VIBRATIONAL CHARACTERISTICS OF BUILDINGS

Part 1

Vibrational Characteristics of Actual Buildings Determined by Vibration Tests

by

Morio TAKEUCHI *

Introduction In this paper, are summerized the results of the experimental studies relating to the vibrational characteristics of about sixty buildings which were of steel-framed reinforced concrete and reinforced concrete constructions (rahmen-type constructions).

In the experiments of these buildings, the vibrator was used to set them into forced vibrations with small amplitudes. From the synchronization, we could determine the natural periods of vibrations and further, from the amplitudes of vibrations measured on various floors, we could make clear the features of deflection, torsional and rocking motions which occurred in the buildings.

Here, we are proposing a simple empirical formula which gives the relation between the period of vibration and the height of the building considering the effect of the aseismic walls in the building. Surely, the height of the building is the most essential factor which determines the natural period, as is clear in the fact that the period increases approximately with increasing height of the building. A further examination of this relation, however, reveals that the existence of the outside and inside aseismic walls of the building may not be overlooked as the secondary factor which gives effect upon the period of vibration. The other factors are not so effective that they may be neglected.

Method of experiments

The forced vibration of small amplitudes was imparted to the building by means of the vibrator which was essentially consisted of three wheels, each having an eccentric mass. The rotative speed of this vibrator is 7 revolutions per second in the maximum. During the rotation of the wheels, the exciting period changes gradually. At the time of resonance, the amplitude of the building-vibration becomes maximum. The period at that time may be practically determined as the natural period of the building. Further, from the amplitudes thus become maximum on various floors, we can determine the feature of the deflection of building at the time of resonance. Similarly, from the amplitudes of horizontal motions measured at various points lying on a certain floor, the torsional vibration can be determined. The rocking of the building is usually determined from the vertical component-motions at various points on the roof and the basement floors.

The instruments used in these measurements are the horizontal and

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vertical component-motions seismographs which are of the electro-magnetic type. Beside these, mechanical seismographs were used as the auxiliaries to calibrate the magnifications of the electro-magnetic seismographs.

As mentioned before, the rotative speed of the vibrator can be attained to 7 rev./sec., the eccentric force which is generated at this speed is estimated to be 2.4 tons. The eccentric masses weigh 60 kg in total and they are boltted to the wheels at an eccentric distance of 22 cm, Fig.I.

Mechanical constants of seismographs

(1) Electro-magnetic seismograph, Fig.2.

By changing the pick-up (transducer), we can measure the horizontal or the vertical component-motion. According to the magnitude of the ground motion, a suitable magnification of the seismograph is obtainable by changing the gain of the amplifier. In most of the cases, the magnification is adjusted to be nearly 700~1,000.

Results of measurements

Mainly, the buildings which stood inTokyo,Osaka and their environs were measured. They were about sixty in number. Their dimensions and the periods of vibrations are shown in Table I.

As already mentioned, the wall-ratio, γ , which is the value obtained by dividing the total length of the walls (m), being measured in the plan, by the sum of the floor-areas of all stories (m^2). The thickness of the wall is not considered in our case.

The value of γ is generally less than unity, and practically, it is less than 0.1.

As to the deflection of the building, the following facts have been found. In the sound and safe building, the deflection may be represented generally by a smooth curve corresponding to an elastic deformation, Fig. 3. However, in the building which was damaged at their footings at the time of the great Kwanto earthquake of 1923, the deflection curve shows a peculiar form as if it were of a hinged-free beam. At a glance of this curve, we can easily detect the damage caused in this building.

In the case of a remarkable unequality in the areal distribution of the stiffness, the torsional vibration of the building can be seen occurring with a centre lying in the part of higher stiffness. As references, some examples are shown in Fig. 3.

These results serve well as good data for the general inspection of the safety of the construction members.

In the following paragarphs, a consideration will be made on the natural period of vibration and the occurrence of the rocking, basing on the results of the measurements.

It is clear that the period of vibration is a powerful factor by which we can estimate the stiffness of the building. It may be thought that among the factors in the structure which give influence on the period are the height (above ground surface), the width and the length. So, trials were made to find out the relation between these factors and the period by taking the factors individually or in some combinations, but we have found that the height has the most remarkable and linear relation with the natural period of the building, Fig. 4. Especially, this relation can be seen clearly in the building in which the measurements were made before and after the additional construction works with the results that the periods increased with increased heights.

The existence of the aseismic walls is also considerable as a powerful factor which gives influence upon the period. It has been thought that in the earthquake-proof design of the building now made in Japan, the existence of the walls has a decisive effect upon the vibrational characteristics of the building. This fact has been proved by our experiments which were performed statically, as well as, dynamically with rubber and steel models for these several years. As a reference, the results of these experiments will be described as below.

(a) The stiffness of the building will be increased by increasing the aseismic walls, even if they were few in number, and regardless of their positions.

(b) It is desirable to distribute the walls separately in the building wholly, when they are seen in the plan and elevation, especially it is most effective to distribute the walls in the diagonal directions in the elevation.

(c) In the building which stands on an elastic ground, the rocking is generally observable. The amount of this motion has been found to be inversely proportional to the natural period of vibration. In other words, the rocking is remarkable as the stiffness of the building becomes higher relatively to the hardness of the ground.

(d) The stiffness of the building as a whole is lowered by the existence of the openings made in the walls. The rate of decrease in the stiffness is influenced by the peripheral length of the opening rather than its area. In the case of the openings of equal areas, the long and slender opening is undesirable from the aseismic point of view and the square opening is preferable.

As is clear in these results, it may be said that the number of the walls in the building has a remarkable effect upon the period of vibration.

As mentioned in (b) and (d), the wall has a complicated relation with the period of vibration such that the period depends on the distribution of the walls and also on the shape of the opening in the wall. As we are now proposing a practical formula which is available for the period of vibration, we do not desire to make things complicated. Therefore, only the result described in (a) is considered and the effect of the wall- distribution will be overlooked. In other words, we are assuming that the walls have the same effect upon the period without regard to the positions where they are distributed. The shape of the opening in the wall is not also taken into consideration and only the effective lengths of the walls are considered which are determined in the plan. It must be added that in the actual building, the effect of the difference in the shape of the opening may be comparatively so small that it will be negligible. Further, the thickness of the wall is not considered in our investigation.

As to the aseismic walls, their effect upon the period may be represented by introducing the wall-ratio, γ , which has been described before. To find out the effect of this ratio, we determined the deviations of the points from the straight lines which is shown in Fig.5 and plotted them in the ordinates against the wall-ratio in the abscissae. Although, there is some complicated relation, yet there can be seen a general tendency that the deviation shows a decrease in value when the wall-ratio becomes large and it increases when the wall-ratio becomes small.

From these considerations, it may be said that the effect of the wall-ratio which is calculated for the aseismic walls upon the period of vibration is secondary when the effect of the height is taken as the primary factor. Thus, in Fig. 6 , the values of $4 + H(1 - 4\gamma)$ are plotted in the abscissae versus the periods in the ordinates. As will be seen in this figure, most of the points are distributed in the sectorial area bounded by two lines which may be expressed by the following expression.

$$T = \frac{1}{50} \sim \frac{1}{80} \{4 + H(1 - 4\gamma)\}$$

where, T denotes the period of vibration in second, H the height of building above the ground in metre and γ the wall-ratio.

Thus, the period of the steel-framed reinforced concrete and the reinforced concrete buildings may be calculated approximately by this expression if the height and the wall-ratio are known.

For the points which deviate upward from the sectorial area remarkablly, the following remarks may be made.

Buildings Nos.8, 16 and 23 were once damaged by fire in the past ; buildings Nos.9 and 25 are top heavy owing to the installed machinaries and the heavy structural members specially designed to support them ; in building No.11, all structural members are rather small in their cross-sections. The construction work is thought to be imperfect and the safety factor may be comparatively small. Building No.26 is of a new type of construction in which the aseismic walls are distributed few in number, especially in the first storey. A fairly large deviation from the sectorial area suggests the defective nature of this type of construction against the vibration.

Thus, the formula which we are proposing is helpful to determine the dynamical safety of the building from the period of vibration actually observed.

Although, there might be a relation between the rocking and the stiffness of the building relative to the hardness of the ground, the effect of the rocking upon the period is included implicitly in the period actually measured.

The shape of the plan, that is, the length and the width of the building are not taken into account directly, but their effects may be included in the wall-ratio to some extent.

Conclusion

The period of vibration of the structure afore-mentioned, may be calculated approximately by the expression given in this paper. The building of which period ranges between the values calculated by this expression may be said to be sound and safe, whereas if the actual period deviates considerablly upwards from the sectorial area, the building may be judged as defective.

It must be added that in the studies so far made, the relation between the vibration of the building and the ground condition has not been taken into consideration. This relation will be left for our future study.

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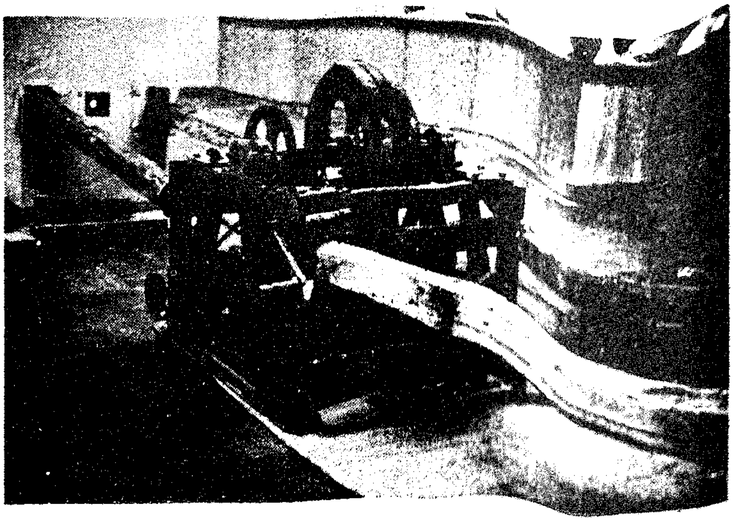

Fig. 1

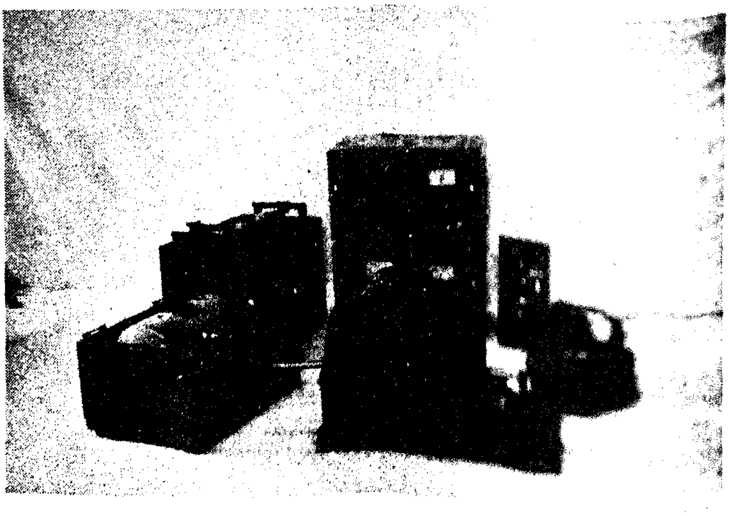

Fig. 2

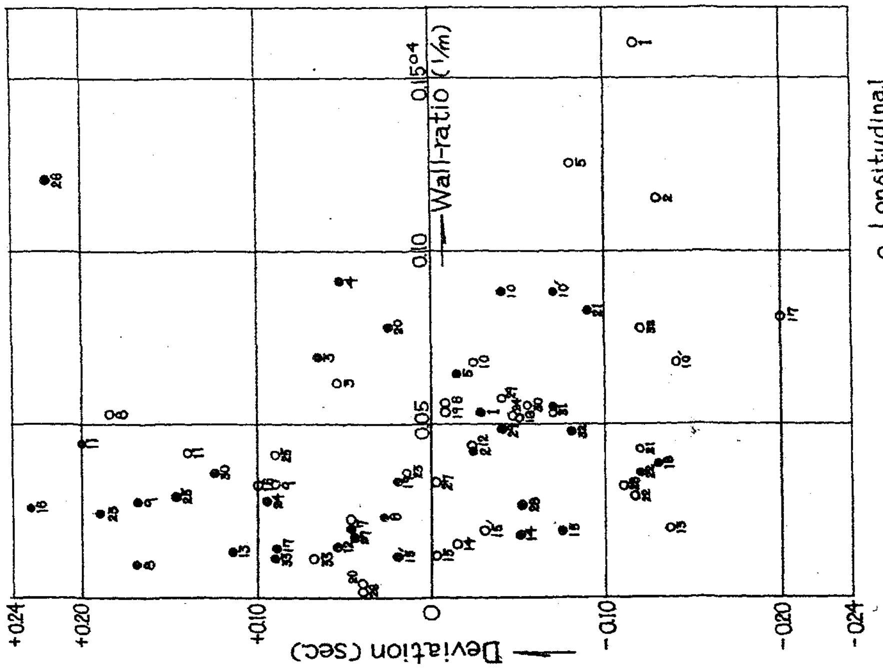
Wall-ratio (1/m)
Deviation (sec)
0.05
0.10
0.15
+0.24
+0.20
+0.10
0
-0.10
-0.20
-0.24
o Longitudinal
● Transverse

Fig. 5

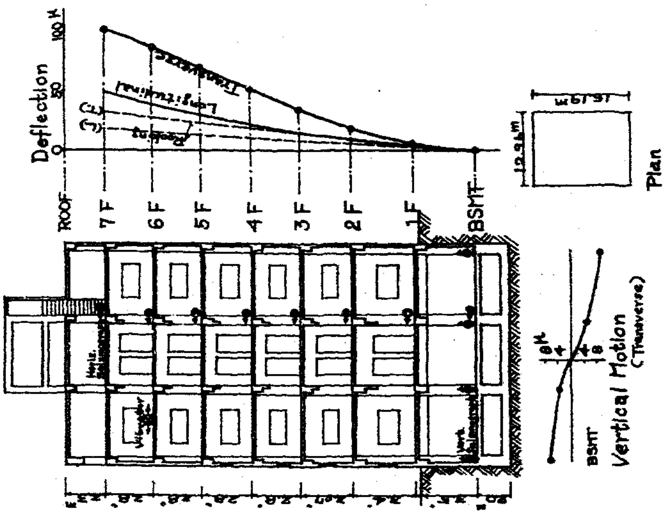
Deflection
ROOF
7 F
6 F
5 F
4 F
3 F
2 F
1 F
BSMT
Plan
Vertical Motion
(Transverse)

Fig.3

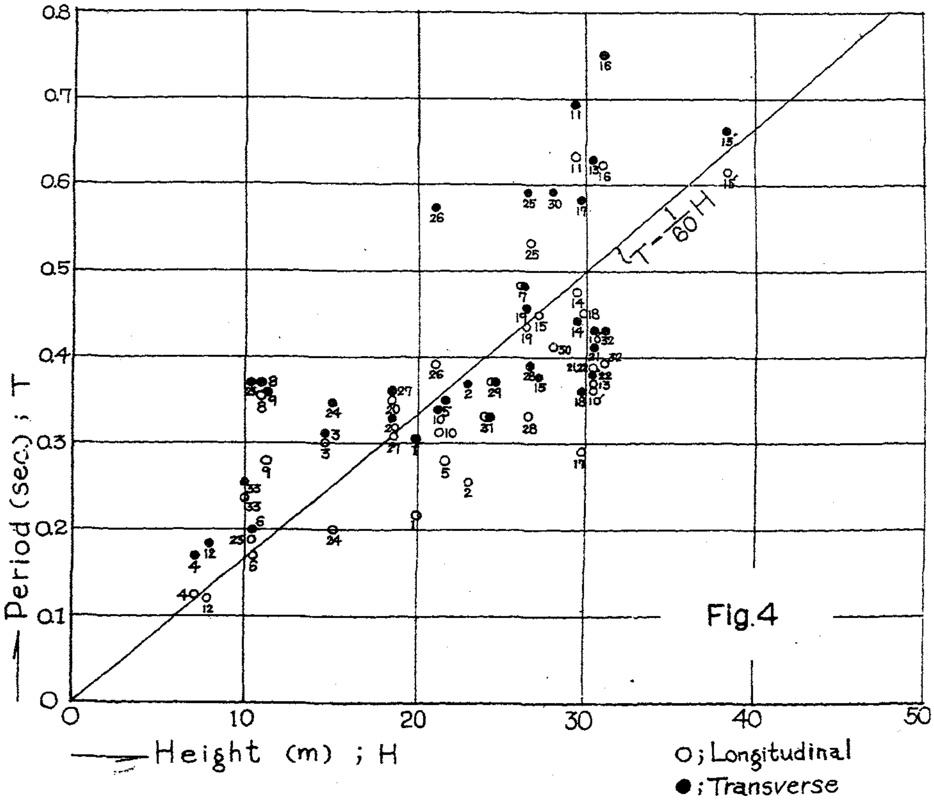
Period (sec.) ; T
Height (m) ; H
$T = \frac{1}{60} H$
o ; Longitudinal
● ; Transverse

Fig.4

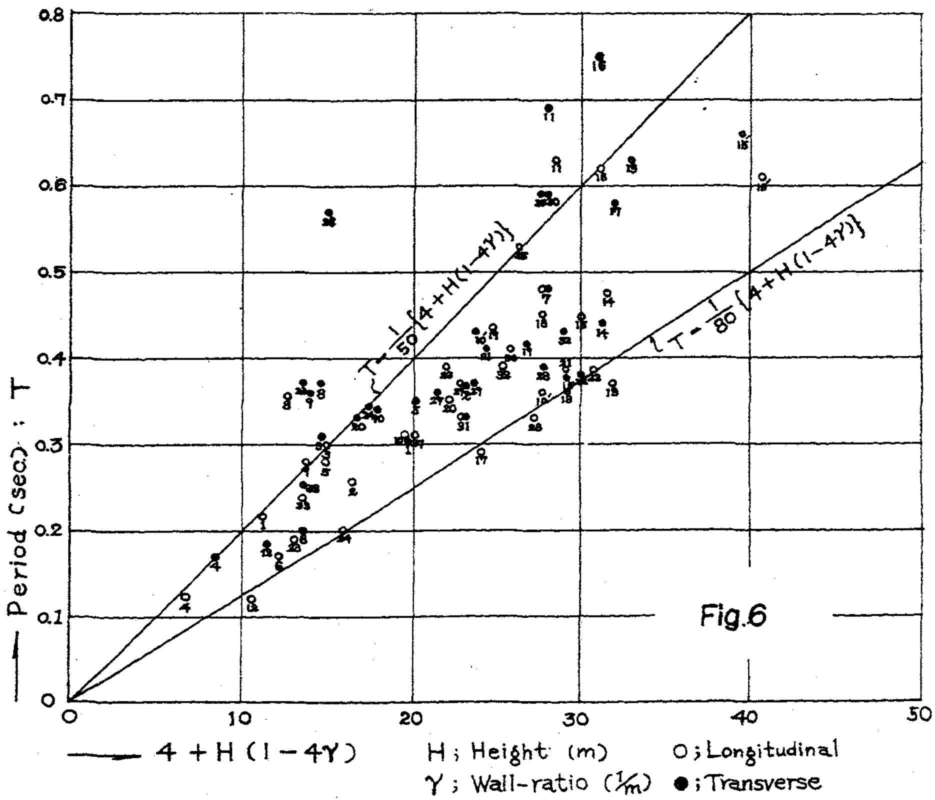
Period (sec.) ; T
$4 + H(1-4\gamma)$
$T = \frac{1}{50}\{4 + H(1-4\gamma)\}$
$T = \frac{1}{80}\{4 + H(1-4\gamma)\}$
H ; Height (m)
γ ; Wall-ratio (1/m)
o ; Longitudinal
● ; Transverse

Fig.6

Table 1

Dimension / No.	Height (m)	Length (m)	Width (m)	Period (sec.)		Wall-ratio ($\frac{1}{m}$)	
				Long.	Trans.	Long.	Trans.
1	19.97	16.19	12.96	0.217	0.302	0.160	0.0535
2	23.0	28.50	19.0	0.255	0.366	0.115	0.042
3	14.6	15.5	10.0	0.30	0.31	0.064	0.0693
4	7.0	6.0	4.0	0.122	0.17	0.155	0.0915
5	21.6	30.65	16.0	0.28	0.35	0.125	0.0642
6	10.4	54.4	12.0	0.17	0.20	0.0558	0.0227
7	26.0	47.6	37.0	0.48	0.48	0.0222	0.0193
8	10.9	59.4	10.4	0.355	0.37	0.0528	0.0096
9	11.1	37.2	31.6	0.28	0.36	0.0324	0.0274
10	21.35	17.0	10.6	0.31	0.34	0.0677	0.088
10'	30.4	17.0	10.6	0.36	0.43	0.0677	0.088
11	29.39	23.33	21.0	0.63	0.69	0.0418	0.0445
12	7.87	61.65	17.65	0.12	0.183	0.0424	0.0145
13	30.3	79.2	36.6	0.365	0.625	0.020	0.0128
14	29.39	48.0	51.5	0.475	0.44	0.015	0.0178
15	27.12	77.35	64.84	0.448	0.378	0.0116	0.019
15'	38.38	77.35	64.84	0.61	0.66	0.0116	0.019
16	31.0	39.6	25.2	0.62	0.75	0.0318	0.0255
17	29.7	25.3	7.2	0.29	0.58	0.0803	0.0135
18	29.8	35.3	24.8	0.45	0.36	0.0516	0.0386
19	26.3	29.9	24.0	0.435	0.455	0.053	0.0332
20	18.4	58.0	8.6	0.35	0.33	0.0035	0.0778
21	30.3	36.4	22.9	0.385	0.41	0.0432	0.0825
22	30.3	26.0	22.9	0.385	0.38	0.0297	0.0358

Dimension / No.	Height (m)	Length (m)	Width (m)	Period (sec.)		Wall-ratio ($\frac{1}{m}$)	
				Long.	Trans.	Long.	Trans.
23	10.3	51.0	11.4	0.19	0.37	0.035	0.024
24	15.0	83.0	29.7	0.20	0.345	0.052	0.0274
25	26.63	105.0	22.4	0.53	0.59	0.0402	0.0288
26	21.0	39.0	11.6	0.39	0.57	0.0037	0.120
27	18.6	36.0	23.6	0.31	0.36	0.0323	0.0173
28	26.6	70.0	33.2	0.33	0.39	0.0317	0.0264
29	24.3	22.1	18.8	0.37	0.37	0.0572	0.0483
30	28.0	90.0	24.0	0.41	0.59	0.055	0.0356
31	24.0	48.0	37.0	0.33	0.33	0.0538	0.0546
32	31.0	80.0	45.0	0.39	0.43	0.078	0.048
33	9.9	13.0	10.6	0.238	0.253	0.0104	0.011

VIBRATIONAL CHARACTERISTICS OF BUILDINGS

PART II

VIBRATIONAL CHARACTERISTICS OF REINFORCED CONCRETE BUILDINGS EXISTING IN JAPAN

by KYOJI NAKAGAWA*

INTRODUCTION

The author presented a report on large amplitude vibration tests conducted for various structures of actual size test buildings to the First World Conference held at Barkley in 1956 (1). As a continuation he presents here a summaritical report on vibrational characteristics of existing reinforced concrete buildings mainly obtained by minute amplitude vibration tests.

These results give us some data of linear characteristics of structures which are useful for the analyses of vibration phenomena in small earthquakes or initial stage of severe earthquakes, and further-more the results themselves are treated as criteria for judging the soundness of a reinforced concrete building.

EXPERIMENTAL RESULTS OF VIBRATIONAL CHARACTERISTICS OF EXISTING BUILDINGS BY MEANS OF VIBRATION TESTS

Vibration test of an existing building is usually carried out by the vibrator located on the upper floor. The eccentric moment of the vibrator used by the author was in the range of 169 to 3440 kgcm.

The names of buildings and their dimensions etc. are indicated in Table 1 (2), (3). The measurements of vibration were carried out mainly by mechanical vibrographs whose magnification factor was about 200 and self period of pendulum was 1.0 sec., but sometimes electrical instruments with high magnification factor were used.

The most typical resonance curve is, for an example, given in Fig. 1. The resonance characteristics of each building are obtained from these resonance curves measured at each floor. The natural periods thus obtained are written in Table 1 and fundamental vibrational modes are given in Figs. 2a, 2b and 2c. The arrow on each curve in Fig. 2 with numerical figure means the value of exciting force amplitude at resonance conditions in kg.

The vertical amplitudes on base and roof floors were measured simultaneously with horizontal amplitude of each floor for some buildings whose tests were recently carried out; the resulting whole modes are indicated in Figs. 3 and 4. The percentages of amplitudes due to swaying and rocking of a building as a whole to overall amplitude on the top floor are calculated as shown in Table 2.

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In Table 1 the fractions of critical damping of buildings calculated from the shape of resonance curves are given in the last column, and the values in parentheses are calculated by Dr. K. Kanai who conducted the experiements on almost buildings with the author (4).

In case of building No. 17, torsional vibration test by means of eccentric location of vibrator on its roof floor was also carried out. The Comparison of the resonance curves with those in case of central location of the vibrator can be seen in Fig. 5, and the torsional modes for a few example are indicated in Fig. 6. These are not so different shape in comparison with translative vibrational modes shown in Fig. 2.

The most important conclusive remarks from these experiment are that the swaying and rocking of a building as a whole due to the deformation of soils under its base are very dominant not only for small rigid structures but also even for rather large buildings, and furthermore the modes of vibration are slightly curved lines or can be seen as almost straight lines.

RELATION BETWEEN THE AMOUNT OF WALLS AND EARTHQUAKE DAMAGE

Our Building Code was written before the time when the modern buildings become to be constructed, so the Code expects, without any description, that the building has some moderate amount of walls. For the purpose to know the structural effect of walls in case of severe earthquake, the author investigated the following statistical investigations (5), (6).

On the suffered 77 reinforced concrete buildings in 1923 Kanto earthquake, relation between the average wall length per unit floor area above the ground floor and four damage classification are plotted in Figs. 7a and 7b. The critical amount of wall for rapid increase of earthquake damage seems to be some around 5 to 10 cm/m^2.

The same investigation were carried out also in case of 1948 Fukui earthquake, because in case of Kanto Earthquake, almost buildings were designed without any seismic provisions. The result shown in Fig. 8 indicates almost the same tendency as that in Fig. 7 even there were not sufficient data in this case.

NATURAL PERIOD MEASURED BY NEW SIMPLE INSTRUMENT

In 1953 Dr. K. Kanai and Mr. T. Tanaka completed the new instrument so called natural period meter by the fund of Construction Ministry. The instrument is a vibration counter of micro-tremor of a building. The frequency curve of various periods contained in the vibrogram of tremor measured in upper part of a building has its peak at the natural period of the building and has an almost symmetrical figure spreading down to the both sides of the peak. So the average period during a few minute vibration conincides with the natural period of the building.

The natural periods of 53 modern buildings constructed after 1953 were measured by the new instrument (5), (6). The frequency curve of wall amount of these buildings has its peak at 4 to 7 cm/m^2. It is obvious that the modern buildings have the tendency that the wall amount therein becomes less and less, even though the selection of these buildings were not at random. The relation between number of stories N and slenderness $H/\sqrt{D}$ for these buildings is represented by the following relation if the exceptionally slender buildings are excluded:

$$H/\sqrt{D} = (1.1 + 0.31N) \sim (2.2 + 0.7N), \quad H, D \text{ in meter}$$

If this relation be put in the "San Francisco formula" of natural period, the following formula will be obtained:

$$T = (0.1 + 0.038N) \sim (0.2 + 0.64N), \quad T \text{ in sec}$$

The range of this formula overlaps with "Taniguchi formula" which defines the relation of $T = 0.07N \sim 0.09N$, in other words, Japanes buildings have the slenderness such that the natural period obtained by these two formulas are not so different.

The measured natural periods indicated in Fig. 9 divided into three groups according the amounts of wall.

THE SAME SUPER STRUCTURES RESTING DIFFERENT SUB-SOIL CONDITIONS

Since 1951, comparative studies on the vibrational characteristics of the same apartment building structures resting on various sub-soil conditions were conducted by Dr. K. Kanai and the author (7). The buildings were almost as same as No. 19 shown in Table 1, i.e. 4 story building having 3 stairways and 24 homes. The natural periods and fractions of critical damping are indicated in Table 3 with the soil conditions which are divided only two kinds of soft and hard but are described in detail by micro-tremor characteristics of each site in Kanai's papers (8) Vibrational modes of these buildings are almost straight line as can be seen in Fig. 10. Because proper practice of foundation construction is adopted for each soil condition, the natural period of building is not usually longer for soft sub-soil, but it is clear that the damping is usually large for soft sub-soil, as it mainly depends on energy dissipation to the ground for such a relatively rigid building. If the relation between quasi-resonance factor 1/D which observed from actual earthquake by K. Kanai (8) and resonance factor 1/2h obtained from vibration test is plotted, a rather fine correlation can be seen as shown in Fig. 11.

CONCLUSION

From the investigation stated above, the author concludes that the Japanese reinforce concrete building has following characters:

(A) Because of the height limitation of 31 m, and design seismic coefficient is larger than in other countries, the Japanese reinforced concrete building has high rigidity. Further-more as they are almost resting on soft sub-soil such as alluvium, the percentage of amplitude due to swaying and rocking to overall amplitude is very predominant.

(B) Because the rocking motion of a building can be seen only in fundamental mode which has larger base moment than in higher modes, only the fundamental period becomes longer in comparison with base-fixed building. They vibrate almost in their fundamental modes and it is not necessary to consider the effect of higher modes. For this verification, vibrational mode of some buildings measured in micro-tremor vibration which is considered not so different from actual earthquake are indicated in Fig. 12.

(C) From the reasons described in (A), (B) and also from the fact that fundamental mode has only slight curvature or seems to be almost a straight line even in case of sway-rocking percentage is small, the vibration phenomena of the building will be treated simply as the motion of a rigid body on elastic foundation, if one uses the apparent coefficient of subgrade reaction in which the elastic deformation of building is amalgamated. In Fig. 13 the author indicates the comparison of principal mode and shearing force between these two cases. S-R-E means the elastic shear building whose rocking and swaying are 50 and 20 percent respectively in its fundamental mode, and S-R-R means the apparent rigid building whose fundamental period equals to S-R-E. It seems the difference in second mode is not so small, but the effect of it to earthquake response is so small that the summaritical error will not be considerable.

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NO.	BUILDING	NO. OF STORIES ABOVE G.L.	NO. OF STORIES BASE	PLAN AREA M²	TOTAL AREA M²	HT. H M	LOC OF VIB FL	ECC. MT. KGCM	DIRECT.	WIDTH D M	H/D	NATURAL PERIOD T SEC	FARACT. OF CRITICAL DAMPING h
1	SANKAIDO	5	1	775	1200	11.9	5	514	NS	39.2	8.17	0.240	
									EW	28.1	11.4	0.265	
2	SANNO A	4	1	123	1672	12.9	4.5	850	NS	8.0	20.7	0.280	0.097(0.15)
									EW	15.0	11.0	0.240	0.107(0.14)
3	SANNO B	4	1	1288	5530	12.7	4.5	1275	EW	40.3	4.02	0.191	0.143
									NS	24.0	6.76	0.210	(0.11)
4	SANNO C	4	1	1360	5020	12.7	6.0	1275	NS			0.180	
									EW	16.0	10.1	0.223	
5	MAMPEI	4	2	812	4250	17.8	R	1275	NS	21.3	14.9	0.416	0.079 (0.10)
									T			0.325	
									EW	44.7	7.23	0.400	0.112
6	NONOMIYA	7	1	482	4060	24.3	R	1275	NS	33.2	17.76	0.380	0.112 (0.10)
									EW	20.5	28.8	0.475	0.055 (0.05)
7	MANTETSU	6	1	1650	11581	25.4	8.5	1275	NS	21.3	30.3	0.500	0.0675
									EW	67.7	9.53	0.435	0.083
8	KANZAI	3	1	447	1642	14.7	3.0	514	EW	18.4	11.73	0.323	0.0275
									NS	28.8	7.51	0.310	0.0666
9	AKASHI	3	1	2660	9000	12.0	3.0	1275	EW(6)	42.0	3.41	0.203	0.0487
10	KURABO A	2	1			9.5	2.0	1275	NS	37.9	3.30	0.130	
11	KURABO B	3	1			11.2			NS			0.140	
12	KURABO			726	2350				EW	24.1		0.140	
13	OKURA	5	1	555	3330	21.8	R	1275	EW	33.7	14.1	0.217	0.0609
									T			0.224	
									NS	16.4	29.0	0.375	0.0376
14	CHUBU ELEC.	4	1	990	5170	16.70	R	514	NS	36.4	7.64	0.404	0.0635
									EW	27.4	102	0.366	0.07
									T			0.192	
15	CHUBU ELEC. AFTER REPAIR					16.70	R	1275	NS			0.275	
							R	503	NS				0.0566
16	CHUO KORON	6	1	202	1483	22.5	R	3440	EW			0.526	
									NS				
17	ITO CHU	8	2	913	9400	29.9	R	3440	NS			0.434	0.217
									T			0.287	
									EW			0.436	0.159
18	KUMEGAWA POINT	5	0	141	705	1428	4.5	637	NS	14.5	14.02	0.280	0.045
							5	169	NS			0.280	
19	KUMEGAWA FLAT	4	0	225	1105	11.34	4	637	NS	5.6	23.0		0.0355
									EW				
							4	169	NS				0.0690
20	BANCHO	4	0	301	903	10.88	4	169	NS			0.394	

TABLE I DIMENSIONS & VIBRATIONAL FEATURES OF TEST BUILDINGS

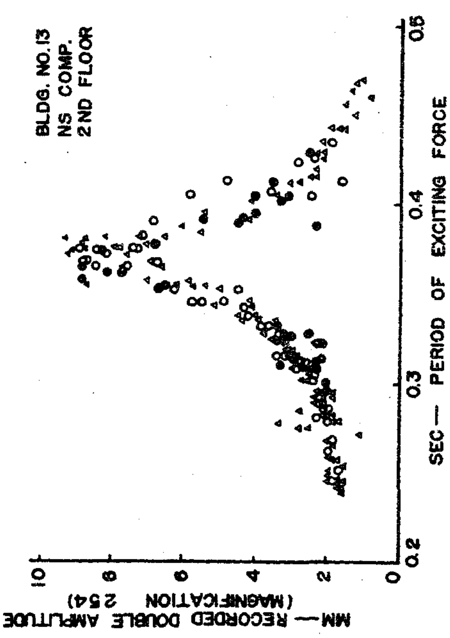


FIG. 1 TYPICAL RESONANCE CURVE

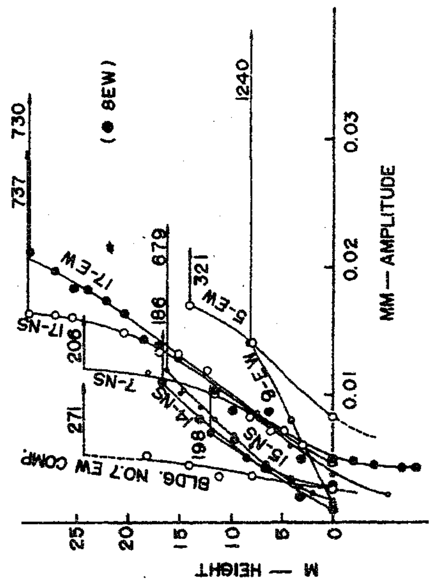


FIG. 2a VIBRATIONAL MODE

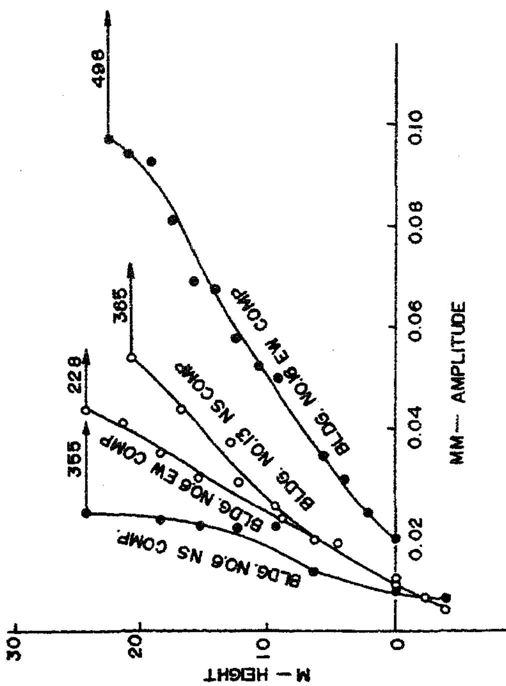


FIG. 2b VIBRATIONAL MODE

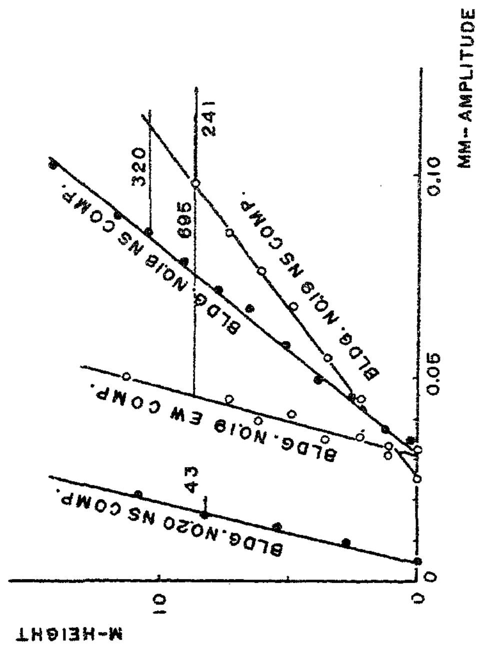


FIG. 2c VIBRATIONAL MODE

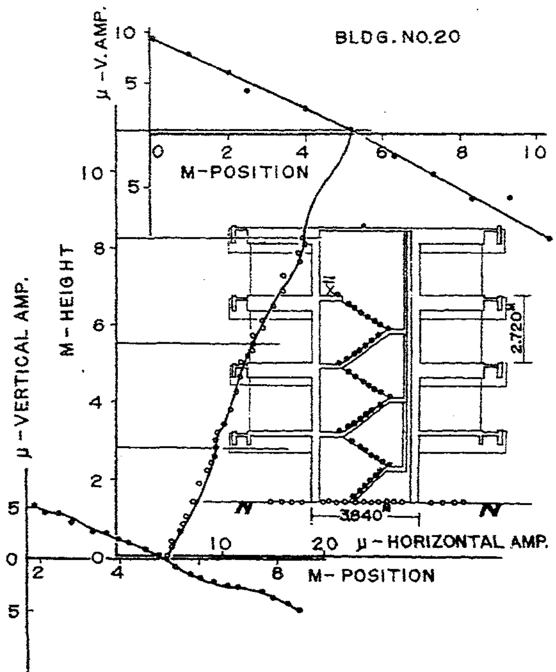


FIG. 3 VIBRATIONAL MODE OF BUILDING AS A WHOLE

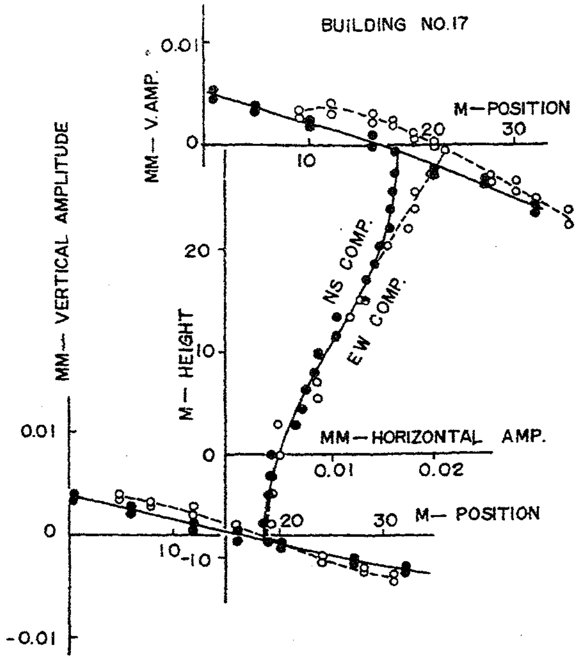


FIG. 4 VIBRATIONAL MODE OF BUILDING AS A WHOLE

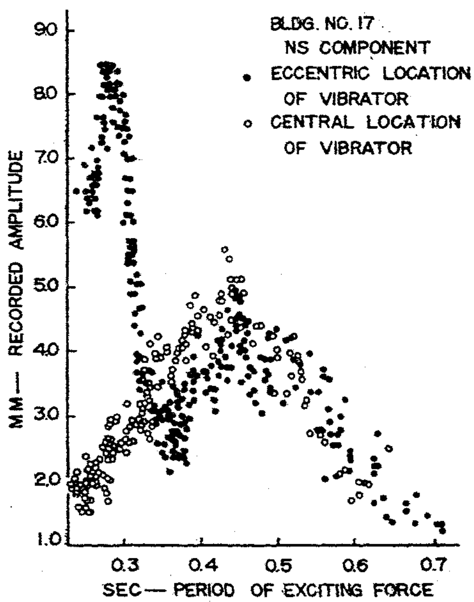


FIG. 5 COMPAPISON OF RESONANC CURVES

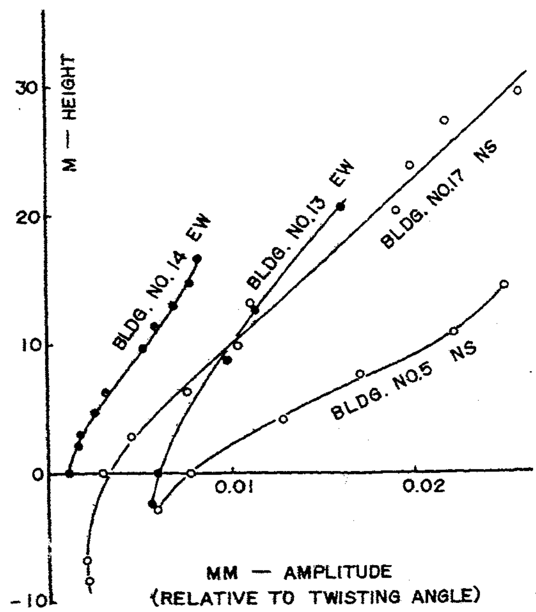
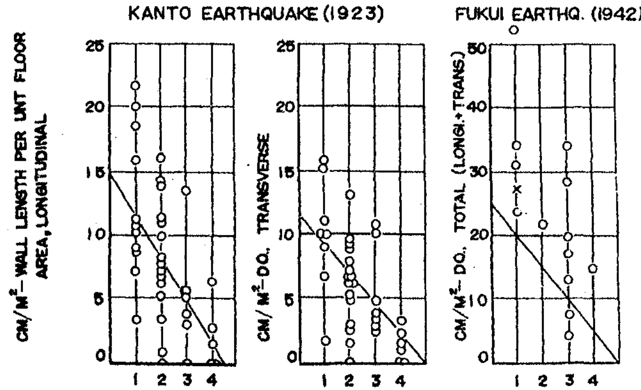


FIG.6 MODE OF TORSIONAL VIBRATION

BUILDING	AMPLITUDE MM/%				
	OVER ALL	SWAYING	ROCKING	S + R	ELASTIC
18 NS	0.0238 / 100	0.0076 / 32.0	0.0121 / 50.8	0.0197 / 82.8	0.0041 / 17.2
19 NS	0.0316 / 100	0.0061 / 19.4	0.0228 / 72.1	0.0289 / 91.5	0.0027 / 8.5
20 NS	0.0225 / 100	0.0047 / 20.9	0.0161 / 71.5	0.0208 / 92.4	0.0017 / 7.6
17 NS	0.0168 / 100	0.0032 / 19.1	0.0086 / 51.1	0.0118 / 70.2	0.0050 / 29.8
17 EW	0.0211 / 100	0.0036 / 17.0	0.0112 / 53.1	0.0148 / 70.1	0.0063 / 29.9

TABLE 2 PERCENTAGES OF AMPLITUDES DUE TO SWAYING & ROCKING



DAMAGE CLASSIFICATION, 1: NO, 2: SLIGHT, 3: HALF, 4: TOTALLY

FIG. 7a FIG. 7b FIG. 8

RELATION BETWEEN AVERAGE WALL AMOUNT & EARTHQ. DAMAGE

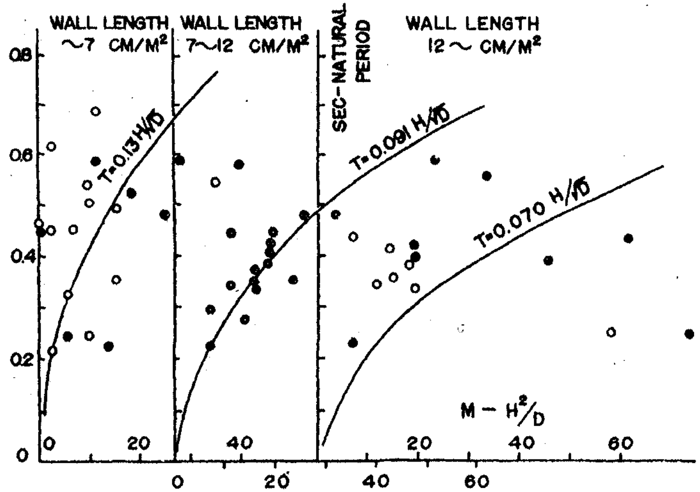


FIG. 9 NATURAL PERIOD MEASURED BY NATURL PERIOD METER

NO.	BLDG.	YEAR	SOIL.	T SEC	h	EXCIT. FORCE AMP. KG	AMP-LITUDE 1/100MM
21	TAKEMARU	1951	HARD	0.204	0.0926	643	4.7
22	TAKEMARU	DO.	HARD	0.225	0.0572	643	8.45
23	NISHINOMIYA	DO.	SOFT	0.268	0.115	643	7.8
24	KOMAGOME	1952	HARD	0.324	0.037	275	11.15
25	SENJYU	DO.	SOFT	0.324	0.0691	275	4.8
26	TOTSUKA	1953	HARD	0.332	0.0480	310	13.7
27	HONMURACHO	DO.	SOFT	0.231	0.0948	639	8.95
28	ISHIKAWACHO	DO.	SOFT	0.229	0.0504	707	12.6
29	GOTOKUJI	1954	HARD	0.346	0.0380	286	12.0
30	TAISHIDO	DO.	HARD	0.363	0.0360	260	13.25
31	KYODO	DO.	SOFT	0.385	0.0713	231	2.2

TABLE 3 VIBRATIONAL FEATURES OF THE SAME BLDG. STRUCTURES

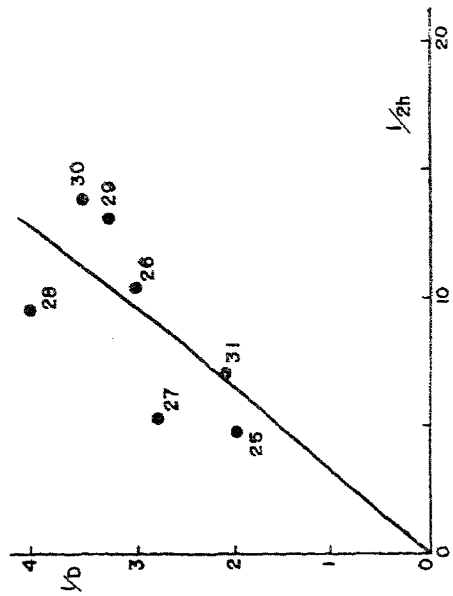


FIG. 11 RELATION BETWEEN 1/D & 1/2h

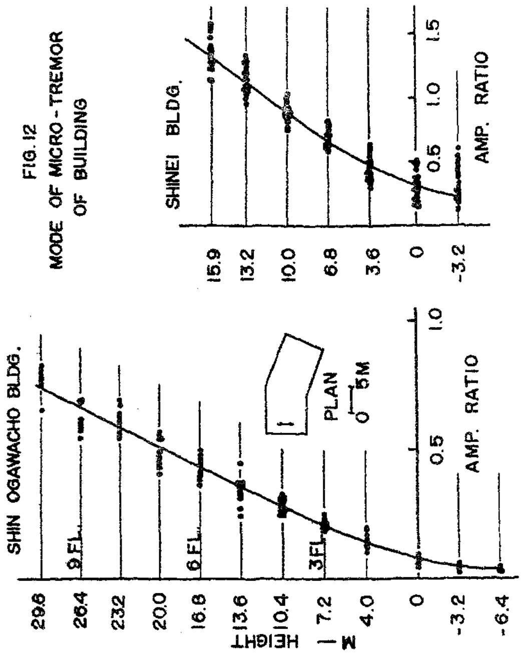


FIG. 12 MODE OF MICRO-TREMOR OF BUILDING

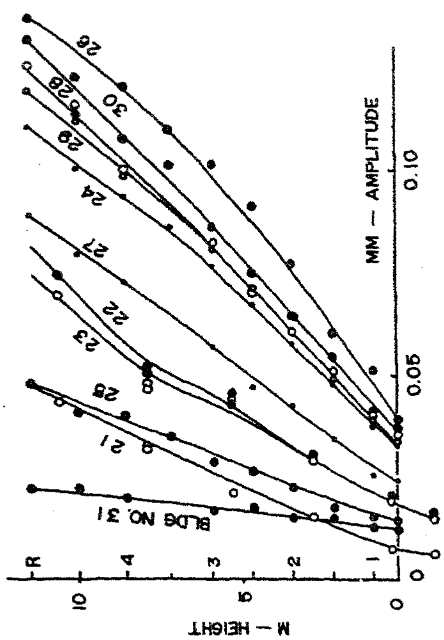


FIG. 10 VIBRATIONAL MODES OF THE SAME BUILDING STRUCTURES RESTING ON VARIOUS SUB-SOIL CONDITIONS

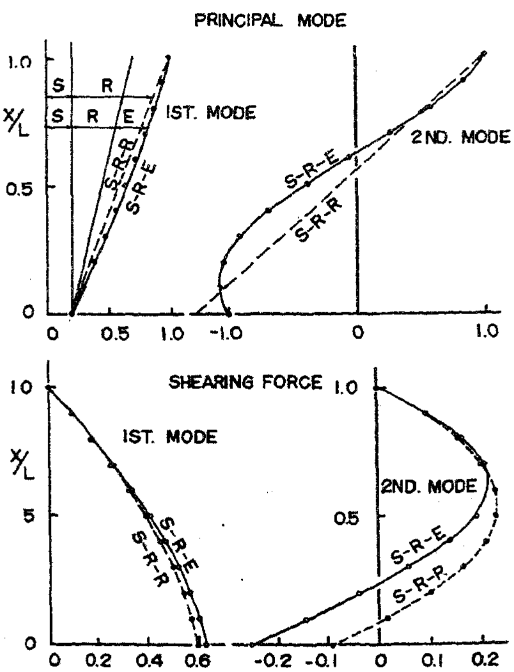


FIG.13 ANALYTICAL COMPARISON OF SWAY-ROCKING-RIGID BUILDING & SWAY-ROCKING-ELASTIC BUILDING