THE VIBRATIONAL ANALYSIS OF A STEEL STRUCTURE
THE VIBRATIONAL TEST OF OHBAYASHI-GUMI BUILDING

By

S. Watanabe[1] and Y. Kida[2] and M. Higuchi[3]

SYNOPSIS

Ohbayashi-gumi building, a nine storied steel frame office building, has been constructed in Osaka, Japan. In order to investigate the dynamic properties of the structure during construction at 3 stages, as the frame was erected, concrete slabs were added, and curtain-wall and other elements were added, the natural periods of vibration, static deflection, and damping constants of each stage were measured. Measured periods and modes of each stage were compared with calculated values, and the damping properties of the structure were discussed.

INTRODUCTION

Some multi-storied buildings are under construction in Japan, and they are designed on the basis of highly developed dynamic analysis of structure. On the design of earthquake-resistant structures, there exist some assumptions which must be checked by tests or measurements and many studies are there on these subjects (1)-(7).

During the construction of a steel-framed 9 storied building, 3 stages were selected to measure the dynamic peoperties of the structure; 1st stage when the frame was erected, 2nd stage a week after the concrete slabs were added, and 3rd stage when curtain-walls and other elements were added. At each stage, natural periods, damping constants, and static deflections were measured, and these values were discussed together with calculated values in this paper. At 2nd stage a resonance curve was obtained.

Though the measured values depend on minute amplitude vibration tests, the dynamic properties in elstic range will be clarified.

It is difficult to estimate damping characteristics only with a damping constants of 1st mode. Damping constants of higher modes of internal damping increase in proportion to natural frequency of higher mode, but on the contrary those of external damping decrease in inverse propotion to natural frequency of higher mode. Therefore the real damping constants of higher modes are important as well as the damping properties in nonlinear range. In this paper, the damping constants of higher modes were given from a resonance curve and discussed together with those assumed in theoretical analysis. The curves given by using the analogue computer of B. R. I. were compared with those measured and discussed.

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1. DESCRIPTION OF BUILDING

Although this building is a smaller one as shown in Fig. 1-1 and 1-3, it was built experimentally to signify a tall building and the techniques used in its structural design or constructions would be applicable to taller buildings. Since the entire main structure is of steel, the dynamic behavior is similar to that of a taller building. The floor plan together with the structure is simple, so it was thought to be easier to analyse its dynamic properties and this building is a typical one for the experiments to study the problems on earthquake engineering.

The girders, beams and columns above ground are H-sectional rolled steel covered by asbestos for fire-proofing. Some of the joints of steel frames were welded in the factory and other joints were constructed in field as frictional joints with high tensile strength steel bolts. To reduce the weight of the building, light weight concrete is used for the floor slabs. The exterior walls on the north and south sides are aluminium curtain-walls, but the walls on the east and west sides together with the interior walls are made of sypolex. The connections between the walls and the frames were so constructed as to be safe against about 1 inch deformation which may be caused by earthquakes.

The structure under ground is of reinforced concrete. The foundations are supported by the short piles of reinforced concrete on the gravel diluvium called Tenma Layer that underlies around Osaka City.

The main structure is three rigid frames in the east-west direction and truss type frames in the north-south direction which are reinforced by the bracings to resist lateral forces. The floor slabs and the girders or beams are connected with the shear connectors of steel bars to keep stiffness in the horizontal plane.

The structural calculation was made according to the Japanese Building Code and was examined through the earthquake responces by the electric analogue computer. The records finally adopted in this computation was the type of Taft Earthquake and the maximum accerelation assumed was 330 gal. As the results, the maximum stress of every member caused by the earthquake forces, was in the elastic range of steel members. So the structure was expected to be safe against stronger lateral forces by the ductility of steel members.

Vibrational tests were made mainly in the east-west direction, because rigid frames are more general than truss type in this kind of the building.

2. OUTLINE OF TESTS

The vibrational tests on this building were made under the following three stages.

1st Stage — After the steel frames were constructed
(The concrete of 1st and the under ground floors was cast)

2nd Stage — After the concrete of all floors was cast

3rd Stage — After completion of the construction

To measure vibration of structure, the moving coil type pick-ups having the natural period 0.5 sec and 1.0 sec were used in the tests. The records obtained by these instruments are considered approximately as the velocities of the vibration in some frequency ranges, and the magnification of the record was from 30 to 200 in respect to the displacement. In order to get the statical deflections of the building, the relative displacements to the adjoining building were measured by the dial-gages that had the accuracy 0.01 mm.

Measuremnts were made at 5 station on 2nd, 4th, 6th, 8th floor and the roof, and this building was assumed as 5 mass system model in the analysis. The measured directions of the horizontal vibrations are shown in Fig.2-1. The means causing the vibrations and the direction are as follows.

In every stage of the experiments, R - FL was pulled by the lateral forces 1^t, 2^t and 3^t, relaxed suddenly and the free vibration were caused from the initial displacements. Vibrational test was limited to the east-west direction alone.

In the 2nd stage, a vibrator was equipped on the top of the column B-2 to generate forced vibrations. The vibrator had the revolutions from 180 to 900 r.p.m. and the maximum output force was 5^t. The vibrations caused were in the east-west and the north-south direction, but in this paper, discussion is confined to the E-W direction alone.

3. DESCRIPTION OF VIBRATION RECORDINGS

Measured curves are summarized in Fig.3-7 - Fig.3-9. Summary of natural periods during construction is shown in Table.3-1. Natural periods of the vibrations in the N-S direction were obtained from minute free vibrations. Using the equation $T = \alpha \cdot H / \sqrt{D}$ proposed by the Joint Commitee of San Francisco, the value α is calculated by the period of 1st mode at 3rd stage, which is 0.089 in the E-W and 0.054 in the N-Sdirection.

Period ratios of higher modes to fundamental period were compared with calculated shear period ratios at 2nd stage, and they are shown in Table.3-2. The torsional vibrations were not observed during 3 stages of construction not only in free vibrations but also in forced vibrations

Static deflections of 8 FL at 3 stages are shown in Table.3-3. At 1st and 2nd stages, the added lateral forces were 3 tons, and at 3rd stage the added lateral force was 2.6 tons.

The structural rigidities at 1st and 3rd stage were calculated in the method where the shear stress given by lateral force is divided by the relative displacement of that floor to the upper and lower floor. They were shown in Table.3-4, Table.3-5 and in Fig.3-1.

In the 1st stage, the difference of the statical deflection between the central frame and the framed at the both ends of the building was only 2.1%, so it was thought that the deformations in the horizontal plane could be taken into no account in case of vibration of the steel frames without walls.

The fractions of critical damping of fundamental mode during construction are shown in Table.3-6, and damping coefficients are shown in Fig.3-2, and Fig.3-3.

The resonance curve at 2nd stage are shown in Fig.3-4, and modes at each resonance periods are shown in Fig.3-5. From the resonance curve, fractions of critical damping of higher modes at 2nd stage were calculated by using modal analysis. These values are shown in Table.3-7, in comparison with those calculated from the system with internal damping and with those calculated from the system with external damping each by theoretical analysis.

The modes assumed in calculation of fractions of critical damping of higher modes, and those obtained from resonance curve are shown in Fig.3-6.

4. METHOD OF THEORETICAL ANALYSIS

To examine the results above, we computed the vibration of the building in the following two cases under the same conditions as these vibrational tests.

We assumed the 5 mass system model in Fig.4-1 to show the vibrational characteristics of the building. At first, the mass and rigidity distributions in Table.4-1 were estimated. The free vibrations of the 5 mass system model were computed by the electric analogue computer from the same initial conditions as those of the vibrational tests. As the damping is an anbiguous factor for structures, we assumed two kinds of damping, that is, the internal and external damping and each case was computed independently.

The fundamental equations are as follows.

(1) The case wherein internal damping is assumed

$$m_i \ddot{x}_i - C_{Ii+1}(\dot{x}_{i+1}-\dot{x}_i) + C_{Ii}(\dot{x}_i-\dot{x}_{i-1}) - k_{i+1}(x_{i+1}-x_i) + k_i(x_i-x_{i-1}) = -m_i \ddot{x}_0 \qquad (4\text{-}1)$$

(2) The case wherein external damping is assumed

$$m_i \ddot{x}_i + C_{Ei}\dot{x}_i - k_{i+1}(x_{i+1}-x_i) + k_i(x_i-x_{i-1}) = -m_i \ddot{x}_0 \qquad (4\text{-}2)$$

CIi and CEi in the equation (4-1), (4-2) are

$$C_{Ii} = \frac{2h k_i}{\nu} \qquad (4\text{-}3)$$

$$C_{Ei} = 2h\nu m_i \qquad (4\text{-}4)$$

The damping codfficients with same fractions of critical damping are shown in Table.4-2.

The shear deformations here considered include the bending deformations and the displacements due to the rotations of the foundations. The spring constants in the 3rd stage are the values measured directly by the test, for it was difficult to estimate the stiffness of the wall. The natural periods and the modes of vibrations computed from the constants above, are shown in Fig.4-2.

The damping factors were obtained by the fraction of critical damping of funamental mode obtained by vibrational tests.

The theoretical curves of the 5 mass system model are in Fig.4-3-Fig.4-8.

For the purpose of reference, the responces of the model in the 2nd stage to the disturbances of the ground moving were computed by the two equations (4-1) and (4-2). They are in Fig.4-9 and 4-10.

Fig.4-9. The responce of the model with internal damping to the ground moving.

Fig.4-10. The responce of the model with external damping to the ground moving.

5. DISCUSSION OF RESULTS

As for the periods of a structure during construction, detailed data were given by John A. Blume(7). The same characteristics of a history of change in periods is also adequate in this structure. On the step from 1st stage to 2nd stage, the natural period takes a large value with increase of the mass gravity. On the step from 2nd stage to 3rd stage, the natural period decreases with the effect of curtain-wall rigidity.

In table.3-2, period ratios of higher modes to the fundamental period are shown. The natural periods of 4th mode and 5th mode at 2nd stage have large period ratio to the fundamental period.

There can be seen great difference in structural rigidity during construction, ie at 2nd stage, the rigidity increases in comparison with 1st stage, and it is considered that it depends on effects of both slabs and beams, which is adequate in theoretical analysis, the rigidity at 3rd stage increases much greater than the other stages under effect of the curtain-wall rigidity. The rigidity calculated from static deflection at 3rd stage shows that the rigidity decreases as the lateral force becomes large even in small deflection. The decrease in the initial rigidity at 3rd stage is considered to be caused by the decrease in the curtain-wall rigidity which shows nonlinear behavior on load deflection curve, so it is difficult to determine an equivalent linear rigidity in elastic range such measured values.

From the fractions of critical damping of 1st mode during construction shown in Table.3-6, it may be evident that the fraction of critical damping of structural frame is 2.8 - 3.0%, and it becomes nearly 2 times as much by the effect of the curtain-wall.

From the resonance curve at 2nd stages shown in Fig.3-4, the fractions of critical damping of higher modes were obtained using modal analysis and they are shown in Table.3-6. On the assumption of principal vibration of fundamental mode, the damping constant in many cases is determined in proportion to relative velocity of that floor to the upper and the lower floor, which is due to internal damping, and the fraction of critical damping of higher modes in the assumption of 3% fraction of critical damping of fundamental mode are shown in Table.3-6. It is evident from Table.3-6 that fractions of critical damping in internal damping only are too much larger than those of measured values. So it is apparent that, using the assumption of the usual theoretical analysis, the effect of higher modes in deflection curve is much under-estimated. Especially the amplitudes of the peak on resonance curve at 4th and 5th modes are much larger than that of 3rd mode, and these make it clear that the damping characteristics of a real structure can not be considered only with internal damping. On the contrary, fractions of critical damping in external damping, in which damping constant is in proportion to relative velocity of that floor to foundation, are shown also in Table.3-6. Fractions of critical damping of higher modes are small in comparison with those of measured values.

In comparison with the results from the analogue computer to the measured curves Fig.3-7 - Fig.3-9, on Fig.4-3 - Fig.4-5, which are the results with internal damping only, the effect of higher mode can not be observed as seen on the measured curves; on the contrary the effect of higher mode is distinguished on Fig.4-6 - Fig.4-8, which are the results with external damping. From these two cases, it seems valid that the real damping characteristics of a structure must be considered with both internal and external damping. Internal damping depends on the viscosity considered in the structure, and external damping, dissipation of energy into ground through the foundation and so on.

Dissipation of energy into ground is discussed in the paper by Dr. K. Sezawa, and Dr. K. Kanai(8)(9)(10). But many problems are left on these damping properties.

Vibration test described in this paper was limited to small deflection, so the tests in large deflection will be further desired.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to Dr. T. Hisada, Dr. K. Nakagawa and Dr. M. Izumi, of Building Research Institute for their guidance and helpful suggestions, and to Dr. H. Umemura, Prof. of Tokyo Univ., member of A. I. J. for his useful advices. The analogue computer used in this paper is the Hitachi ALM-502T analogue computer installed in Building Research Institute.

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Table 3-1. Summary of Measured Periods.

Stage No.	E-W Direction 1st Mode	2nd Mode	3rd Mode	N-S Direction 1st Mode
1.	0.67			0.35
2	1.00	0.33	0.195	0.50
3	0.75-0.8			0.47-0.48

Table 3-2. Period Ratio of 2nd Stage.

Mode	Period Ratio	Shear Period Ratio
1	1.00	1.00
2	3.00	2.68
3	6.85	4.21

Table 3-3.
Static Deflection.

Stage No.	Defl. (μ)
1	2,570
2	1,830
3	770

Table 3-4.
Static Rigidity at 1st Stage.

Floor	Stiffness ton/cm
8	31.2
6	39.0
4	47.2
2	146.0

Table 3-5. Static Rigidity at 3rd Stage.

Floor	Lateral Force (ton) 1.0	2.0	2.6	
8	77.0	75.5	69.4	
6	167.0	138.0	124.0	
4	156.0	210.0	150.0	
2	1,670.0	1,340.0	1,130.0	ton/cm

Table 3-6. Summary of Fractions of Critical Damping.

1st Stage	h = 0.0256 — 0.0308	
2nd Stage	h = 0.0246 — 0.0286	
3rd Stage	h = 0.0236 — 0.0369	for small deflection
	= 0.0483 — 0.063	for large deflection

Table 3-7. Fractions of C.D. of Higher Modes at 2nd Stage.

Mode	h (Measured)	Internal Damping	External Damping
1	0.028-0.03	0.03	0.03
2	0.0385	0.091	0.01
3	0.077	0.154	0.006
4	———	0.272	0.003
5	———	0.334	0.0027

Table 4-2. Damping Coefficients with Same Fraction of C.D.

Stage	C.I	C.E	$\frac{C.E}{C.I}$
1	0.193	0.0242	1/8.0
2	0.449	0.0523	1/8.6
3	1.21	0.197	1/6.2

C.I; Internal Damping Coeff.
C.E; External Damping Coeff.

Table 4-1. Calculated Weights and Stiffnesses.

Floor	1st Stage mi	1st Stage ki	2nd Stage mi	2nd Stage ki	3rd Stage mi	3rd Stage ki
R	0.037	29.4	0.171	38.3	0.280	61.0
8	0.039	34.3	0.150	43.5	0.230	69.4
6	0.040	40.1	0.140	50.8	0.210	214.0
4	0.042	49.7	0.156	63.0	0.235	150.0
2	0.177	118.0	0.267	130.0	0.380	1,130.0

mi; Mass $ton.sec^2/cm$
ki; Stiffness ton/cm

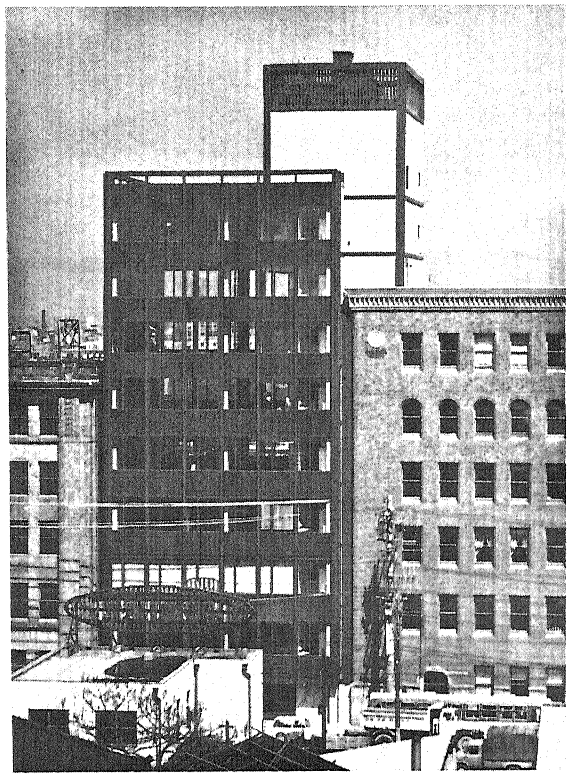


Fig 1-1. South Elevation of Ohbayashi-gumi Building

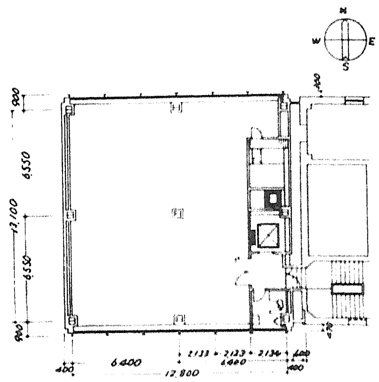


Fig 1-3. Schematic Plan.

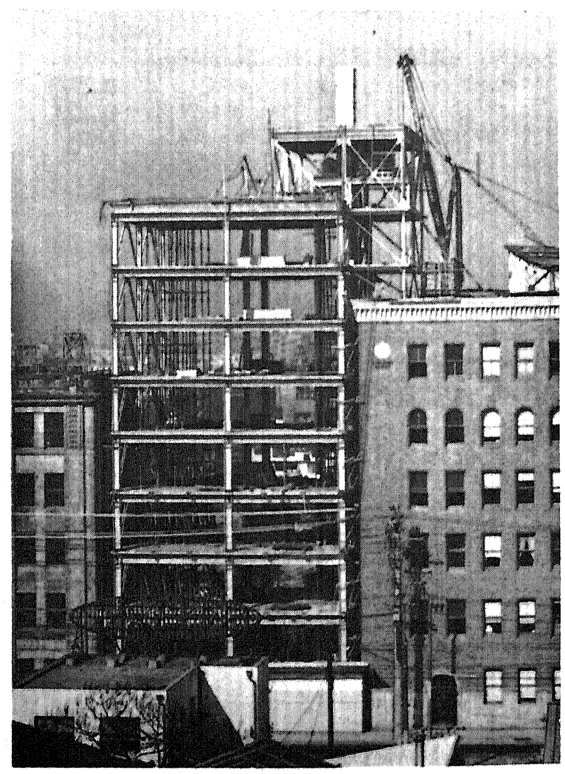


Fig 1-2. View of Building Construction.

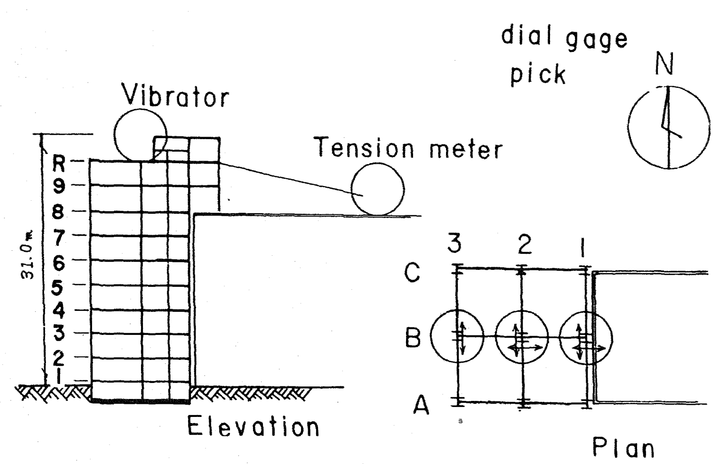


Fig. 2 - 1

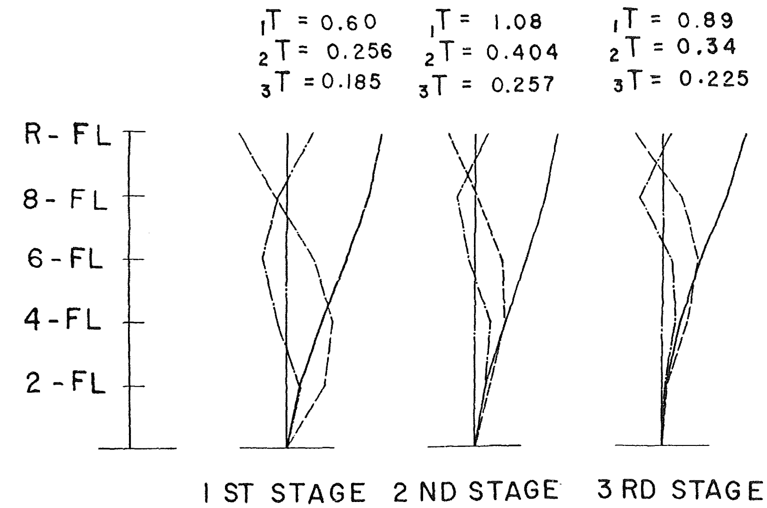
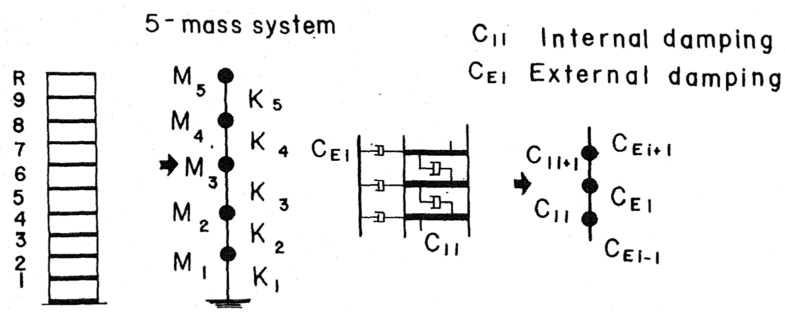


Fig. 4 - 2



Fig'. 4 - 1

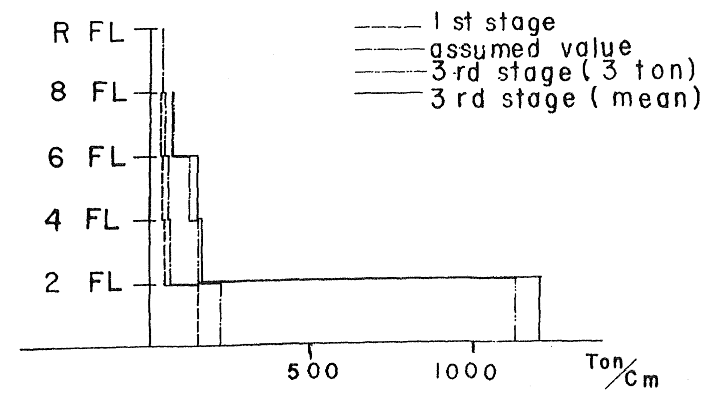


Fig. 3-1 Static Rigidity at Each Stage

Fig 3-4 Resonance Curve at 2nd stage

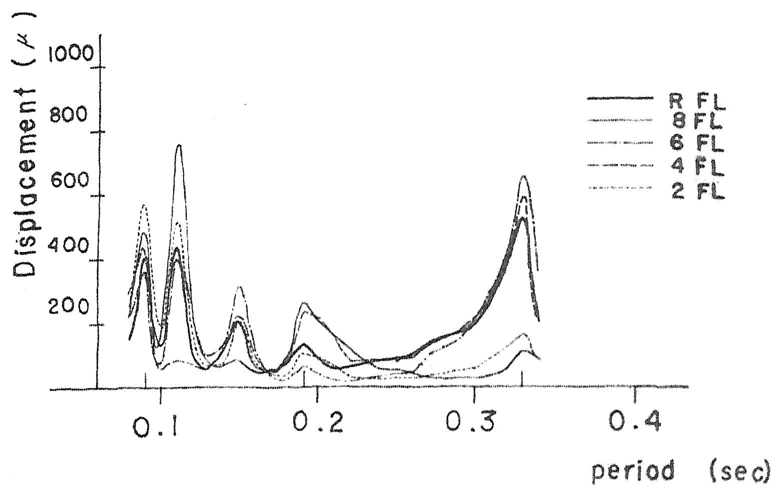


Fig.3-2

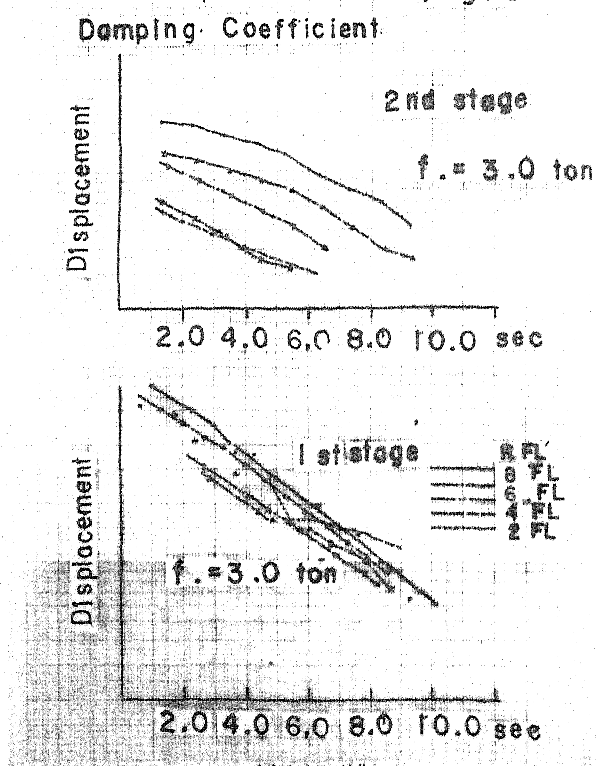


Fig.3-3

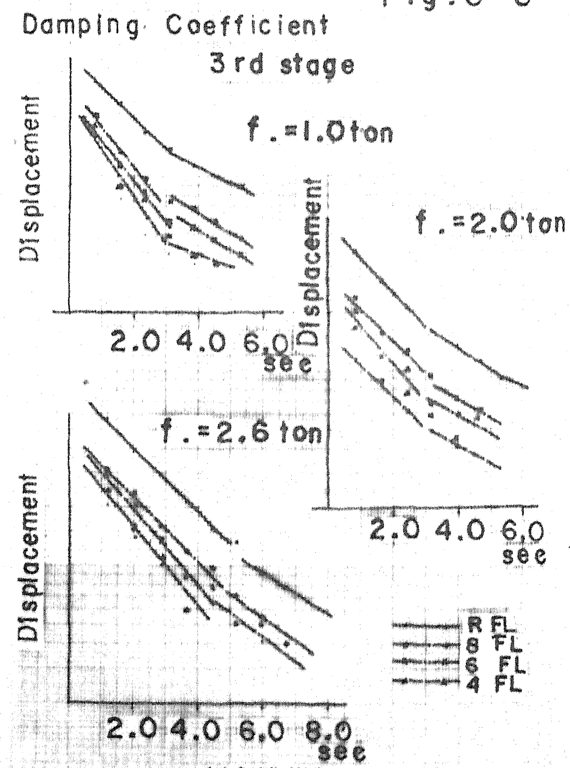
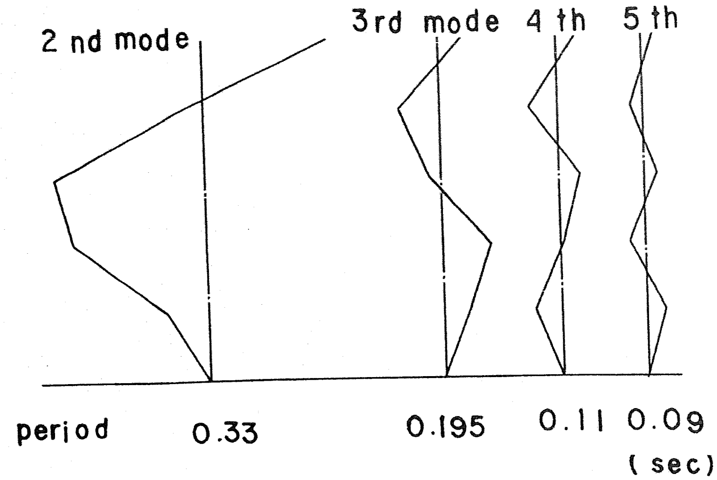


Fig 3-5 Messured mode to constant acceleration



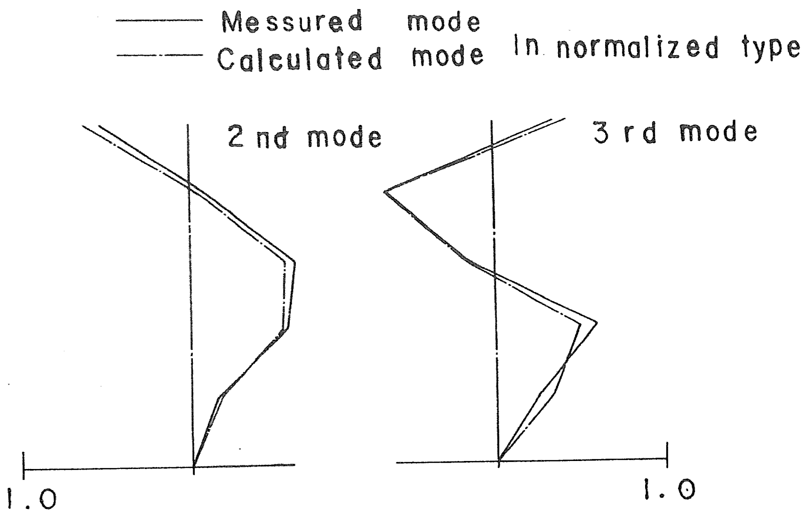


Fig. 3 - 6

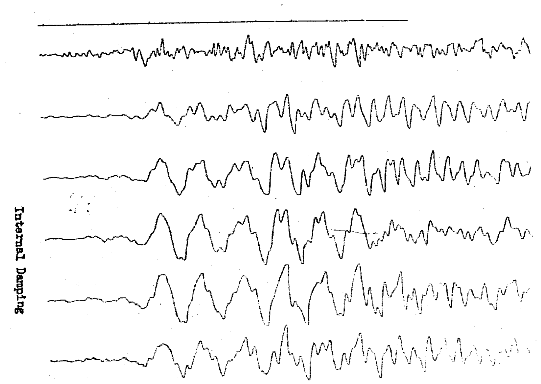


Fig 4-9. Earthquake Response

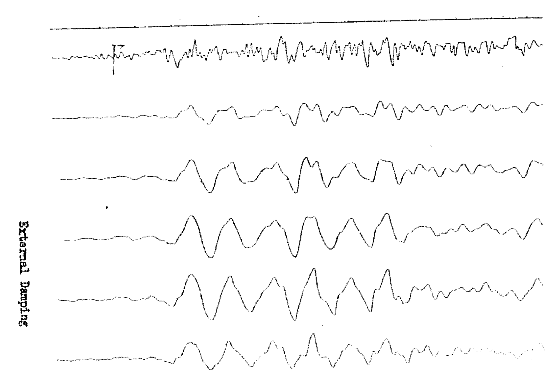


Fig 4-10. Earthquake Response

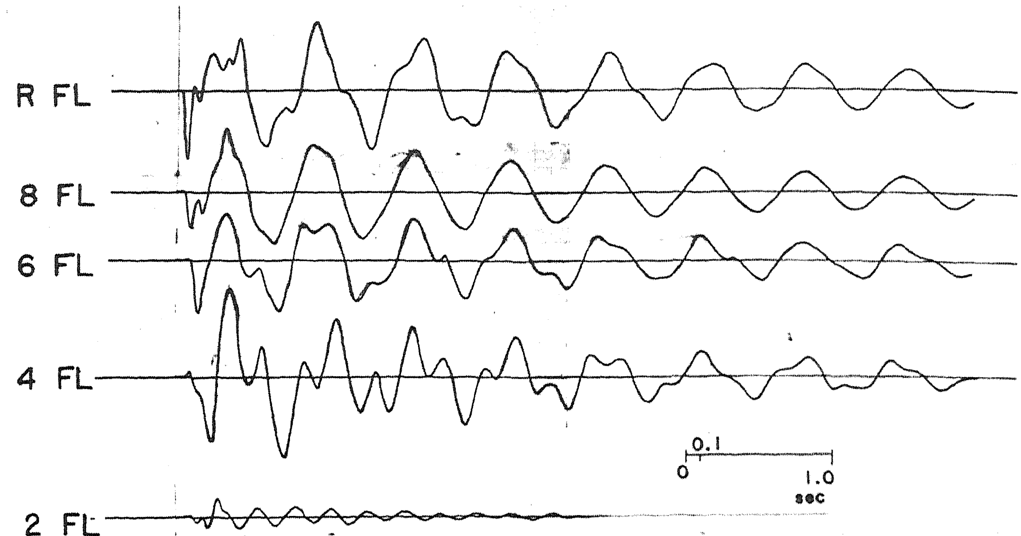


Fig. 3-7 MEASURED CURVE at 1 st stage

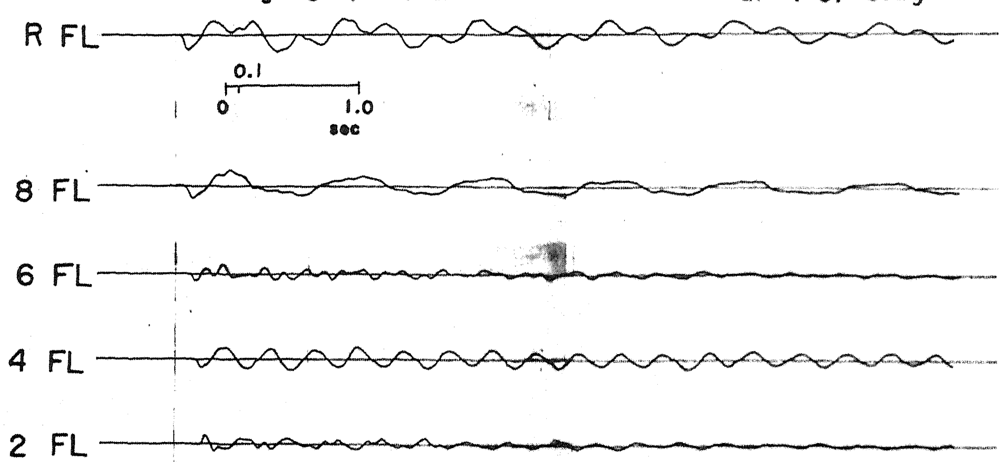


Fig. 3-8 MEASURED CURVE at 2 nd stage

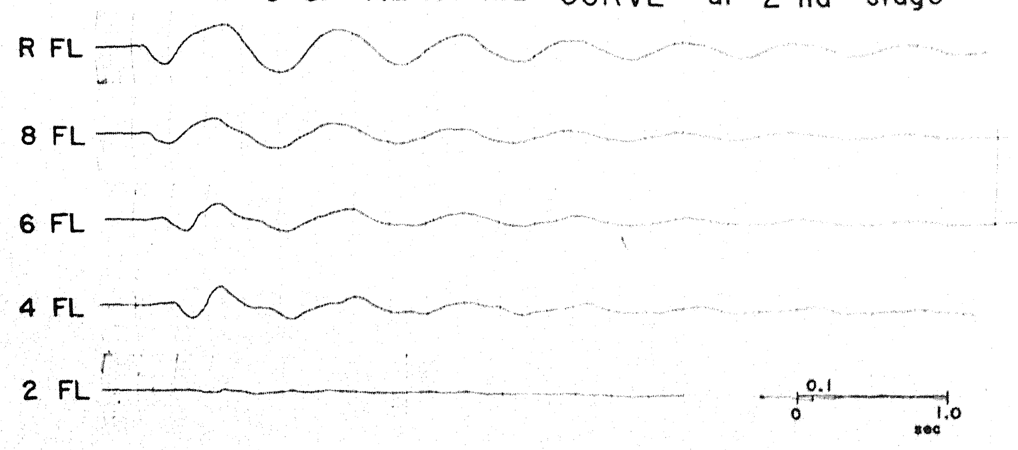


Fig. 3-9 MEASURED CURVE at 3 rd stage

Fig. 4-3 I st stage

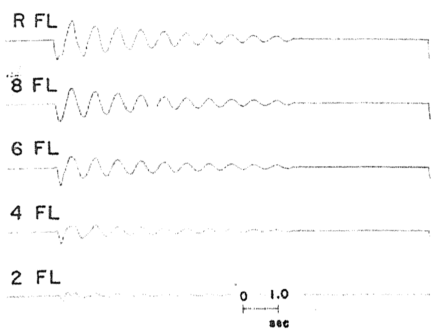


Fig. 4-4 2 nd stage

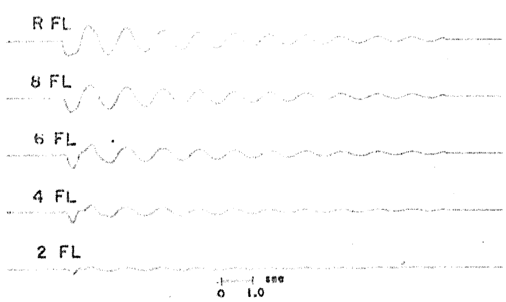
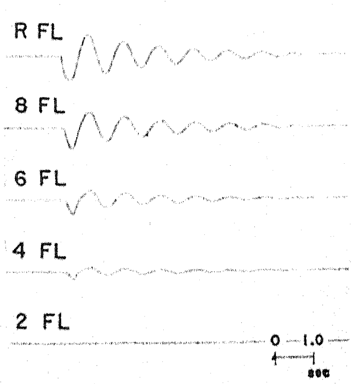


Fig. 4-5 3 rd stage



Internal Damping is considered

Fig. 4-6 I st stage

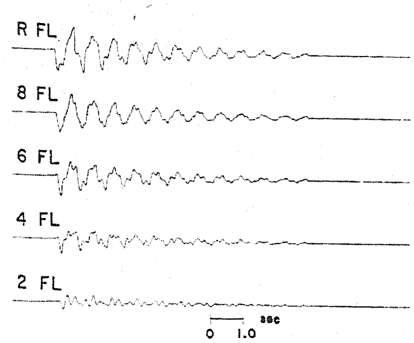


Fig. 4-7 2 nd stage

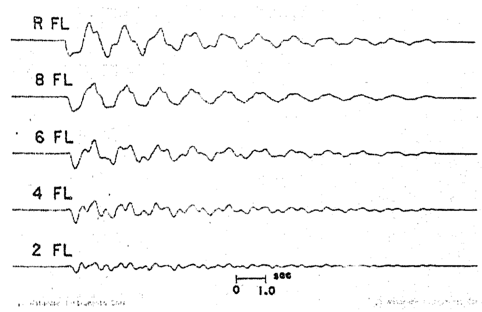
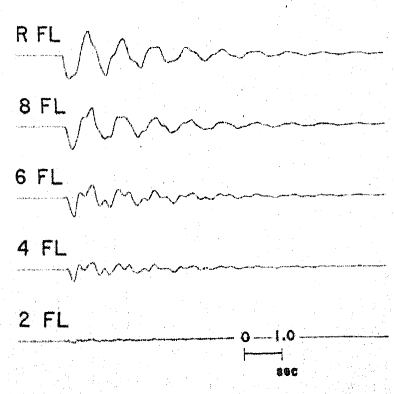


Fig. 4-8 3 rd stage



External Damping is considered

E R R A T A

THE VIBRATIONAL ANALYSIS OF A STEEL STRUCTURE (THE VIBRATIONAL TEST OF THE OHBAYASHI - GUMI BUILDING)

BY S. WATANABE, Y. KIDA, M. HIGUCHI

PAGE 697: Para 8, line 3: "...relative displacement of that floor to the upper and lower floor"
should read: "...relative displacement of that floor to the lower floor"

PAGE 7[illegible]: Table 3-2, Mode 3: delete: 6.85
replace by: 5.13

PAGE [illegible]3: Table 4 - 1, K_6 at 3rd stage: delete: 214.0
replace by: 124.0

THE VIBRATIONAL ANALYSIS OF A STEEL STRUCTURE (THE VIBRATIONAL TEST OF OHBAYASHI-GUMI BUILDING).

BY S. WATANABE, Y. KIDA AND M. HIGUCHI.

QUESTION BY: J.G. BOUWKAMP - U.S.A.

The authors have presented an interesting study on the actual behaviour of a steel building. I believe that on page 5 last paragraph, line 3, Table 3-6, should read Table 3-7. Furthermore I would appreciate if the speaker could give the magnitude of the deflections indicated as "small" and "large" in Table 3-6. I would also appreciate if the authors would care to clarify the information presented in figures 3-2 and 3-3.

Finally I wish to suggest to expand the discussion as presented in the paper now available, in order to learn in more detail about the results of this important study.

AUTHORS' REPLY:

1. As you pointed out, the references to Table 3-6 are incorrect. They should read Table 3-7 on page 5 last paragraph, line 3, page 5 last paragraph line 8; on page 6, first paragraph, line 9.

2. The details at third stage on Table 3-6, are as follows:

a) Added lateral force 1.0 ton.

Deflection	large deflection $\delta > \Delta$			Defl.	small deflection $\delta < \Delta$		
$\delta(\mu)$	ε^*	λ'^*	h^*	Δ (μ)	ε	λ'	h
R - F.L.	0.411	0.33	0.053	190-200	0.199	0.15	0.0238
8 - F.L.	0.476	0.38	0.061	114	0.256	0.192	0.0305
6. - F.L.	0.491	0.393	0.063	83	0.228	0.171	0.0272
4 - F.L.	0.432	0.346	0.055	66	0.139	0.104	0.0166

b) Added lateral force 2.0 ton.

Deflection $\delta(\mu)$	large deflection $\delta > \Delta$			Defl. Δ (μ)	small deflection $\delta < \Delta$		
	ε^*	Λ'^*	h^*		ε	Λ'	h
R - F.L.	0.424	0.339	0.054	226	0.235	0.176	0.028
8 - F.L.	0.395	0.316	0.0503	125	0.198	0.148	0.0236
6 - F.L.	0.464	0.371	0.059	97	0.159	0.117	0.019
4 - F.L.	0.398	0.318	0.051	72	0.198	0.147	0.0236

where $\Lambda' = 2 \log_e \nu = \varepsilon T$ logaritnmic decrement

$\nu = e^{\frac{\varepsilon T}{2}}$ damping ratio

$T = \dfrac{2\pi}{n(\sqrt{1-h^2})}$ foundamental period of the system

$h = \varepsilon / n$ fraction of critical damping

3. On Figures 3-2, 3-3, ratios of the deflection to time duration where plotted, using half logarithmic section paper.

Therefore, using two points - A(ξ_1, t_1), B(ξ_2, t_2), - on a curve in these Figures, the fraction of critical damping is calculated as follows:

$$\varepsilon = \frac{\log_e A_2 - \log_e A_1}{t_2 - t_1} = \frac{\log \frac{A_2}{A_1} \times \log_e 10}{t_2 - t_1} = \frac{2.3 \times \log \frac{A_2}{A_1}}{t_2 - t_1}$$

$$h = \varepsilon \times \frac{T}{2\pi}, \qquad (\sqrt{1-h^2} \simeq 1)$$