11. Energy Dissipation in Seismic Vibrations of Actual Buildings of Unlike Structure.

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(Read Oct. 15, 1935.—Received Dec. 20, 1935.)

1. In our last paper[1] we discussed the nature of decay in the seismic vibrations of three actual buildings of different stories, namely, 3–, 4–, and 5–, storied reinforced concrete structures, all owned by the Mitsubishi Co. The assumption in the problem regarding the condition of the floors was that the beams, etc., forming the members of the floors, are of such rigid construction that the inclination of columns at both their ends always remains vertical even in seismic vibrations[2]. It may appear improbable that exactly such conditions could exist in an actual building, and although the inclination under consideration in vibratory condition would differ somewhat, it would be very slight, compared with the undisturbed condition. It follows then that, although the nature of decay in seismic vibrations differs very little with conditions, the period of free oscillations of the building differs appreciably from that of the case in which the floor is extremely rigid.

With a view to ascertaining thoroughly the nature of the condition just mentioned, we investigated the vibration problem of three structures (one of which was studied in the previous case) under different assumptions regarding the condition of the structual connection between the vertical members and the floors. The two possible extreme conditions are a structure in which the floors are so infinitely rigid that they cannot bend and one in which the floors are so flexible that they are unable to impart any elastic resistance to horizontal oscillations. The buildings now under discussion were restricted to 3–storied reinforced concrete structures, the details of whose costruction were kindly supplied by the Mitsubishi Co., the owner of these buildings. One of these called the annexe of the Middle 7 th House

1) K. SEZAWA and K. KANAI, "Energy Dissipation in Seismic Vibrations of Actual Buildings", *Bull. Earthq. Res. Inst.*, **13** (1935), 925~941,

2) Appropriateness of such a condition in actual buildings will be seen from our paper, *Bull. Earthq. Res. Inst.*, **12** (1934), 804~822.

(Naka 7 gô-kan Bekkan) and constructed in March 1925, has no basement; that called the annexe of the middle 8 th House (Naka 8 gô-kan Bekkan) built on July 1926 has a basement; while the remaining one numbered Middle 13 th House (Naka 13 gô-kan) which was described in the last paper[3], completed in April 1915, also has a basement. The first two are let for restaurants (now "Tikuyôtei" and "Tokiwaya" respectively) and the last for business offices.

The methods of calculation were shown in a preceding paper[4]. As to the correction due to the effect of the basement, we calculated on the assumption mentioned in the last paper[5].

As will presently be seen, notwithstanding the exceedingly long periods of vibration of the structure having extremely flexible floors, it is still possible for the vibrational energy to dissipate into the ground at such a rate (though in somewhat less degree) that the amplitudes under resonance conditions are not very marked.

2. *Annexe of Middle 7th House* (Naka 7 gô-kan Bekkan).

The general plan of all the floors being virtually the same, we have given a sketch of only the first floor (Nikai), besides a photograph of a profile of the building. Neither construction plans of the different parts of the building nor details of our calculation will be given here owing to the difficulty of presenting them in simple form. The total areas and moments of inertia of cross section of the vertical members, and the total mass concentrated on every floor (the mass of the columns and walls are concentrated on the floor next above them) are shown in Table I. In this table, it is assumed that $E=2{\cdot}1.10^9\,\mathrm{kg}/m^2=2{\cdot}1.10^9.9{\cdot}8$ kg mass m/s^2/m^2, $\rho=2.10^3$ kg mass/m^3, $\mu=8.10^7$ kg mass/ms^2 (corresponding to $\sqrt{\mu/\rho}=200$ m/s). Here again, it is possible to put $I_1=I_2=\cdots\cdots=I$, $m_1=m_2=\cdots\cdots=m$, $l_1=l_2=\cdots\cdots=l$, where I, m, l are the mean values of the respective quantities.

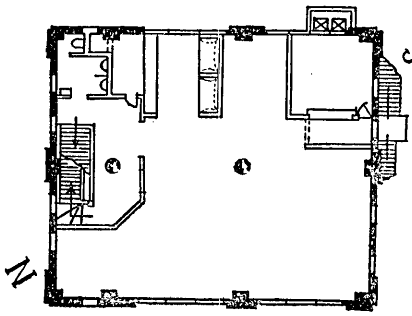

Fig. 2. Naka 7 gô-kan Bekkan. Scale 1/300.

3) K. SEZAWA and K. KANAI, *loc. cit.*, 1).

4) K. SEZAWA and K. KANAI, "Energy Dissipation in Seismic Vibrations of a Framed Structure", *Bull. Earthq. Res. Inst.*, **13** (1935) 698~714.

5) K. SEZAWA and K. KANAI, *ibid.*, 3).

Table I. Annexe of Middle 7th House.
(Naka 7 gô-kan Bekkan)

Floor			1 st	2 nd	Roof	Mean
Height of floor below		l (m)	4·397	3·484	3·789	3·890
Sect. area of columns and walls		a (m²)	9·500	6·648	5·840	7·329
Mt. inertia of columns and walls		I (m⁴)	0·807	0·396	0·390	0·531
Sect. area of columns		a (m²)	7·090	3·420	2·414	4·308
Mt. inertia of columns		I (m⁴)	0·6709	0·0811	0·0382	0·263
Beams	volume	(m³)	15·21	15·21	13·29	14·57
	weight	(kg)	$0{\cdot}365.10^5$	$0{\cdot}365.10^5$	$0{\cdot}319.10^5$	$0{\cdot}350.10^5$
Floors	volume	(m³)	9·83	9·83	9·14	9·60
	weight	(kg)	$0{\cdot}236.10^5$	$0{\cdot}236.10^5$	$0{\cdot}219.10^5$	$0{\cdot}230.10^5$
Columns	volume	(m³)	26·03	9·54	7·33	14·30
	weight	(kg)	$0{\cdot}625.10^5$	$0{\cdot}229.10^5$	$0{\cdot}176.10^5$	$0{\cdot}343.10^5$
Walls	volume	(m³)	10·61	11·59	13·20	11·80
	weight	(kg)	$0{\cdot}255.10^5$	$0{\cdot}278.10^5$	$0{\cdot}317.10^5$	$0{\cdot}283.10^5$
Total weight		(kg)	$1{\cdot}481.10^5$	$1{\cdot}108.10^5$	$1{\cdot}031.10^5$	$1{\cdot}207.10^5$
Area of floor		(m²)	145·5	145·5	145·5	145·5
$\frac{1}{a}\sqrt{\frac{mEI}{l^3\rho\mu}}$	Columns+walls		1·532[6)]	1·535[6)]	1·730[6)]	1·615
	Columns		1·878[6)]	1·355[6)]	1·318[6)]	1·935

The periods of the free oscillation in the case without dissipation are determined from

$$\sqrt{\gamma}=\frac{2\pi}{\mathrm{T}}\sqrt{\frac{ml^3}{EI}}\,,$$

where $\gamma=2{\cdot}3765$, $18{\cdot}6630$, $38{\cdot}9605$ for the case in which the floors and beams are extremely rigid, and $\gamma=0{\cdot}08555$, $3{\cdot}6676$, $26{\cdot}4776$ for the case in which the floors and beams are extremely flexible. The periods (in sec) of free oscillations of the movement in NS-direction in different conditions of loading and resistance of the vertical members are shown in Table II. This table indicates that, although the periods of free vibrations corresponding to the case of extremely rigid floors are somewhat less than those actually observed in similar structures, the periods of vibrations corresponding to the case of extremely flexible floors are too long to be accepted as being their near values.

6) The respective mean values for l and m were used.

Table II. Period in sec.

Floor condition		Rigid	Flexible
Vertical member =columns+walls	Without live load	0·104, 0·037, 0·0257	0·547, 0·0837, 0·0311
	With live load	0·125, 0·0446, 0·0309	0·659, 0·101, 0·0374
Vertical members =columns alone	Without live load	0·147, 0·0525, 0·0364	0·776, 0·119, 0·0441
	With live load	0·177, 0·0633, 0·0439	0·935, 0·143, 0·0531

The maximum values of bending moments $M=E_S I_S (d^2 y_S/dx_S^2)$ at each end of the columns corresponding to the maximum of the acceleration $(2\partial^2 u/\partial t^2)$ of the ground (on which no structurs is standing and in which $\lambda=14\mu$) were determined from the equations shown in the preceding paper[7], namely,

(i) The floors rigid, and $E_1=E_2=E_3$, $I_1=I_2=I_3$, $l_1=l_2=l_3$, $m_1=m_2=m_3$,

$$y_1=\frac{\left(7\sqrt{\frac{\lambda}{\mu}+2}-4\right)}{4\sqrt{P^2+Q^2}}\left[(\gamma^3-60\gamma^2+864\gamma-1728)\right.$$

$$\left.-\gamma(12-\gamma)(36-\gamma)\left\{3\left(\frac{x_1}{l}\right)^2+2\left(\frac{x_1}{l}\right)^3\right\}\right]\cos\left(pt-\tan^{-1}\frac{P}{Q}\right),$$

$$y_2=\frac{3\left(7\sqrt{\frac{\lambda}{\mu}+2}-4\right)}{\sqrt{P^2+Q^2}}\left[-(\gamma^2-36\gamma+144)\right.$$

$$\left.-\gamma(24-\gamma)\left\{3\left(\frac{x_2}{l}\right)^2+2\left(\frac{x_2}{l}\right)^3\right\}\right]\cos\left(pt-\tan^{-1}\frac{P}{Q}\right),$$

$$y_3=\frac{-36\left(7\sqrt{\frac{\lambda}{\mu}+2}-4\right)}{\sqrt{P^2+Q^2}}\left[(12-\gamma)\right.$$

$$\left.+\gamma\left\{3\left(\frac{x_3}{l}\right)^2+2\left(\frac{x_3}{l}\right)^3\right\}\right]\cos\left(pt-\tan^{-1}\frac{P}{Q}\right),$$

7) K. SEZAWA and K. KANAI, *Bull. Earthq. Res. Inst.*, 18 (1935), 698~714.

where

$$P=-9\sqrt{\frac{\lambda}{\mu}+2}\left(\frac{Ej^2}{\mu kl^3}\right)\gamma(12-\gamma)(36-\gamma),$$

$$Q=\left(\sqrt{\frac{\lambda}{\mu}+2}-1\right)(\gamma^3-60\gamma^2+864\gamma-1728),$$

(ii) The floors flexible, and $E_1=E_2=E_3$, $I_1=I_2=I_3$, $l_1=l_2=l_3$, $m_1=m_2=m_3$,

$$y_1=\frac{\left(7\sqrt{\frac{\lambda}{\mu}+2}-4\right)}{2\sqrt{P^2+Q^2}}\Big\{2(13\gamma^3-393\gamma^2+1296\gamma-108)$$

$$+3\gamma(-19\gamma^2+294\gamma-216)\left(\frac{x_1}{l}\right)^2$$

$$+\gamma(-31\gamma^2+372\gamma-108)\left(\frac{x_1}{l}\right)^3\Big\}\cos\left(pt-\tan^{-1}\frac{P}{Q}\right),$$

$$y_2=\frac{3\left(7\sqrt{\frac{\lambda}{\mu}+2}-4\right)}{2\sqrt{P^2+Q^2}}\Big\{4(-23\gamma^2+171\gamma-18)$$

$$+\gamma(7\gamma^2-216\gamma+324)\left(\frac{x_2}{l}\right)+6\gamma(2\gamma^2-13\gamma-18)\left(\frac{x_2}{l}\right)^2$$

$$+\gamma(5\gamma^2+10\gamma-24)\left(\frac{x_2}{l}\right)^3\Big\}\cos\left(pt-\tan^{-1}\frac{P}{Q}\right),$$

$$y_3=\frac{3\left(7\sqrt{\frac{\lambda}{\mu}+2}-4\right)}{2\sqrt{P^2+Q^2}}\Big[12(3\gamma^2+23\gamma-6)+2\gamma(-\gamma^2-15\gamma+234)\left(\frac{x_3}{l}\right)$$

$$-\gamma(\gamma^2+36\gamma+12)\left\{3\left(\frac{x_3}{l}\right)^2+\left(\frac{x_3}{l}\right)^3\right\}\Big]\cos\left(pt-\tan^{-1}\frac{P}{Q}\right),$$

where

$$P=9\sqrt{\frac{\lambda}{\mu}+2}\left(\frac{\mathrm{E}j^2}{\mu kl^3}\right)\gamma(-31\gamma^2+372\gamma-108),$$

$$\mathrm{Q}=4\left(\sqrt{\frac{\lambda}{\mu}+2}-1\right)(13\gamma^3-393\gamma^2+1296\gamma-108).$$

The calculated bending moments due to vibrations in NS-direction of the case in which, the floor being extremely rigid, the resistance

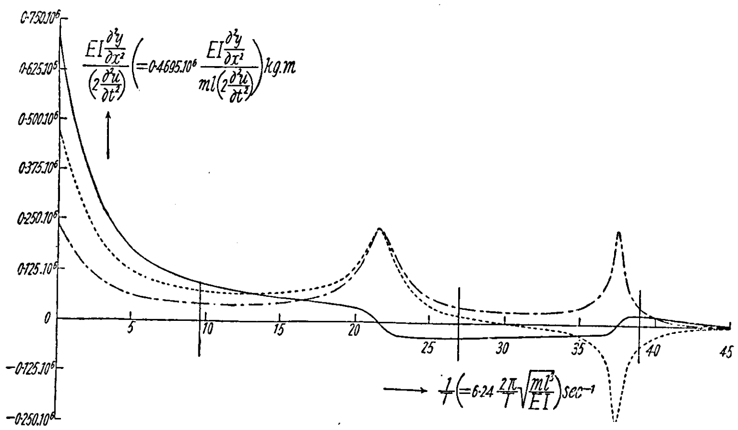

Fig. 3 Naka 7 gô-kan Bekkan. NS-movement (with extremely rigid floors), walls being attached to columns. Full, broken, and chain lines represent moments in columns of ground, first, and second floors respectively.

to bending of columns as well as walls, are shown in Fig. 3, whereas similar moments of the case in which, the floor being extremely flexible, the bending is resisted by the columns alone, is shown in Fig. 4. In both these cases, there is no live load on any floor, and it is to be

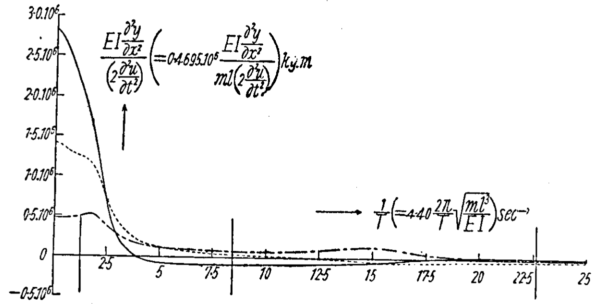

Fig. 4. Naka 7 gô-kan Bekkan. NS-movement (with extremely flexible floors), columns alone being resisting members. Full, broken, and chain lines represent moments in columns of ground, first, and second floors respectively.

remembered that the dimensions of E should be taken in kg mass m/s^2/m^2 and that of $2(\partial^2 u/\partial t^2)$ in m/s^2. The vertical strips in these figures represent cases corresponding to resonance conditions in the case without any dissipation.

These figures show that the bending moments of the columns, etc., even under resonance, are not very marked, provided the values of the acceleration at any period are the same. The moments under resonance in the case of extremely rigid floors are rather less than those at periods out of resonance, whereas the moments under resonance in the case of extremely flexible floors indicate a somewhat different aspect, namely, the moments under resonance are not necessarily smaller than those at periods out of resonance. Upon comparing these two cases it will be seen that making the floors as rigid as possible diminishes the bending moments especially the vertical members in periods approaching resonance. It used to be held that rigid connection of floors with the vertical members diminishes the moments in the case of zero frequency of vibrations, but the present investigation shows that such rigid connection is more advantageous in decreasing the moments under resonance conditions.

It is also a remarkable fact that the bending moments in the columns below the first floor, particularly in the case in which the floors are extremely rigid, become very small at frequencies beyond the first (principal) resonance condition.

If, in order to ascertain the extent to which stress is induced in the structure, we assume that the acceleration of the ground is $2(\partial^2 u/\partial t^2) = g/10$, then, in the former case, namely in the case in which the floors are extremely rigid,

maximum stress in wall $=15{\cdot}07$ kg/cm^2,

do. in column $=8{\cdot}24$ kg/cm^2,

while, in the latter case, namely in the case in which floors are extremely flexible,

maximum stress in column $=66{\cdot}4$ kg/cm^2.

These values are possible only in vibrations of zero frequency.

3. *Annexe* of *Middle 8th House* (Naka 8 gô-kan Bekkan).

For the same reason as in the preceding case, we show a photographic view as well as the general plan of the first floor. While no details of the numerical calculation will be given here, the total areas and moments of inertia of the vertical members and the total mass concentrated on every floor etc., are shown in Table III.

To arrive at the mean height of the floors, we added 1/3 of the part of the basement height to the actual mean between floor height from the ground to the second floors. The assumptions regarding the values of E, ρ, μ are the same as in the preceding case. The periods of free vibrations in NS-direction under different conditions of loading and resistance of the vertical members are shown in Table IV.

This table shows that the periods of vibrations corresponding to the extremely flexible floors are too long to be expected in the case of ordinary buildings.

The maximum bending moments due to vibrations in NS-direction were calculated by using the same equations as shown in the last section. Fig. 7 represents the case in which both columns and walls are taken as resisting members, the floors being assumed to be infinitely rigid, and Fig. 8 the case in which the columns

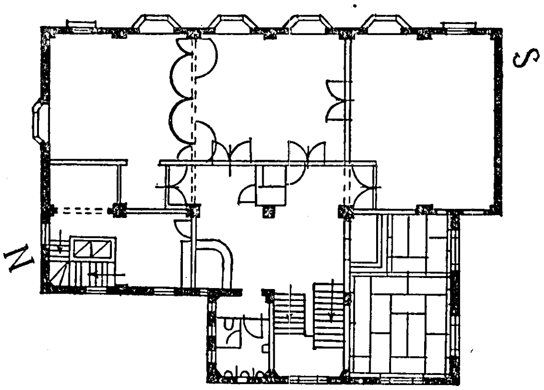

Fig. 6. Naka 8gô-kan Bekkan. Scale 1/300.

Table III. Annexe of Middle 8 th House.
(Naka 8 gô-kan Bekkan)

Floor			1 st	2 nd	Roof	Mean
Height of floor below		l (m)	3·635	3·332	3·795	3·587+0·252
Sect. area of columns and walls		a (m^2)	13·59	12·42	11·16	12·39
Mt. inertia of columns and walls		I (m^4)	1·171	1·354	1·312	1·279
Sect. area of columns		a (m^2)	8·992	7·301	5·412	7·235
Mt. inertia of columns		I (m^4)	0·245	0·178	0·105	0·176
Beams	volume	(m^3)	15·57	14·62	10·04	13·41
	weight	(kg)	0·374.10^5	0·351.10^5	0·241.10^5	0·322.10^5
Floors	volume	(m^3)	20·90	20·92	19·17	20·33
	weight	(kg)	0·502.10^5	0·503.10^5	0·460.10^5	0·488.10^5
Columns	volume	(m^3)	26·83	20·13	16·91	21·29
	weight	(kg)	0·645.10^5	0·483.10^5	0·406.10^5	0·511.10^5
Walls	volume	(m^3)	28·39	29·56	33·34	30·43
	weight	(kg)	0·653.10^5	0·680.10^5	0·768.10^5	0·700.10^5
Total weight		(kg)	2·174.15^5	2·017.10^5	1·875.10^5	2·022.10^5
Area of floor		(m^2)	234·3	234·3	234·3	234·3
$\frac{1}{a}\sqrt{\frac{mEI}{l^3\rho\mu}}$	Columns+walls		1·705[8)]	2·004[8)]	2·200[8)]	1·959
	Columns		1·184[8)]	1·240[8)]	1·285[8)]	1·245

8) The values of l and m were taken similarly as in the last example.

Table IV. Periods in sec.

Floor condition		Rigid	Flexible
Vertical members =columns+walls	Without live load	0·085, 0·0303, 0·021	0·448, 0·0685, 0·0255
	With live load	0·102, 0·0363, 0·0251	0·535, 0·0819, 0·0304
Vertical members =columns alone	Without live load	0·229, 0·0817, 0·0566	1·208, 0·185, 0·0686
	With live load	0·274, 0·0977, 0·0677	1·442, 0·221, 0·0820

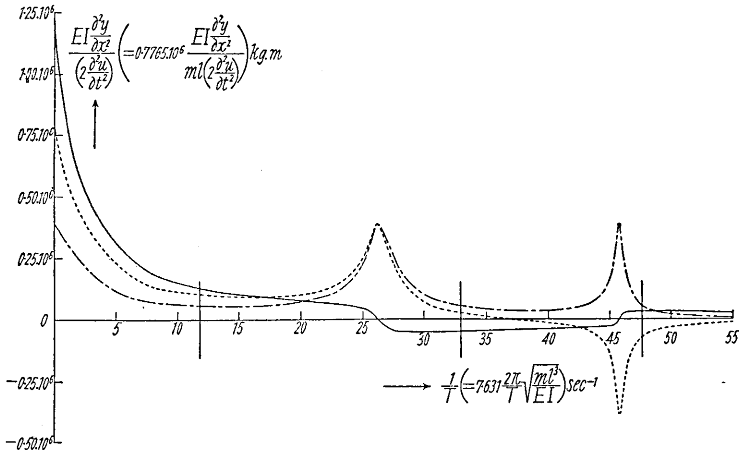

Fig. 7. Naka 8 gô-kan Bekkan. NS-movement (with extremely rigid floors), walls being attached to columns. Full, broken, and chain lines represent moments in columns of ground, first, and second floors respectively.

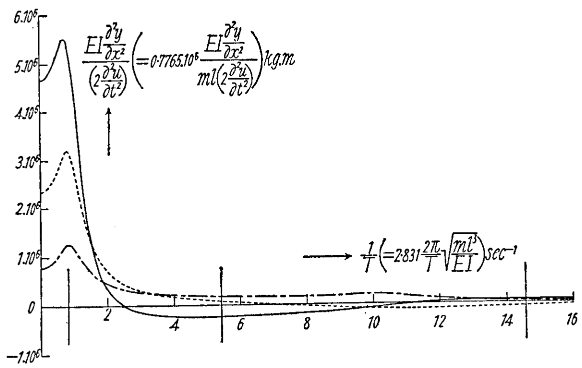

Fig. 8. Naka 8 gô-kan Bekkan. NS-movements (with extremely flexible floors), columns alone being resisting members. Full, broken, and chain lines represent moments in columns of ground, first, and second floors respectively.

alone resist vibrations, the floors being assumed to be extremely flexible. The difference in the feature of the bending moment curves for the present structure with the assumed difference in its structural conditions, namely the difference between the case in which the floor is extremely rigid and the one in which the floor is extremely flexible, is almost similar to that in the case of the Annexe of the Middle 7th House. But, in the present case, if the floors be extremely flexible and the columns alone be resisting members, the bending moment in the first (principal) resonance condition would be rather larger than the moment at zero frequency of vibrations. This arises from the fact that the difference in the resistance of vertical members of the two extreme cases in the present structure is more pronounced than in the structure of the last case.

In the present case, too, the bending moments in the columns below the first floor, particularly in the case in which the floors are extremely rigid, become very small at frequencies beyond the first resonance condition.

To ascertain the stress induced in vertical members, we put $2(\partial^2 u/\partial t^2) = g/10$, and obtain

(i) The fibre stress in the walls running in NS-direction for the case in which the floors are extremely rigid and the vibration frequency is zero $= 9{\cdot}31$ kg/cm²,

(ii) The fibre stress in the columns attached to the walls in the same case as (i) at zero frequency of vibrations $= 2{\cdot}756$ kg/cm²,

(iii) The fibre stress in the columns for the case in which the floors are extremely flexible and the vibration frequency is zero $= 80{\cdot}3$ kg/cm²,

(iv) The same stress in the first (principal) resonance condition $= 94{\cdot}5$ kg/cm².

4. *Middle 13th House* (Naka 13 gô-kan).

We gave in the last paper a special assumed case with regard to the structural condition of this building, namely, the condition wherein the floors are extremely rigid. We shall show in the present paper another condition, namely, the condition in which the floors are extremely flexible and the columns alone are taken as resisting members. The general plan being shown in the last paper,[9] only a photographic profile of the building is shown in Fig. 9. The result of the calculation with respect to the important elements is shown in Table V, although some of the elements were partly described in the last paper. The method

9) K. SEZAWA and K. KANAI, *loc. cit.*, 1).

of making the correction to be made for intermediate floor heights due to presence of the basement is the same as in the preceding case.

Table V. Middle 13th House (Naka 13 gô-kan).

Floor			Ground	1 st	2 nd	Rcof	Mean
Height of floor below		l(m)	2·58	3·82	3·52	3·52	3·62+0·32
Sect. area of columns and walls		a (m²)	33·83	31·34	31·34	22·66	28·45
Mt. inertia of columns and walls		I (m⁴)	11·23	2·54	2·54	2·54	2·54
Sect. area of columns		a (m²)	10·98	10·98	8·25	8·25	9·16
Mt. inertia of columns		I (m⁴)	0·1043	0·1043	0·0748	0·0748	0·0846
Beams	volume	(m³)	46·75	46·19	46·75	47·02	46·65
	weight	(kg)	$1{\cdot}123.10^5$	$1{\cdot}109.10^5$	$1{\cdot}123.10^5$	$1{\cdot}129.10^5$	$1{\cdot}120.10^5$
Foors	volume	(m³)	72·54	72·54	72·54	72·54	72·54
	weight	(kg)	$1{\cdot}741.10^5$	$1{\cdot}741.10^5$	$1{\cdot}741.10^5$	$1{\cdot}741.10^5$	$1{\cdot}741.10^5$
Columns	volume	(m³)	18·85	27·97	25·32	25·32	26·20
	weight	(kg)	$0{\cdot}453.10^5$	$0{\cdot}672.10^5$	$0{\cdot}608.10^5$	$0{\cdot}608.10^5$	$0{\cdot}629.10^5$
Walls	volume	(m³)	87·3	110·50	101·75	71·27	94·51
	weight	(kg)	$2{\cdot}095.10^5$	$2{\cdot}653.10^5$	$2{\cdot}442.10^5$	$1{\cdot}711.10^5$	$2{\cdot}269.10^5$
Total weight		(kg)	$5{\cdot}412.10^5$	$6{\cdot}106.10^5$	$5{\cdot}845.10^5$	$5{\cdot}120.10^5$	$5{\cdot}690.10^5$
Area of floor		(m²)	600	600	600	600	600
$\frac{1}{a}\sqrt{\frac{mEI}{l^3\rho\mu}}$	Columns+walls		1·670[10]	1·854[10]	1·854[10]	2·625[10]	1·939
	Columns		1·018[10]	1·018[10]	1·144[10]	1·144[10]	1·100

Under the same assumptions regarding the values of E, ρ, μ as in the preceding cases, the periods of free vibrations of the structure in NS-direction under different conditions of loading as well as of elastic resistance, are shown in Table VI. The abnormal largeness of the

Table VI. Periods in sec.

Floor condition		Rigid	Flexible
Vertical members =columns+walls	Without live load	0·110, 0·0375, 0·0260	0·554, 0·0846, 0·0314
	With live load	0·118, 0·0421, 0·0291	0·621, 0·095, 0·0353
Vertical members =columns alone	Without live load	0·577, 0·206, 0·142	3·035, 0·464, 0·173
	With live load	0·647, 0·231, 0·160	3·405, 0·521, 0·194

10) The values of l and m were taken similarly as in the last two cases.

periods arising from the conditions that the floors are extremely flexible and that the columns alone are resisting members, is particularly pronounced in the present case.

The maximum values of the bending moments at each end of the columns, when the floors are assumed extremely flexible and the columns alone are the resisting members, are shown in Fig. 10. In this case, the moments under resonance conditions are very marked.

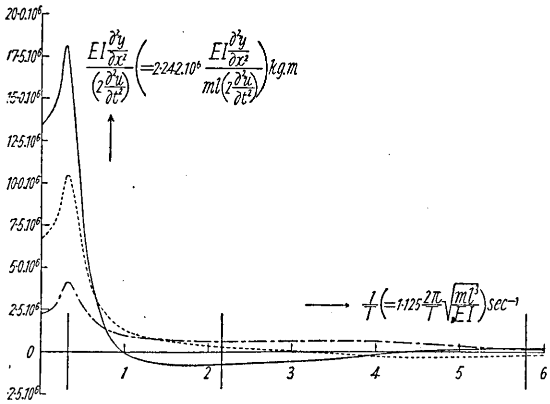

Fig. 10. Naka 13 gô-kan. NS-movement (with extremely flexible floors), columns alone being resisting members. Full, broken, and chain lines represent moments in columns of ground, first, and second floors respectively.

The bending stresses at zero frequency of vibrations, as well as under the first (principal) resonance condition, are shown below.

(i) Fibre stress in columns at zero frequency $=361\ \mathrm{kg/cm^2}$,

(ii) Same under resonance $=473\ \mathrm{kg/cm^2}$.

These stresses are induced in the columns below the first floor. It will be seen how large stresses would be induced in the columns were the condition of the problem so adjusted that the bending moments under resonance condition assume large values.

5. *Comparison of results and conclusion.*

We have now obtained the periods of free oscillations of three 3-storied structures as well as the bending moments induced in the vertical members of these structures due to a prescribed acceleration of the ground. The bending moment curves present somewhat different features according as the periods of the respective structures themselves differ. If (a) denote the case in which the floors are infinitely rigid and the columns and walls are resisting members, and (b) the case in which the floors are extremely flexible and the columns alone are resisting members, then the ratio of the period of fundamental free vibrations (principal period) in case (b) to that in case (a); the ratio of the bending moment under resonance to that at zero frequency of vibration for case (a); and the same ratio for case (b) are all shown in Table VI.

Table VII.

Name of structure	Fnnd. T in (b) / Fund. T in (a)	Mt. in (a) at reson. / Mt. in (a) at zero freq.			Mt. in (b) at reson. / Mt. in (b) at zero freq.		
		Ground col.	1st col.	2nd col.	Ground col.	1st col.	2nd col.
Annexe of Mid. 7. Ho.	7·47	0·133	0·160	0·180	0·741	0·875	1·050
Annexe of Mid. 8. Ho.	14·2	0·110	0·132	0·140	0·14	1·37	1·60
Mid. 13. Ho.	27·6	0·109	0·128	0·139	0·30	1·55	1·85

This table shows that, whereas the ratio of the bending moment under resonance to that at zero frequency in case (b) increases with increase in the ratio of the period in (b) to that in (a), the same ratio of the bending moments in (a) diminishes somewhat with increase in the period ratio under consideration. The fact that the ratio of the bending moments in question in case (b) is much greater than that in case (a), is too obvious to need further comment.

From the experimetal data due to observers who have studied other buildings, it appears that the actual vibration periods of the structures are somewhat longer than those calculated by assuming infinitely rigid floors,[11] but much shorter than those calculated by assuming extremely flexible floors. This suggests that the conditions of the floors and beams differ slightly from an extremely rigid one. Even if the floors were not extremely rigid the moment ratios already mentioned would not go outside the two limiting values, as shown in Table VII, namely cases (b) and (a) as upper and lower critical limits. It is thus possible to ascertain both of these critical limits in the nature of the dissipation of vibrational energy into the ground.

Even if the floor were assumed extremely flexible, which could never be the case in actual structures, the bending moments under resonance conditions would not differ much from those in the case of

11) It is obvious from our preceding paper (*loc. cit.*, 2)) that, if the floors be infinitely rigid, the vibration period of a structure is proportional to its height. From SAITA's study by means of results of observations, which were shown in papers, K. SUYEHIRO, *Proc. American Soc. Civ. Eng.*, **58** (1932), No. 4, as well as P. BYERLY, J. HESTER, & K. MARSHALL, *Bull. Seism. Soc. America*, **21** (1931), 268~278, the condition of such proportionality seems nearly satisfied in actual buildings.

zero frequency, both under the same acceleration of the ground, in which no structure is standing. This conclusion however seems to be valid only in the case of buildings of moderate height. In the case of high buildings, the bending moments under resonance conditions would not assume such small values were the floors extremely flexible; in other words, dissipation of the vibrational energy in such a condition would not be very large.

The most important fact related to condition (b) is that the peaks in the bending moment curves (namely, the peaks between successive resonance frequencies) are not so marked as those in the bending moment curves of case (a). Thus, the curves in case (b) gradually die away with increase in vibration frequency. Another fact to which attention is called is that, notwithstanding their largeness in zero frequency, the bending moments in the columns below the first floor, particularly in case (a), become very small at frequencies beyond the first resonance condition. The fact as observed, that damage to a building in a great earthquake occurs in the vertical members on the 1st or 2nd floor rather than in those on the ground floor, seems thus to ascertain that the floors is nearly extremely rigid. This, together with the remarks given in footnote 11) on the vibration periods of actual buildings, gives us the confirmation on the conditions of floors in actual buildings.

In conclusion, we take this opportunity of expressing our thanks to the members of the staff of the Mitsubishi Co , through whose courtesy valuable data were obtained for the present study, and also to the Council of the Foundation for the Promotion of Scientific and Industrial Research of Japan, for aid that greatly assisted our work.

Fig. 1. Naka 7 gô-kan Bekkan.

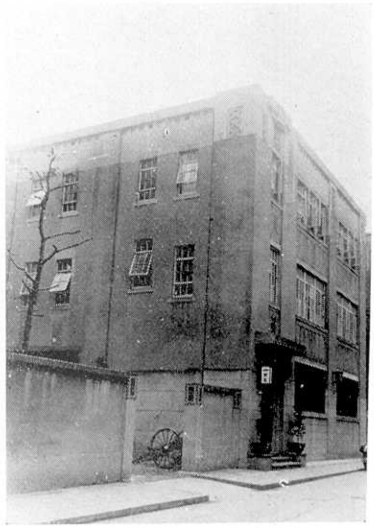

Fig. 2. Naka 8 gô-kan Bekkan.

Fig. 9. Naka 13 gô-kan.

11. 三菱仲7號館別館，仲8號館別館，仲13號館の種々の狀態に於ける震動勢力の逸散性

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前の論文で實在の3階4階5階の建物の震動逸散性を述べた場合には，床の固定狀態が極端に剛いといふ條件のもとに震動を取扱つたけれども，その固定狀態が多少違つてゐるときには固有振動週期は別として勢力逸散性に非常な影響があるかどうかを見る爲にこの研究を試みたものである．今回も三菱地所課の御好意によつて題名の如き3種の建物（何れも地上3階鐵筋コンクリート造）を選んで研究することができた．狀態の變化といふのは(a)床が極端に剛い場合と(b)床が極端に軟く從て水平動に對して何等の彈性的抵抗を與へぬ場合とである．尙附加へて構造壁が抵抗を及ぼさぬ場合をも考へて見たのである．

(a)の場合の固有週期は他の建物の觀測から推定されるものよりも稍短く算出されるけれども，(b)の場合の固有週期は如何に考へてもありそうもない程長くなるものである．さて，震動勢力が地中へ逸散するために，種々の地震動週期による建物の屈曲モーメントが如何なる値を取るかをしらべて見ると，(b)の場合と雖も3階建の場合には逸散性がそれ程少くはならぬやうに思はれる．しかし共振附近のモーメントが零の frequency の場合と同じ位かそれよりも稍大きい位のものである．

(a)の場合に1階にある柱や壁に起る屈曲モーメントは，zero frequency では最も大きいのに拘らず第1次共振以上の frequency では非常に小さくなる．(b)の場合には何れの床の柱と壁に起る屈曲モーメントも frequency の増加と共に漸次同じやうに小さくなるから，1階だけが屈曲モーメントが小さくなるとはいはれない．この事柄は地震動で4階や3階に被害が多い事に照して面白い現象と思はれ，同時に床を相當に剛い狀態にあるものと假定する方が實際に近いことがわかるのである．別に，齊田理學士が種々の建物の固有週期を比較して見たところが，その週期が大體に於て建物の高さに正比例することがわかつた．これは我々が彙報第12號に於て述べたやうに床が極端に剛い場合にのみ取り得る週期の割合であるから，齊田理學士の比較は結局建物の床が非常に剛いことを證明するのに他ならぬ譯である．即ち，逸散性を考へたときに地震動のための被害と，固有週期の比較とから同時に床の剛さの程度がわかつた譯である．

たとひ床の狀態が極端に剛くないとしても，共振の場合の振幅が，床の極端に軟い場合以上には出ないことがわかるのである．又，(b)の狀態に於ける固有週期と(a)の狀態に於ける固有週期との比が大きな建物では，(b)の共振に於ける屈曲モーメントと(b)の零の frequency に於ける屈曲モーメントとの比が大きな値を取る．然るにその場合(a)の共振に於ける屈曲モーメントと(a)の零の frequency に於ける屈曲モーメントとの比は小さな値を取るものである．

ここに注意すべきことは，只今の研究では3階の場合だけを比較したのであつて高層建築の場合はどうなるかわからない．高層建築の場合には床を極端に軟くすると，恐らく逸散性が非常に減じ，共振の振幅が比較的に大きくなるもののやうに思はれる．

この論文では床や壁の種々の場合を態と論じたけれども，實際の建物がその樣な種々の狀態を取る譯ではなく，寧ろ床が極端に剛い場合の狀態，即ち共振に於ける屈曲モーメントが非常に小さくなる場合が事實に最も近いのであつて，この論文の目的もそれを確めることにあつたのである．