HORIZONTAL LOADING AND VIBRATION TEST ON 2-STORIED REINFORCED CONCRETE STRUCTURES

by

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ABSTRUCT

Four types of 2-storied reinforced concrete frames of office and hospital buildings for the Japanese National Railways were tested in order to find out their yield characteristics and destruction behaviour under static and dynamic horizontal load.

The results are as follows:-

(1) Stiffness, yield resistance force, and natural period showed good coincidence with calculated value in the usual type of open frame and composite type of walled frame and open frame (Figs. 5-11).

(2) Both these types had sufficient safety factors for the Japanese usual design load (seismic coefficient 0.2) (Figs. 5-7).

(3) The behaviour of the shear wall type was observed to be something differed from that expected from the laboratory test. This might be due to effect of the initial cracks on the wall, a small opening at the bottom of the wall, and the footing rotation.

NOMENCLATURE

- P : resistance force
- D : stiffness
- y : lateral displacement relative to the base
- δ : relative displacement
- T : natural period
- h : damping coefficient
- Si : i-th case of static horizontal loading test
- Di : i-th case of dynamic vibration test
- iCj : symbol of column at the i-th story and j-th type
- iGj : symbol of girder at the i-th floor and j-th type
- N : value of standard penetration test

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PURPOSE AND METHOD

The buildings in which the frames were tested had to be removed prior to the construction of the new Tokaido trunk line, and tests of actual buildings were performed to know the actural yield resistance force of frame, change of stiffness, natural period and damping coefficient, especially quantitative change in static and dynamic behaviour in a nearly destroyed actual frame.

Types of the frames tested are as follows:-

(1) A pair of two-spanned Open Frame of Maibara Car Maintenance Office JNR (Fig. 1).

(2) Composite type of two-spanned Walled Frame and open frame of Hamamatsu Hospital JNR (Figs. 2, 3).

(3) Balanced frame type of single Open Frame of Hamamatsu Hospital JNR (Fig. 4).

(4) The same form as (3) type but with a seismic Shear Wall at the 1st story of Hamamatsu Hospital JNR (Fig. 4).

1. Maibara Open Frame Test

The outside view and the skeleton of this building are shown on Photo. 1 and Fig. 1. Footings are supported with 4-5 precast concrete piles with 25 cm. diameter and 6 m. length, and the ground soil consists of the alternating soft clay (N = 2-9) and sand-gravel (N = 9-20) layers that extends over 17 m. depth.

The building was separated vertically in two parts by concrete breakers and the one of two spans in the longitudinal direction of this building was tested, and the other remainder of three spans of original frame was used to support the testing load. At the center of the roof of the test frame, a vibration exciter was installed with four bolts (Losenhausen type of two unbalanced weights rolling about respective axis, with 0-2,460 kg.cm. unbalanced moment, 4-20 c/s frequency range, 2,000 kg. maximum vibration force, 850 kg. weight) (Photo. 4).

Between the test frame and the original frame, a pair of 50 t. oil jacks (Photo. 2) and a wire rope - chain block system (Photo. 3) were set to make it possible to push and pull the test frame.

The horizontal displacement of each floor was measured by 1/100 mm. dial gauges and optical transits, and also the vertical displacement at the exterior column base was measured by dial gauges.

In the dynamic test, the horizontal movement of each floor was measured by accelerographs and moving coil type vibrographs.

2. Hamamatsu Walled Frame Test, Open Frame Test and Shear Wall Test

Skeletons of each test are shown in Figs. 2, 3, and 4. These footings stand immediately upon the 9 m. deep sand-gravel foundation.

In the walled frame test, two spans of the frames were separated from the original buildings as in the Maibara open frame test.

In the open frame test the lateral force is applied by means of 100 t. jacks from outside original structures (Photo. 9, 14).

Both in the walled frame and the open frame tests, the base floors and footing beams were not cutted off.

In the shear wall test 100 t. jacks were set between the test wall and the original frames.

The vibration exciter was unable to use at Hamamatsu, and only a small scale free vibration tests with 2-4 mm. initial deflections were performed.

Observation methods at Hamamatsu were the same as at Maibara.

RESULTS

As a result of the static tests, load-displacement curves in Fig. 5 to Fig. 8 show the change of their stiffness and the yield resistance force of each static test with the calculated values of them.

And as a result of dynamic tests, observed natural periods compared with calculated natural periods from the stiffness of each static test, and observed damping coefficients are shown in Fig. 9 to Fig. 12.

To know the destruction behaviour of these test frames, ultimate distributions of the cracks in them are shown in Fig. 13 to Fig. 16.

In order to give the considerations for these results, the theoretical values of the elastic stiffness, the maximum yield resistance force, and the natural period were carried out under the several assumptions.

Outline of the evaluation of theoretical values is as follows:-

(1) In the Maibara open frame test sections of members were assumed to be the design standard value with finishing mortars and in other tests without finishing mortars.

(2) Flexual parts of members assumed to range face to face.

(3) Young's modulus of concrete $E_c = 2.1 \times 10^2$ t/cm^2, and that of steel $E_s = 2.1 \times 10^3$ t/cm^2.

(4) Yield point stress of reinforcing bar, ranging 3.12 t/cm^2 - 3.53 t/cm^2, was determined by the tension test. Specimens were taken from the other parts of the buildings.

(5) Compressive strength of concrete was assumed to be 0.3 t/cm^2.

(6) For the calculation of the frames, the following methods were used:

(a) stiffness ----- Muto method[1] (D-value calculation)

(b) ultimate resistance force ---- Plastic theory; using the rectangular compressive stress distribution (85% compressive stress of concrete) in the section of members affected by bending moments,

(c) natural period ---- as a two-degree-of-freedom system, converted from the statically observed initial stiffness after the compared dynamic test.

(7) For the calculation of the shear wall, the following equations were used:

(a) deflections

$$\delta_s = \frac{Kh}{GA}P \quad \text{----------} \quad (1)$$

$$\delta_B = \frac{h^3}{3EJ}P \quad \text{---------} \quad (2)$$

$$\delta = \delta_s + \delta_B \quad \text{--------} \quad (3)$$

δ_s	:	shear deflection	(cm)
K	:	the coefficient of shearing angle	(1.2)
h	:	story height	(cm)
G	:	modulus of rigidity of concrete $G = E/2.3$	(90 t/cm^2)
A	:	sectional area of wall	(cm^2)
δ_B	:	bending deflection	(cm)
E	:	Young's modulus of concrete	(210 t/cm^2)
J	:	moment of inertia of area	(cm^4)

(b) ultimate resistance force (P max.)

$$P_{max} = \tau_{max} . A$$

In this equation τ_{max} is the maximum shearing strength of concrete (15 kg/cm^2)

Considering above-mentioned calculated value and that of observed at each test, the following description can be made.

1. Maibara Open Frame Test

(1) Maximum value of horizontal resistance force (P_Y) was measured as 80 t., while the calculated value was 61.4 t.

(2) The initial measured value of stiffness (D) was 50 t/cm while its calculated value was 80.3 t/cm.

And the value of stiffness at ultimate state was 30 t/cm (Fig. 5).

(3) The observed natural period (T) and damping coefficient (h) of building covered a range of 0.28 sec. (initial) to 0.41 sec. (ultimate) and 0.04 to 0.095, respectively. The calculated value (T_0) was 0.28 sec. (Fig. 9).

(4) Process of formation of yielding points under the horizontal load is as follows:

First, all of the tops and bottoms of the 2nd story columns cracked under bending moment; second, when the top of the frame moved about 6 cm., shear cracks developed at the top of the 2nd story interior column.

With an increase of the frame deflection, the shear crack developed at the bottom of the 2nd story interior column and the top and bottom of the 2nd story exterior column.

1st story columns were cracked under bending moment, but still there were little shear cracks and no failure under shear force. —(Fig. 13)(Photo. 7).

2. Hamamatsu Walled Frame Test

(1) P_Y of this frame was measured as 130 t., while the calculated value was 101 t. It showed that the value of safety factor for lateral force of the usual design (2nd story 24 t., 1st story 42 t.) was sufficient.

The most remarkable fact was that both the walled frame and the open frame (Fig. 2, 3) bore horizontal load with a great reserve of ductility, even after yield resistance force and even when the lateral displacement of them attained 30 cm. at the top of frames 7.5 m. high.

However, the development of shear cracks in the walled frame was remarkable in comparison with those in the open frame shown in Fig. 3. (see Fig. 14 and Photo. 12)

(2) The observed values of T and h covered a range of 0.18 sec. to 0.29 sec. and 0.03 to 0.04, respectively, while the calculated value of T_0 was 0.16 sec. (Fig. 10).

3. Hamamatsu Open Frame Test

(1) P_Y of this frame was measured as 63.4 t., while the calculated value was 55 t. The value of the safety factor for lateral force of the usual design (2nd story 8.4 t., 1st story 13 t.) was sufficient (Fig. 7).

(2) The fact of the decrease in static stiffness with repetitions of horizontal loading was confirmed, in view of the observed relations between load and displacement through these three horizental loading tests.

(3) The observed values of T and h covered a range of 0.14 sec. to 0.29 sec. and 0.03 to 0.08, respectively (Fig. 11).

(4) In the deflected open frame the rotation angle of the joint (θ) at the end of interior column was considerably large as compared with the mean deflection angles (R) for the column, while the value of θ at the end of exterior column was nearly equal to zero because the exterior column was of thin wall type with low stiffness.

4. Hamamatsu Shear Wall Test

(1) P_Y of this wall was measured as 76 t., while the calculated value was 72 t. (Fig. 8).

(2) The wall base slipped on the footing beam and the upside of the footing and the bottom of the inner column had been heavily cracked (Fig. 16).

(3) The result of free vibration test showed that the natural periods ranged from 0.14 sec. to 0.20 sec. and the damping coefficients from 0.03 to 0.06. These values seem to be independent of the yielding of the wall or the amount of amplitude in this case (Fig. 12).

(4) In comparison with recent experiences of seismic shear wall tests in laboratory, it can be noticed that the maximum shear resistance force of the actual wall becomes considerably smaller than expected, owing to the initial cracks due to piping or duct. The shear stress of the concrete was measured as 12 kg/cm^2 for the full section of the wall; 15 kg/cm^2 for the section without duct space.

ACKNOWLEDGEMENT

Thanks are expressed to Dr. Hajime Umemura, Professor of Faculty of Engineering, University of Tokyo, and Dr. Yutaka Osawa, Assistant Professor of Earthquake Research Institute, University of Tokyo, and other many co-operaters, for their kind suggestion and excellent support in carrying out these field tests.

REFERENCE

1. Seismic Analysis of Reinforced Concrete Buildings (1956) Kiyoshi Muto, Proceeding 1956 World Conference on Earthquake Engineering.

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Table I.

PROGRAM OF TEST

Test Series	Sand Used	Load Applied
1 --- A	Dense, Air-Dried Sand	Vertical Load
	Loose, Air-Dried Sand	
1 --- B	Dense, Saturated Sand	
	Loose, Saturated Sand	
2	Dense, Air-Dried Sand	Diagonal Load
	Loose, Air-Dried Sand	

Table II.

PHYSICAL PROPERTIES OF SAND USED.

Coefficient of Uniformity		3.78
Specific Gravity		2.59
Water Content of Air-Dried Sand		1.95
Void Ratio	Dense, Air-Dried Sand	0.54
	Loose, Air-Dried Sand	0.62
	Dense, Saturated Sand	0.58
	Loose, Saturated Sand	0.63
Relative Density	Dense, Air-Dried Sand	77 %
	Loose, Air-Dried Sand	53 %
	Dense, Saturated Sand	68 %
	Loose, Saturated Sand	50 %

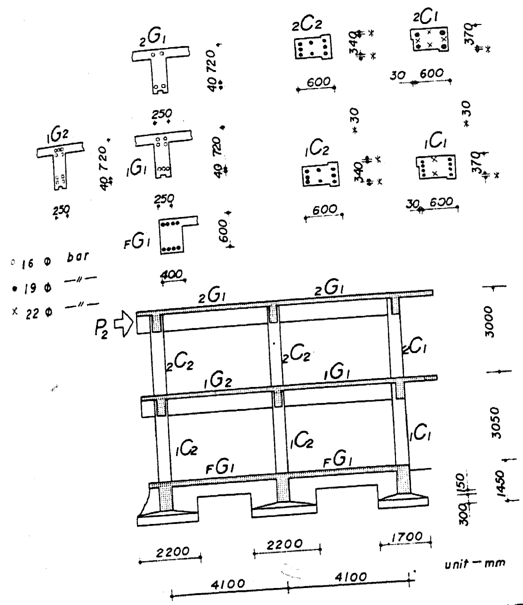
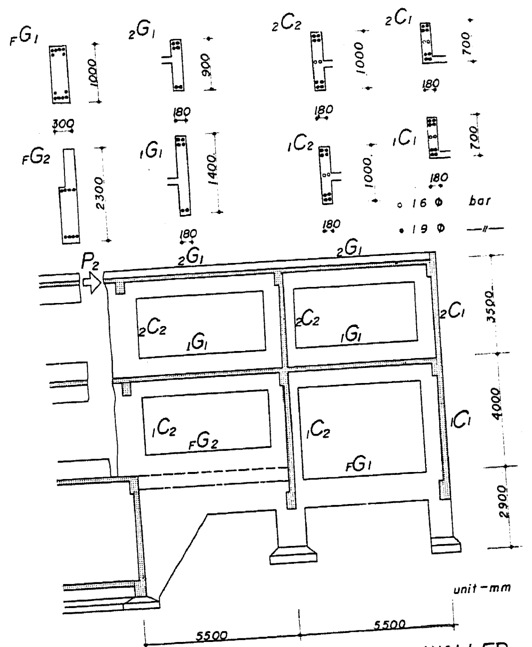
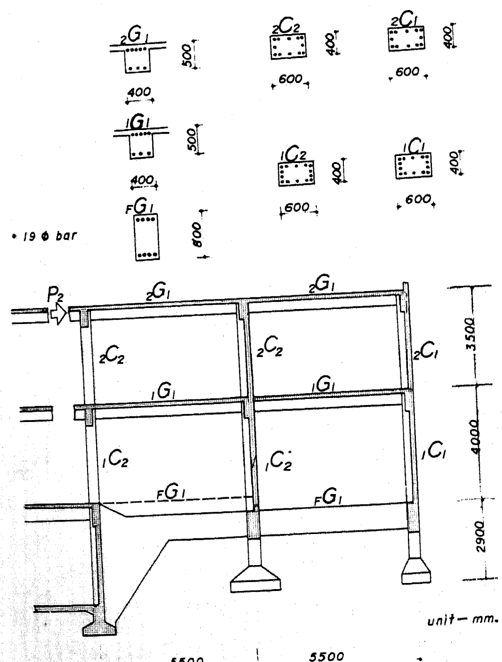


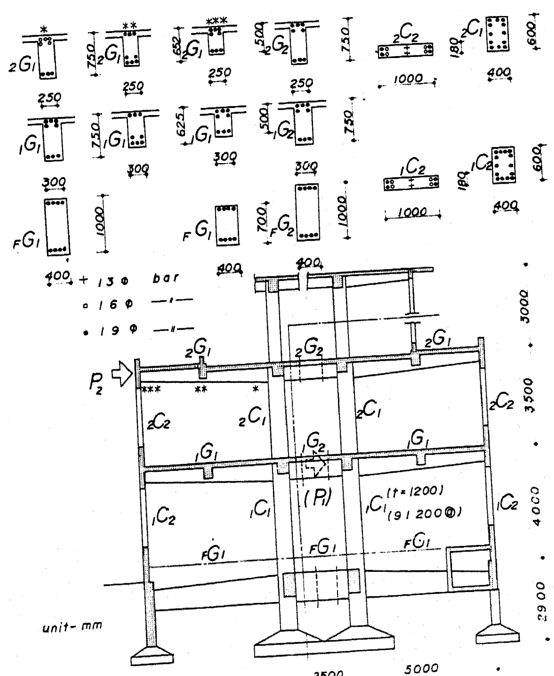
FIG. 1 SKELETON OF MAIBARA OPEN FRAME



FIG·2 SKELETON OF HAMAMATSU WALLED FRAME (A PAIR OF FIG·3 FRAME)



FIG·3 SKELETON OF HAMAMATSU FRAME (A PAIR OF FIG.2 WALLED FRAME)



FIG·4 SKELETON OF HAMAMATSU OPEN FRAME [( ) ····SHEAR WALL ONLY]

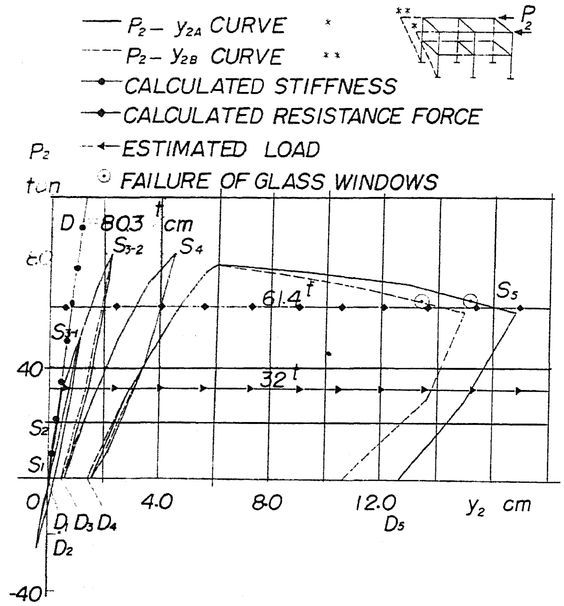


FIG.5 LOAD-DISPLACEMENT CURVE OF MAIBARA OPEN FRAME

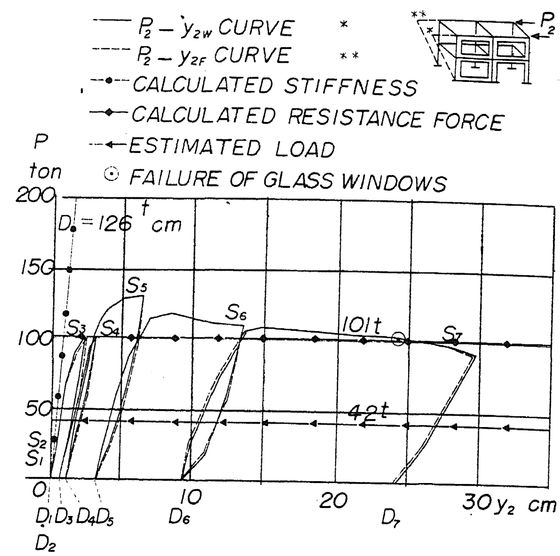


FIG.6 LOAD - DISPLACEMENT CURVE OF HAMAMATSU WALLED FRAME

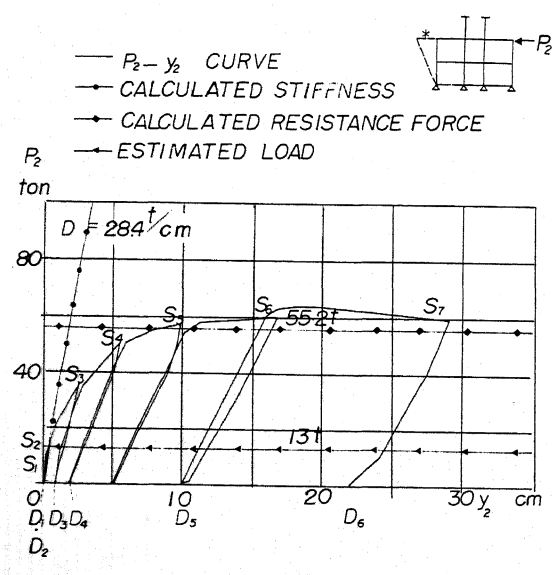


FIG.7 LOAD-DISPLACEMENT CURVE OF HAMAMATSU OPEN FRAME

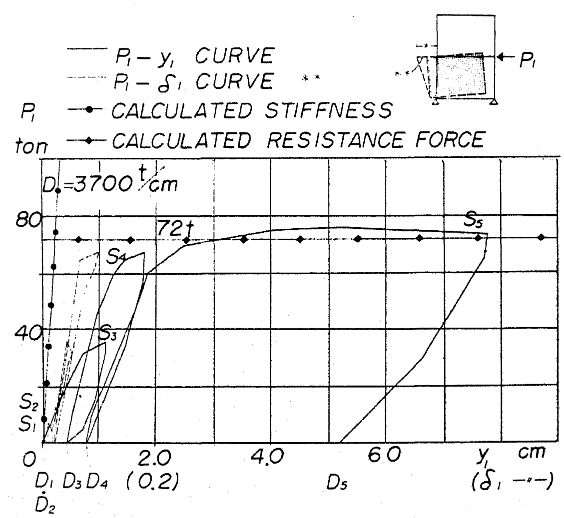


FIG. 8 LOAD- DISPLACEMENT CURVE OF HAMAMATSU SHEAR WALL

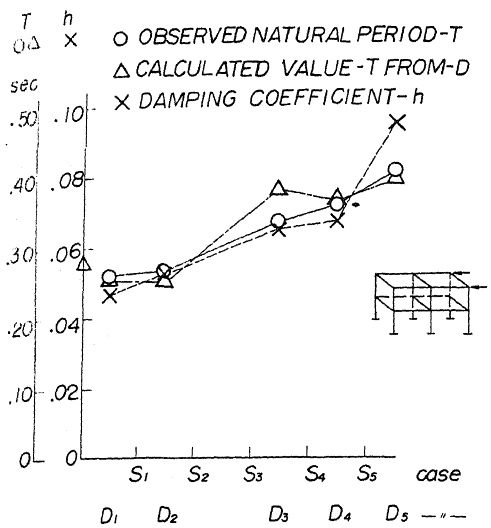


FIG.9-CHANGE OF STATIC AND DYNAMIC PROPERTIES OF MAIBARA OPEN FRAME IN EACH LOADING CASE

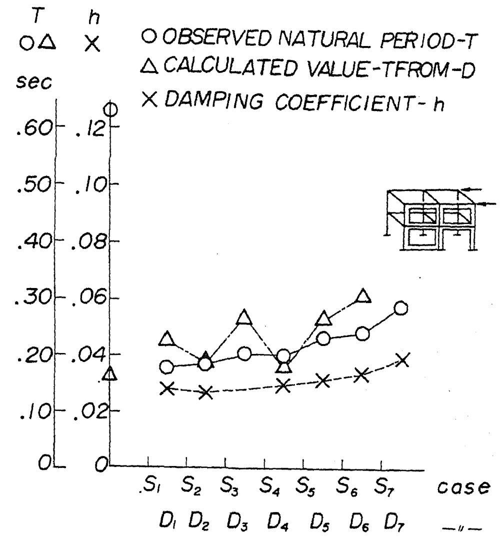


FIG.10-CHANGE OF STATIC AND DYNAMIC PROPERTIES OF HAMAMATSU WALLED FRAME IN EACH LOADING CASE

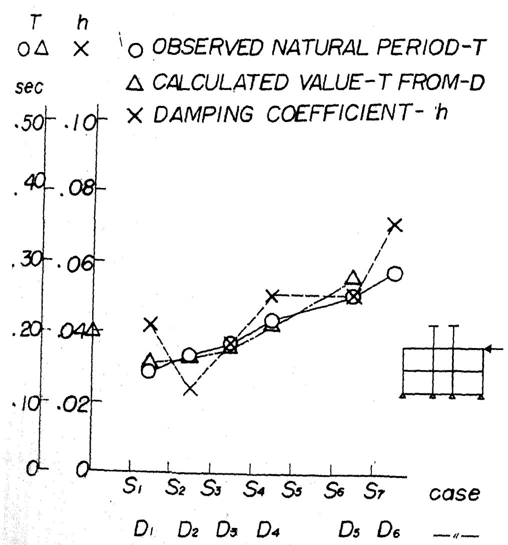


FIG.11-CHANGE OF STATIC AND DYNAMIC PROPERTIES OF HAMAMATSU OPEN FRAME IN EACH LOADING CASE

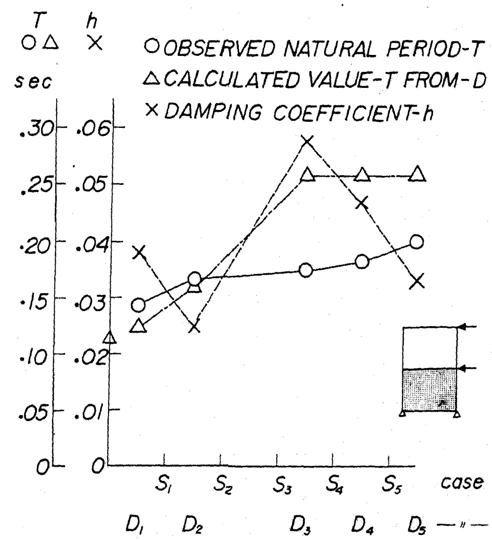


FIG.12- CHANGE OF STATIC AND DYNAMIC PROPERTIES OF HAMAMATSU SHEAR WALL IN EACH LOADING CASE

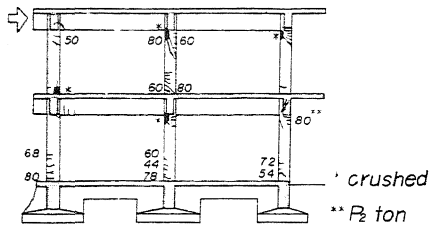


FIG. 13 CRACK IN MAIBARA OPEN FRAME

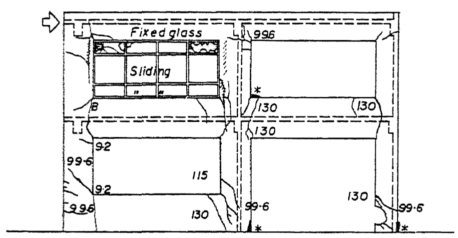


FIG. 14 CRACK IN HAMAMATSU WALLED FRAME

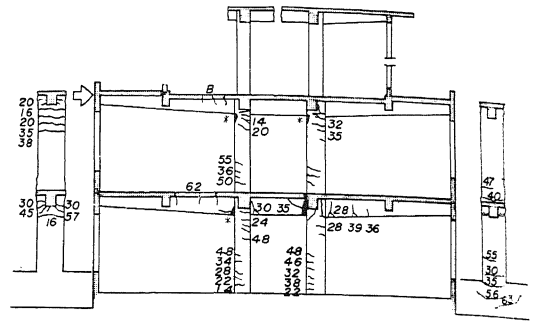


FIG. 15 CRACK IN HAMAMATSU OPEN FRAME

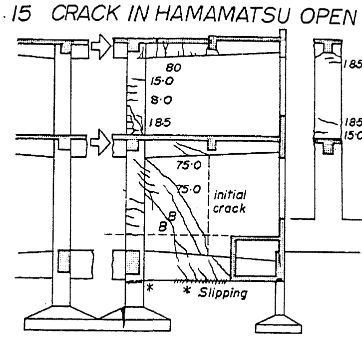


FIG. 16 CRACK IN HAMAMATSU SHEAR WALL

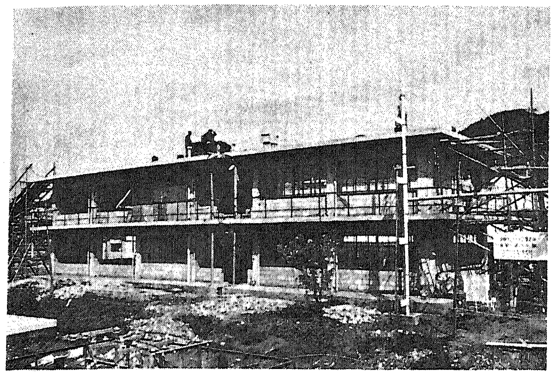


Photo 1. Outside view of Maibara Open Frame.

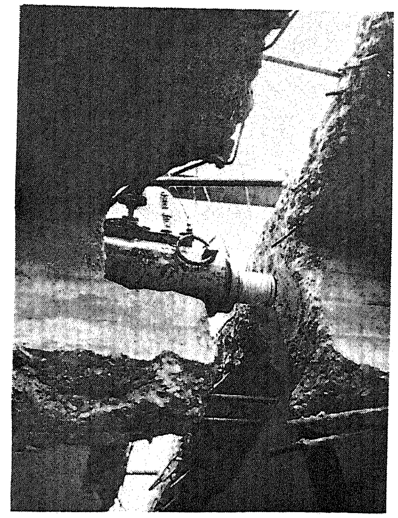


Photo 2. Setting of 50ft. Oil Jack (Maibara).

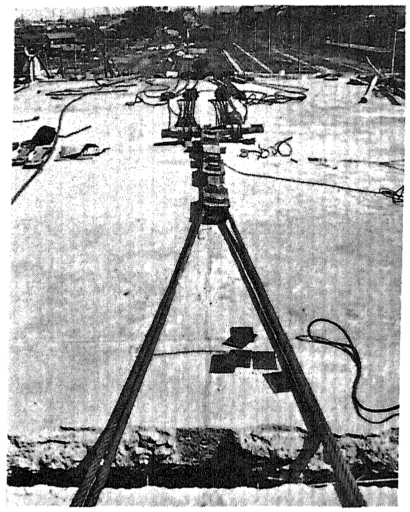


Photo 3. Wire-Rope, Chain Block System (Maibara).

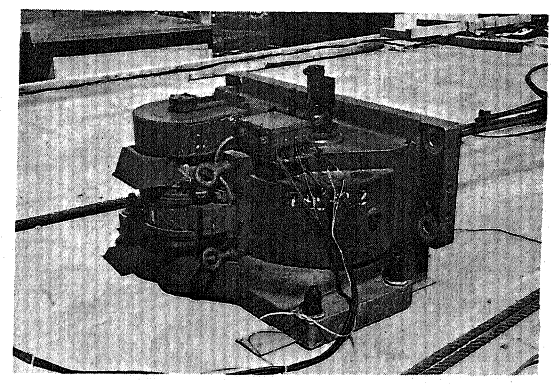


Photo 4. Vibration Exciter.

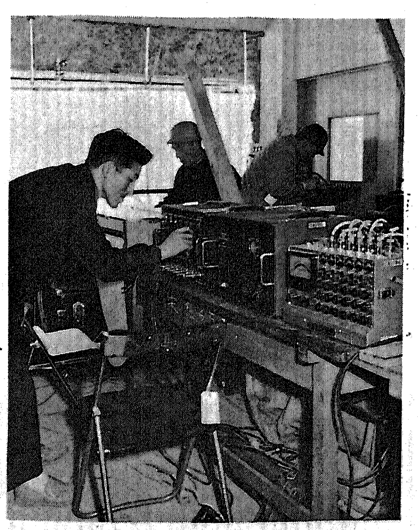


Photo 5. Amplifier and Recorder of Accelerometer.

Photo 6. View in Maibara Static Test.

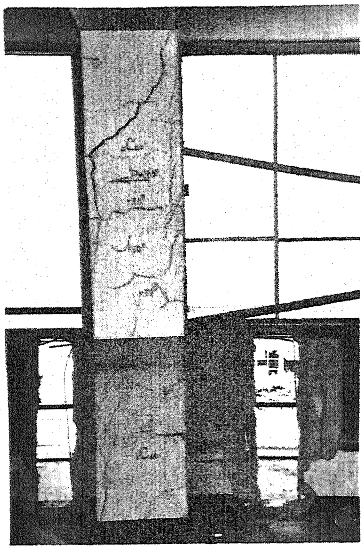


Photo 7. Shear Cracks in 2nd Storey Interior Column (Maibara).

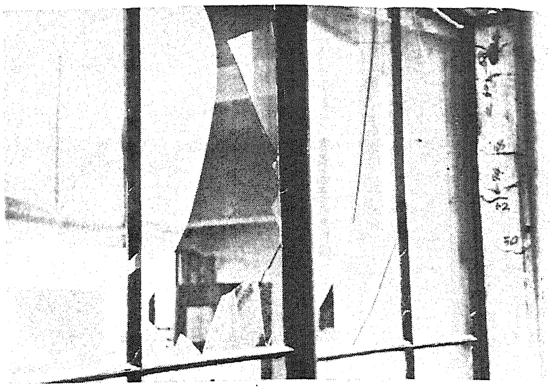


Photo 8.. Glass Failure.

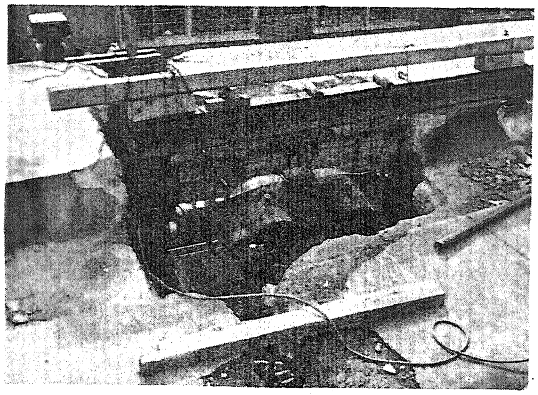


Photo 9. Setting of 100ft. Jacks.

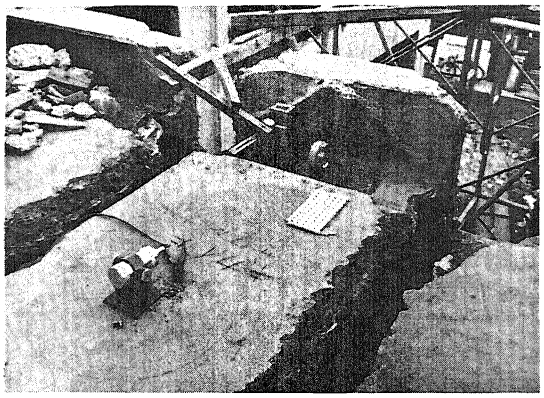


Photo 10. Dial Gauge P.V. Vibrograph and Coincidence Levelmeter (Hamamatsu).

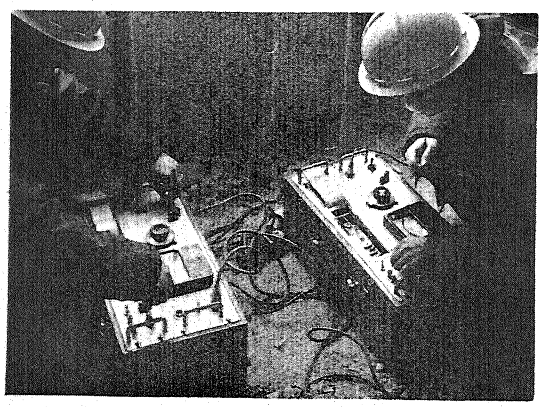


Photo 11. View of Recording by Vibrograph (Hamamatsu).

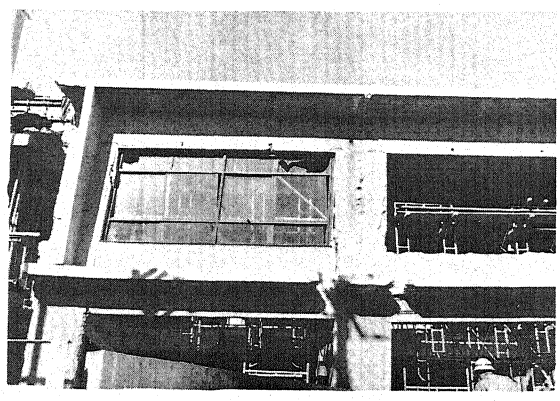


Photo 12. Cracks in Walled Frame and Glass Failure (Hamamatsu).

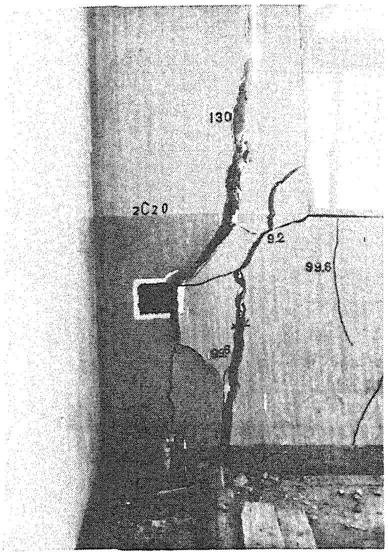


Photo 13. Cracks in 2nd Storey Interior Column (Hamamatsu Walled Frame).

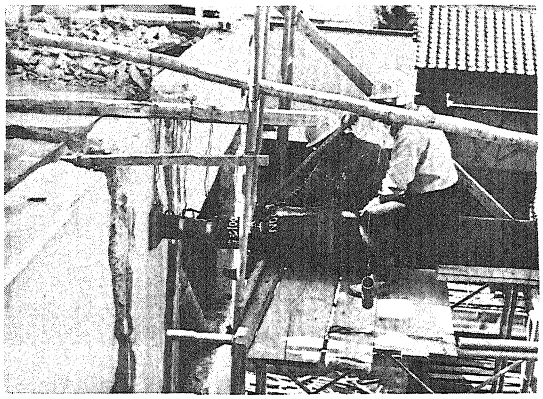


Photo 14. Setting of 100ft. Jacks. (Hamamatsu Open Frame Test).

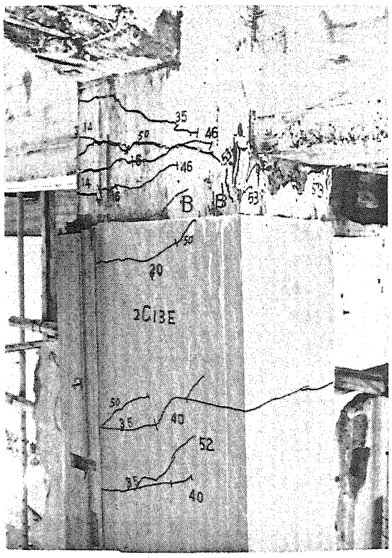


Photo 15. Cracks at top of 2nd Storey Interior Column (Hamamatsu Open Frame Test).

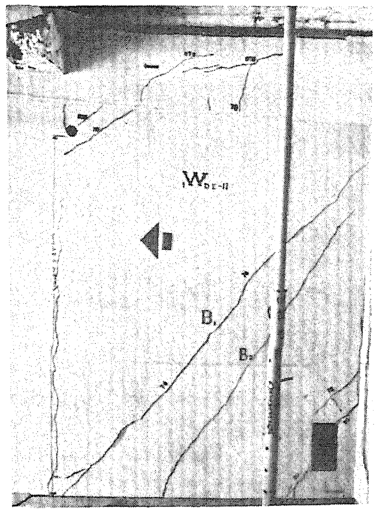


Photo 16. Cracks of the Shear Wall (Hamamatsu Shear Wall Test).

Photo 17. Cracks of the Footing of Interior Column (Hamamatsu Shear Wall Test).

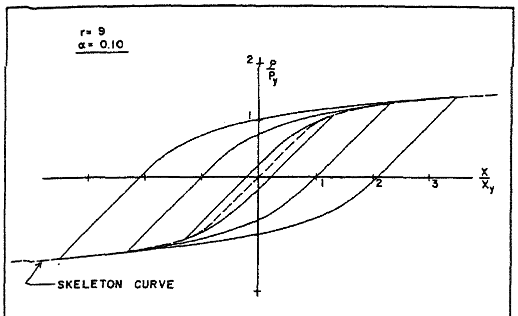


FIG. 16 SKELETON CURVE AND HYSTERESIS LOOPS FOR $\alpha = 0.10$ AND $r = 9$.

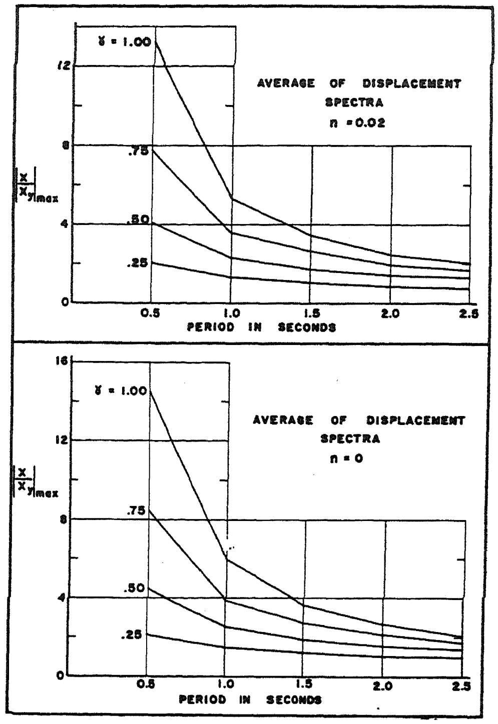


FIG. 17 AVERAGE DISPLACEMENT SPECTRA FOR $n = 0.02$ AND $n = 0$.

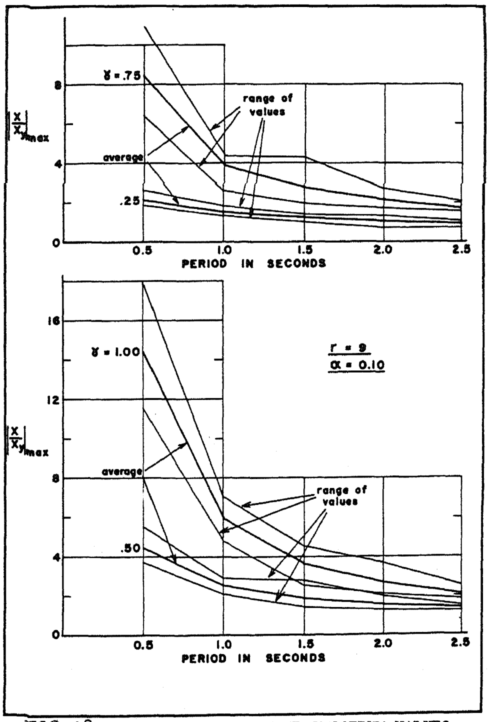


FIG. 18 RANGE OF DISPLACEMENT SPECTRUM VALUES FOR $n = 0$.

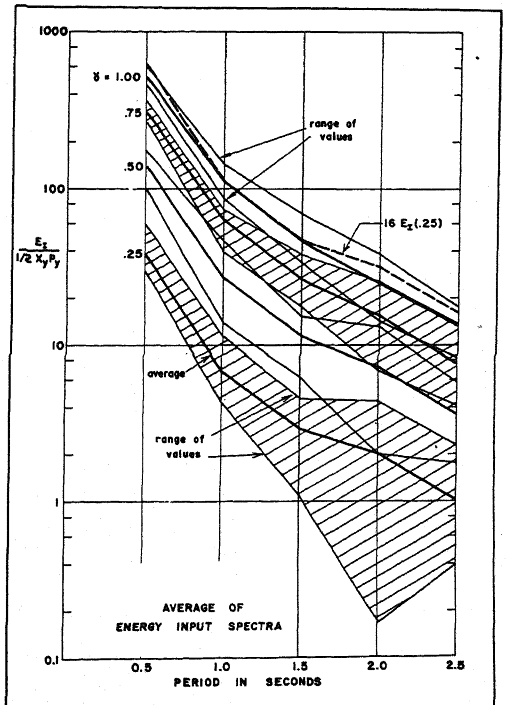


FIG. 19 AVERAGE ENERGY INPUT SPECTRA FOR $n = 0.10$, 0.05, 0.02, and 0.

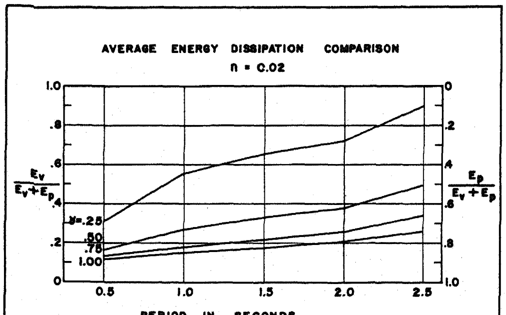


FIG. 20 AVERAGE RELATIVE AMOUNTS OF ENERGY DISSIPATED BY YIELDING AND VISCOUS DAMPING FOR $n = 0.02$.

Fig. 18

RESPONSE CURVE OF MASS M SHOWN IN Fig. 8 FOR 0 - 10.0 SEC & 10.0 - 24.5 SEC RECORD OF EL CENTRO EARTHQUAKE OF MAY 18, 1940 USING N-S & E-W COMPONENTS HAVING AN ANGLE OF $\theta = 22.5°$ WITH AXIS X AND CONSIDERING SLIGHT DIFFERENCE OF ELASTICITY OF MAIN AXIS AS SHOWN IN Fig. 8 & Fig. 15

Fig. 18-1 $\theta = 22.5°$ T = 0 - 10.0 SEC

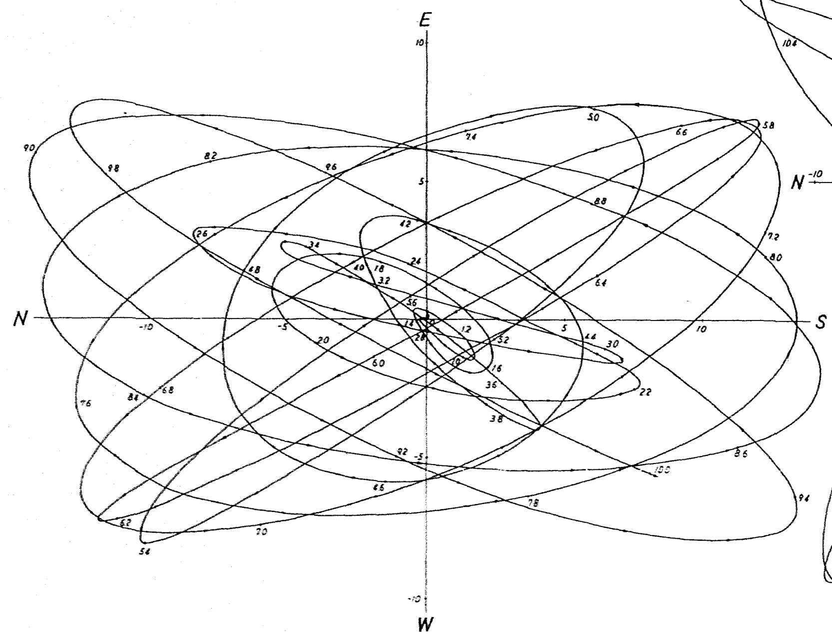


Fig. 18-2 $\theta = 22.5°$ T = 10.0 - 24.5 SEC

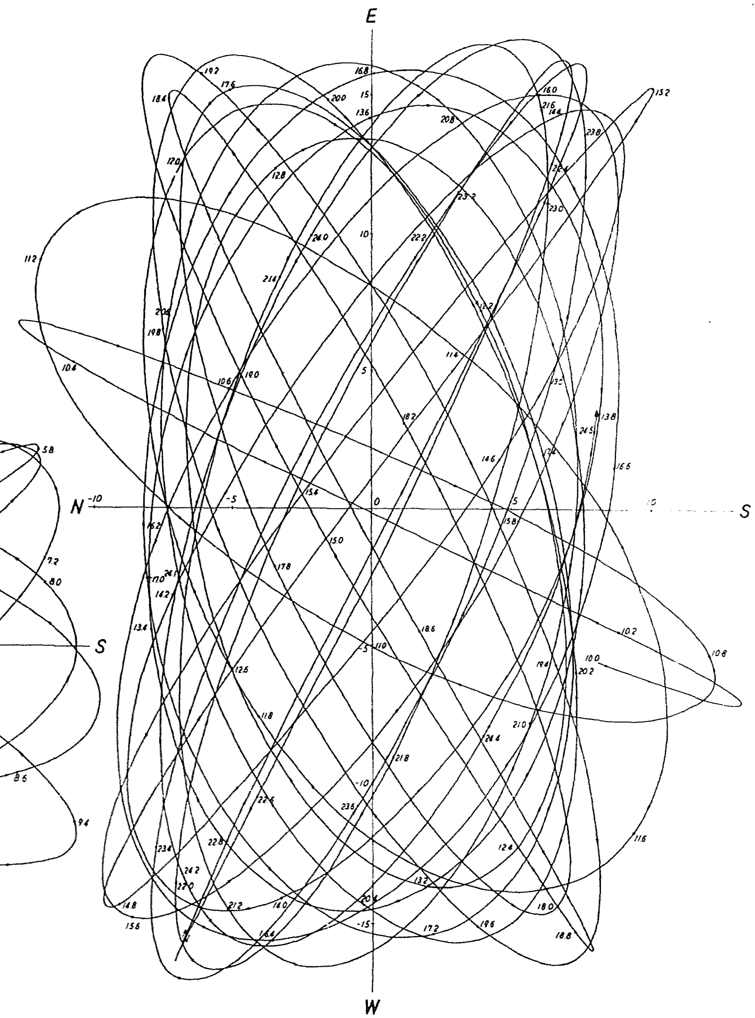


Fig. 19

RESPONSE CURVE OF MASS M SHOWN IN Fig. 8 FOR 0 - 10.0 SEC & 10.0 - 24.5 SEC RECORD OF EL CENTRO EARTHQUAKE OF MAY 18, 1940 USING N-S & E-W COMPONENTS HAVING AN ANGLE OF $\theta = 45°$ WITH AXIS X AND CONSIDERING SLIGHT DIFFERENCE OF ELASTICITY OF MAIN AXIS AS SHOWN IN Fig. 8 & Fig. 15.

Fig. 19-1 $\theta = 45°$ T = 0 - 10.0 SEC

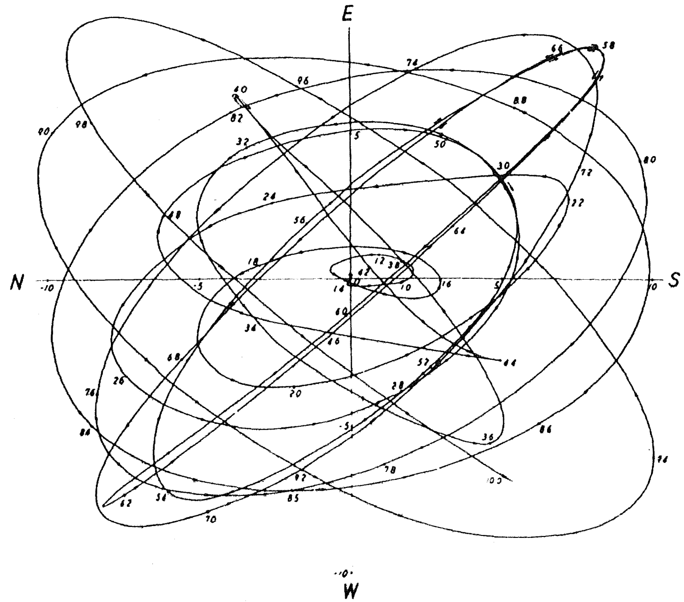


Fig. 19-2 $\theta = 45°$ T = 10.0 - 24.5 SEC

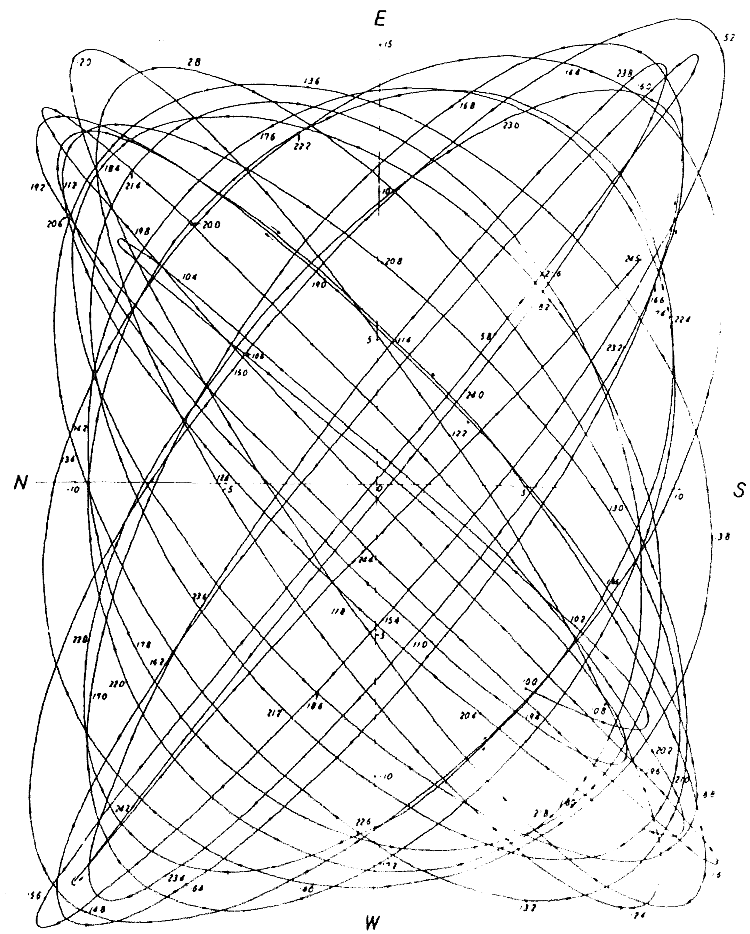


TABLE 1

NATURAL FREQUENCY OF VIBRATION

$$p = C_1 \sqrt{k/m}$$

n	S_b/S_c	VALUE OF $C_1 \times n$			
		1ST. MODE	2ND. MODE	3RD. MODE	4TH. MODE
	∞	1.0000			
	10	0.9763			
1	5	0.9549			
	2	0.9014			
	1	0.8367			
	∞	1.2361	3.2361		
	10	1.1750	3.1746		
2	5	1.1236	3.1224		
	2	1.0086	3.0013		
	1	0.8876	2.8657		
	∞	1.4232	4.1542	6.5486	8.4126
	10	1.3218	3.9027	6.2721	8.2288
5	5	1.2405	3.6986	6.0433	8.0737
	2	1.0683	3.2591	5.5369	7.7215
	1	0.8999	2.8205	5.0137	7.3455
	∞	1.4946	4.4504	7.3068	10.0000
	10	1.3763	4.1129	6.7993	9.3949
10	5	1.2828	3.8451	6.3935	8.9058
	2	1.0886	3.2863	5.5388	7.8618
	1	0.9024	2.7483	4.7091	6.8344
	∞	1.5321	4.5873	7.6156	10.5993
	10	1.4047	4.2100	7.0030	9.7750
20	5	1.3048	3.9140	6.5213	9.1246
	2	1.0994	3.3041	5.5263	7.7762
	1	0.9042	2.7244	4.5791	6.4891

TABLE 2

FACTOR F, FOR FINDING EQUIVALENT LENGTH OF COLUMN

$$F = \left\{ \left(p_{rd}/p_{fe} \right)^{2/3} \right\}_{1st\ Mode}$$

n	F					Value of C_1 for rigid structure 1ST. MODE
	$S_b/S_c=1$	$S_b/S_c=2$	$S_b/S_c=5$	$S_b/S_c=10$	$S_b/S_c=\infty$	
1	1.126	1.072	1.031	1.016	1.000	1.0000
2	1.248	1.145	1.065	1.035	1.000	0.6180
5	1.358	1.211	1.096	1.050	1.000	0.2846
10	1.400	1.235	1.108	1.056	1.000	0.1495
20	1.421	1.256	1.111	1.059	1.000	0.0766

E R R A T A

HORIZONTAL LOADING AND VIBRATION TEST ON 2-STORIED REINFORCED CONCRETE BUILDING

BY M. IHARA AND C. UEDA

PAGE 226:	Table I. delete: "Program of Test"					
	Table II. delete: "Physical properties of sand used"					
PAGE 231:	Photo	2.	for 50 ft	read	50 t	
PAGE 232:	"	9.	" 100 ft	"	100 t	
	"	10.	" P.V.	"	P.U.	
PAGE 233:	"	14.	" 100 ft	"	100 t	

HORIZONTAL LOADING AND VIBRATION TEST ON 2-STORIED REINFORCED CONCRETE STRUCTURES

BY M. IHARA AND C. UEDA

QUESTION BY: W.E. SAUL - U.S.A.

Analogous to the pressure test for acceptance of pressure vessels would it be feasible to require a shaking test for acceptance of aseismic buildings in the code?

REPLY BY: C. UEDA.

The author cannot give an immediate answer to these questions, because the methods of free vibration or forced vibration by an exciter which can be used at present are not sufficient in vibrating energy to produce such a shaking test for the acceptance of aseismic buildings in the code.

QUESTION BY: N.M. NEWMARK - U.S.A.

How was the damping factor determined?

Was this done for relatively small oscillations after each permanent displacement Dn that was recorded?

REPLY BY: C. UEDA

The damping factor was determined by the free vibration test (Maibara and Hamamatsu test) and forced vibration test (Maibara test) between each static horizontal loading test, but because of the lack of vibrating energy the amplitude was relatively small.

Then the result of the damping factor cannot be used for the assumption of the behaviour in the large amplitude vibration and as you mentioned it may be somewhat incorrect because of the existence of residual permanent displacement from the previous static test.

The author also considers such a vibration test should be done under equivalent large amplitude to the maximum displacement of the previous static test if it becomes possible.

QUESTION BY: V.A. MURPHY - NEW ZEALAND

The cracks shown in the photograph at the junction of beam and columns are well distributed. Would the author please state the spacing of the ties or stirrups for shear reinforcement?

REPLY BY: C. UEDA

The spacing of stirrups for shear reinforcement is 10cm centre to centre at the end of beams and columns, 15cm centre to centre at the middle part of them; and the stirrup diameter is 6 mm ∅.

QUESTION BY: K. KUBO - JAPAN

Have you had any actual seismic records obtained at these tested buildings?

REPLY BY: C. UEDA

We have not any actual seismic records at these tested buildings as there was no time because these buildings had to be removed as soon as possible.