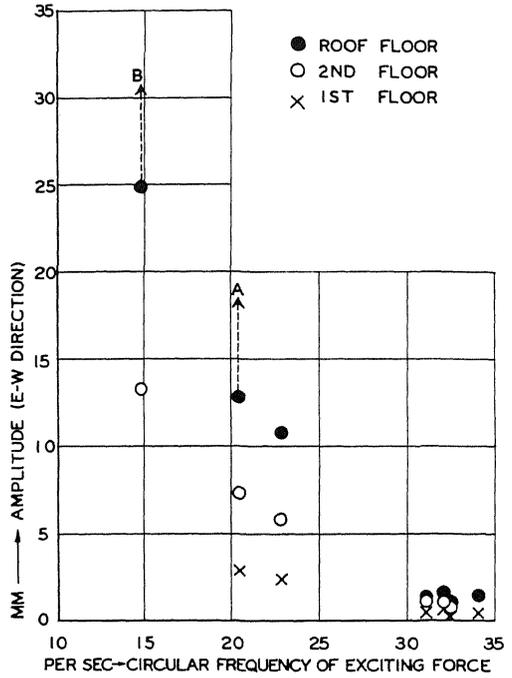


FIG. 6

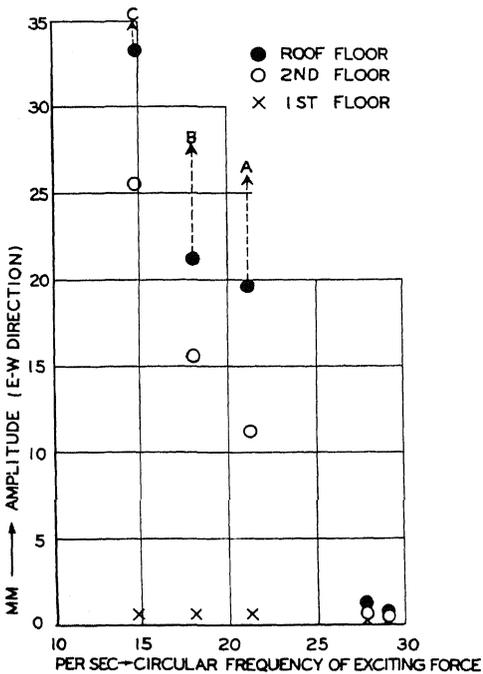


FIG. 7

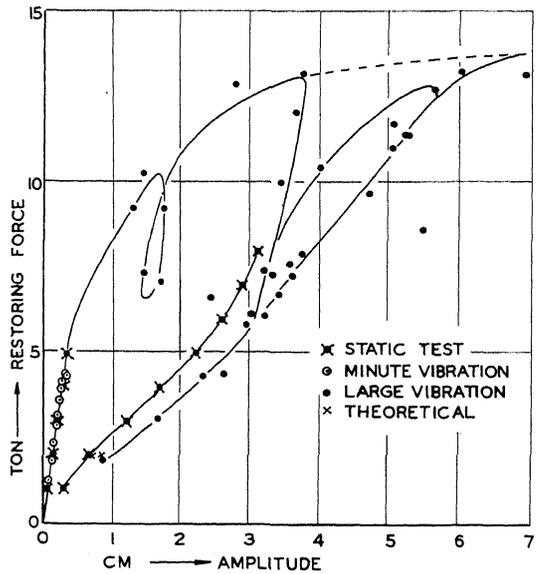


FIG. 8

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Test structure		Construction	No. of Stories	Plan m x m	Ht. m	Wt. t	Found. Type	Length of walls per unit floor area	
No.	Type							NS cm/m ²	EW cm/m ²
1	PASKIN	Reinforced concrete block, precast stressed skin	3	6.0 x 4.0	8.79	9.69 (R) 7.25 (3) 7.56 (2)	Cont. ftg.	—	—
2	R.C. Apt. $\frac{1}{2}$ Size	Reinforced concrete wall type	3	6.42 x 3.28	4.51	9.9 (R) 11.2 (3) 11.4 (2)	Cont. ftg.	—	—
3	TOKEN	Reinforced concrete frame, Prefabricated	2	4.0 x 4.0	6.10	—	Single ftg.	50	25
4	SASAKI	Reinforced concrete block single shell	2	3.79 x 5.60	5.6	15.1 (R) 21.6 (2)	Cont. ftg.	25.4	35
5	MATSUI	Reinforced concrete block Amer. Std. Block	2	3.48 x 4.30	5.8	21.1 (R) 20.0 (2)	Cont. ftg.	49.0	37.4
6	KIKAKU	Reinforced concrete block frame type	2	4.0 x 4.0	5.7	16.2 (R) 23.7 (2)	Cont. ftg.	25.0	25.0
7	Brick	Brick	2	4.15 x 4.59	4.79	16.1 (R) 19.9 (2)	Cont. ftg.	25.3	29.9
8	NAGANO	Reinforced concrete block frame type	1	3.6 x 3.6	2.73	16.2 (total)	Cont. ftg.	15.4	24.6
9	R.C. Frame	Reinforced concrete frame	2	2.5 x 5.0	4.0	19 (total)	Cont. ftg.	0	—
10	Light Steel	Light steel frame and bracing encased in light aggregate concrete	2	4.2 x 4.2	7.58	10.3 (R) 13.3 (2)	Cont. ftg.	12.3	12.3
11	P.S. Concrete	Prestressed concrete frame	2	4.5 x 5.5	5.6	15.5 (R) 10.3 (2)	Single ftg.	0	0
12	Low cost R.C. Apt.	Reinforced concrete frame wall type	1	3.5 x 3.5	3.17	?	Cont. ftg.	14.7	14.7
13	Low cost C.B. Apt.	Reinforced concrete block A.S.B.	1	3.0 x 3.5	3.17	?	Cont. ftg.	14.3	14.3
14	THERMOCON	Cellular concrete, wire-mesh reinforced	2	5.18 x 5.18	5.28	35.8 (total)	Cont. ftg.	11.9	11.9

TABLE

Test structure		Eccentric moment kg-m	Direct.	Natural period sec.	Roof floor of 3 story bldg.		3rd floor, or roof floor of 2 story bldg.		2nd floor, or roof floor of 1 story bldg.		1st floor	
No.	Type				Accel. cm/sec ²	Displ. cm.	Accel. cm/sec ²	Displ. cm.	Accel. cm/sec ²	Displ. cm.	Accel. cm/sec ²	Displ. cm.
1	PASKIN	—	N S	0.325	610	1.64	427	1.14	292	0.783	171	0.458
2	R.C. Apt. $\frac{1}{2}$ Size	—	E W	0.264	725	1.28	554	0.977	385	0.680	245	0.433
3	TOKEN	144.6 144.6	E W N S	0.690	—	—	552 970	6.65 3.83	470 809	5.67 2.37	—	—
4	SASAKI	144.6	N S	0.425	—	—	717	4.21	287	1.65	121	0.695
5	MATSUI	198.3	N S	0.413	—	—	683	2.97	440	1.91	281	1.22
6	KIKAKU	59.0	E W	0.321	—	—	604	1.57	428	1.12	219	0.570
7	Brick	94.3	E W	0.355	—	—	595	1.90	309	0.987	131	0.419
8	NAGANO	117.2	N S	0.319	—	—	—	—	869	2.25	—	—
9	R.C. Frame	—	—	—	—	—	—	—	—	—	—	—
10	Light Steel	78.4	N S	0.58	—	—	805	6.89	440	3.77	149	1.27
11	P.S. Concrete	39.6	E W	0.296	—	—	885	1.97	506	1.13	56	0.124
12	Low cost R.C. Apt.	136.6	N S	0.377	—	—	—	—	1440	5.20	385	1.39
13	Low cost C.B. Apt.	—	E W	0.384	—	—	—	—	1440	4.27	465	1.74
14	THERMOCON	59.0	E W	0.387	—	—	655	2.49	366	1.39	129	0.49

TABLE 2

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Test structure		NO. See Table I	Direct.	Natural period, small Vibrator sec.		Natural period, large Vibrator sec.		Nat. p. small Vib. sec.	Ratio of (4) to (1)
Type				(1) Start of tests	(2) End of tests	(3) Start of tests	(4) End of tests	Rerun using small Vibr.	
Masonry	SASAKI	4	N S	0.178	0.187	0.286	0.425	—	2.38
			E W	0.158	0.162	0.266	0.425	—	2.68
	MATSUI	5	N S	0.171	0.174	0.282	0.415	0.195	2.43
			E W	0.159	0.163	0.282	0.415	0.168	2.61
	KIKAKU	6	N S	0.178	0.181	0.323	0.381	—	2.14
			E W	0.182	0.194	0.323	0.381	—	2.09
	Brick	7	N S	0.187	0.219	0.291	0.434	0.239	2.32
			E W	0.193	0.197	0.291	0.434	0.207	2.25
Framed	TOKEN	3	N S	0.205	0.208	0.396	1.10	—	5.36
			E W	0.229	0.256	0.456	1.10	—	4.82
	Light Steel	10	N S	0.28	0.33	0.435	0.905	0.675	3.25
			E W	—	—	0.435	0.841	—	—
	P.S. Concrete	11	N S	0.209	0.238	0.323	0.439	0.278	2.1
			E W	0.218	0.227	0.296	0.424	0.251	1.94

TABLE 3

Test structure		Direct.	Accel. value	Eccentric moment kg-m	Natural period sec.	Acceleration cm/sec ²				Fract. of crit. damp.
No.	Type					roof floor	2nd floor	1st floor	ground motion	
3	TOKEN	E-W	Obsvd	144.6	0.69	552	470	0	—	0.064
			Calc.	—	0.69	550	470	70	70	
7	Brick	N-S	Obsvd	39.6	0.306	418	246	123	—	0.063
			Calc.	—	0.306	418	290	123	51	
10	Light Steel	N-S	Obsvd	78.4	0.58	805	440	0	—	0.04
			Calc.	—	0.619	710	387	47	47	
11	P.S. Concrete	N-S	Obsvd	39.6	0.323	641	479	0	—	0.056
			Calc.	—	0.325	636	472	45	45	
		E-W	Obsvd	39.6	0.296	888	506	0	—	0.061
			Calc.	—	0.292	913	521	98	98	

TABLE 4