VIBRATION TEST OF STEEL FRAME HAVING PRECAST CONCRETE PANELS

by

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SYNOPSIS

A two-storey, two-bay steel frame model having full-size precast concrete panels was subjected to vibration along its X, Y and D (45° diagonal) axes by means of a vibration generator placed on top of the model. These vibration tests were intended for obtaining some basic data on the effects of precast concrete panels on the vibration characteristics of the high-rise building and for ascertaining in detail the behaviors of the panel fastening system proposed for use in the final design.

1. INTRODUCTION

In addition to metal curtain walls, precast concrete panels have come to be used for some high-rise buildings recently constructed in Japan. Precast concrete panels, however, present some new problems in the structural design of tall buildings because their rigidity and weight are greater and material toughness are smaller than those for metal curtain walls.

It, therefore, is anticipated that these concrete panels will give large effects on such vibration characteristics as stiffness and damping of the building.

If precast concrete panels are to be used for high-rise buildings with satisfactory results, they must be so detailed that they can be fastened to the structural frames in such a way that the weight of each panel is well balanced and that they can cope with a rather large deflection which may occur to the framework when it is subjected to lateral forces such as seismic loads and wind forces.

Recently, the authors adopted granite surfaced precast concrete panels for the cladding of Sanwa Tokyo Building which has 25 floors and 4 basements, with a total height of about 100 m. Structurally, the third and higher floors of the building are designed as genuine steel frames. This present paper reports the forced vibration tests on a full scale

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partial model of the building.

2. SANWA TOKYO BUILDING

Since the tests reported in this paper were conducted in order to obtain the design data for Sanwa Tokyo Building, the structural outline of this building will be briefly introduced. Fig. 1 shows the overall view of the building under construction as of December 1972. Fig. 2 and Fig. 3 indicate the typical beam plan and the framing elevations respectively. In direction of the X axis, the beams are supported by columns placed at narrow intervals of 3.15 m. Along the Y axis, rather long steel-concrete composite beams having spans of 24 m, 19.7 m and 13.6 m are adopted. Seismic loads are carried by rigid framework comprising Frames X2 and Y1 which are combined with bracings as shown in Fig. 2.

From Fig. 4 which shows the natural periods in direction of the X and Y axes, it will be known that this building has almost identical natural periods with respect to both the X axis and the Y axis.

Fig. 5 indicates the design shear force and the result of elastic analysis of the earthquake response in direction of the X axis. The NS component of El Centro Earthquake (1940) caused the greatest response values both with respect to the X axis and the Y axis, and the maximum accelleration due to earthquake motion causing framework on any floor level to reach its elastic design strength was known to be about 300 gals. Fig. 6 shows the result of analysis on the elastic and plastic response to El Centro NS component. The maximum storey deflections under the foregoing condition were 1.5 times and 2.0 times as large as the deflection at elastic limit in directions of the X axis and the Y axis respectively. The deflection of the building under the design shear force is shown in Fig. 7. The overall deflection at the top of the building and the storey deflection were computed and known to be about 46 cm and 2 cm respectively.

In consideration of the foregoing vibration characteristics of the building, it was determined that precast concrete panels to be used should meet the following two conditions: (a) the members to which the panels are fastened should stay in an elastic range until storey deflection exceeds 3 cm, and (b) the panels should be fastened in such a manner that they will not fall off under storey deflection of not exceeding 5 cm.

3. TESTS

Based on the foregoing conditions, beam and column panel arrangement usable for the north and south facades of Sanwa Tokyo Building was assumed, and a series of tests by use of a two-bay, two-storey model of the structure was planned. Fig. 8 indicates the north elevation.

A two-storied test frame having two concrete floor slabs, each 6.3 x 6.0 m in size, supported by square tubular steel columns (80 x 80 x 3.2 mm) was constructed. In this test frame, the center of gravity coincides with the center of rigidity. Thus, theoretically speaking, the test frame, if assessed by a simulated two mass system, is believed to have identical natural periods and modes of vibration with respect to the X, Y, and D (45° diagonal) axes. To one side of the two-bay, two-storey test frame which has a floor height of 3.84 m and a span of 3.15 m, six column-covering panels, four beam-covering panels and steel windows were fastened, all according to the detail proposed for the actual building.

Vertical joints between the panels covering columns and beams were 25 mm wide and horizontal joints between the panels covering upper and lower columns were 20 mm wide, and all joints were caulked. Each column-covering panel weighed 2.8 tons and beam-covering panel, 1.5 tons.

The following types of tests were conducted:

(I) Forced Vibration Test:

A vibration generator was placed at the center on the top of the test frame, and vibration was given in directions of the X axis (within the plane of panels), the Y axis (outside the plane of panels) and the D (45° diagonal) axis.

(II) Free Vibration Test (Vibration within Plane of Panels):

A wire rope was tied to the top edge of the test frame, and it was tensioned to give an overall deflection of 6 cm, or a pre-determined storey deflection of 3 cm. Then, by cutting the rope, the frame was subjected to free vibration. Next, large drifts, i.e., a total deflection of 10 cm and a storey deflection of 5 cm, were caused to the frame in the same manner as described above in order to observe the behaviors of panel fastenings, caulking and window fastenings.

(III) Dynamic Load Tests by giving Vibration during Dismantling of Cladding:

Since precast concrete panels, caulking in panel joints and steel windows were thought to give considerable effects on the vibration characteristics (stiffness, damping, etc.) of the test frame, further vibration tests were conducted upon removal of the foregoing wall components. The tests were done at the following stages:

(a) Firstly, upon removal of panel joint caulking

(b) Secondly, upon removal of steel windows, and

(c) Lastly, upon removal of precast concrete panels, namely, when the steel frame only was left.

Fig. 9 shows the view of the test frame including the test apparatus, Fig. 10 and Fig. 11 show its plan and section, and Fig. 12 indicates the detail of fastening method used for column-covering panels.

Deflection of the structural frame and behavior of precast concrete panels under vibration were measured by means of accellometer and deflection meter.

4. TEST RESULTS

4-1 Forced Vibration Tests

The test frame was subjected to vibration in direction of the X, Y and D axes. For each direction, vibration was given under two different load conditions: one under eccentric moment of 20 kg·m and the other under 200 kg·m. The change of eccentric moments was effected by changing the eccentric mass of the vibration generator.

Under the vibration in direction of the X and D axes, the deflection of panel fastenings enabled the panels to cope with the frame deflection as far as the storey deflection was small. When the deflection became great, it was absorbed by rocking deflection of column-covering panels as intended by the design, and the resonant vibration occurred at different frequencies.

The resonant vibration curves and modes of slab deflection for each case are shown in Fig. 13 and Fig. 14 respectively. Under the vibration acting along the X and D axes, precast concrete panels and steel windows help increase the stiffness of the test model, causing torsional vibration. As can be seen in Fig. 14, the slab tended to rotate about the point near the opposite side on which the precast panels are located when the frequency was 1.9 Hz; however, when the frequency was increased to 3.5 Hz, the center of rotation shifted close to the wall panel side. When the vibration was applied along the Y axis or, in other words, in direction outside the plane of concrete panels, the deflection of precast concrete panels followed the frame deflection as if they had been pin-connected at top and bottom. Thus, resonance occurred at the identical point both under M = 20 kg·m and M = 150 kg·m, no influence of wall cladding on the stiffness of the frame being recognized. Under this condition, resonant vibration was observed at two different points as anticipated from the computed values obtained for the equivalent two mass vibration system.

Vibration along the D axis was accompanied by torsion which resulted from the increased stiffness due to wall panels, and it did not necessarily caused the deflection in direction of the vibration. As the frequency was increased, resonances occurred in almost the same modes, though not exactly at the same frequencies, as under the vibration along the X and Y axes.

4-2 Tests by Free Vibration within Plane of Panels

Shown in Fig. 15 are the recorded wave patterns of free vibration and in Fig. 16, the load-deflection relationship as observed when the rope was tensioned. In Fig.16, a full line represents computed deflection of steel frame whose stiffness was assumed as not having been increased by the presence of precast panels. It can be known from these figures that, under static loading, precast concrete panels give almost no effects on the frame stiffness as far as the rotation angle of members is not more than 1/70.

4-3 Tests by giving Vibration within Plane of Wall during Dismantling of Cladding

Figs. 17 through 19 show resonance curves and deflection diagrams obtained by subjecting the test frame to vibration at various stages of cladding removal. The weight of the test frame stripped of the panel cladding was not equal to the total weight of the frame and wall panels minus the weight of panels because the weight of floor slabs varied depending on the stages of cladding removal. This, unfortunately, made it impossible to compare magnitudes of deflection before and after the dismantling of cladding under the exactly same conditions. However, the test result clearly indicates that vibration given within the plane of panel was accompanied by no torsion when the steel frame only was left.

5. BEHAVIORS OF PRECAST CONCRETE PANELS

Fig. 20 shows modes of deflection of precast concrete panels under vibration in directions of the X and Y axes. When the test frame was vibrated in direction of the X axis, joints between column-covering panels and beam-covering panels at the upper part of the frame remained undeflected even when considerable lateral deflection was caused to the frame. At the lower part of the panel, the joint was deflected by the rotation of column-covering panels. Further, it was observed that beam-covering panels were also caused to rotate slightly during the vibration tests.

Figs. 21 through 23 show the length of deflection of joints, the magnitude of deflection between the panels covering the upper column and those covering the lower column, and the extent of uplift of the column covering panels as observed at the base of the panels. As can be seen in these figures, the measured deflection was in fair agreement with theoretical values represented by full line curves which were computed from the theoretical modes of deflection of panels indicated in Fig. 24. Shown in Fig. 25 is uplift at the base of the column-covering panel during the test.

6. CONCLUSION

The test results are tabulated in Table 1. Fig. 26 indicates the natural frequency of the test frame stripped of cladding, and also the natural frequency of the frame provided with cladding as obtained on the basis of a computed frame stiffness in which the stiffness-increasing effect of panels was disregarded.

Vibration in direction of the X and D axes was accompanied by some torsional vibration which was caused by the stiffening effects of panels and steel windows. Under the vibration in direction of the Y axis, the presence of cladding did not help increase the stiffness of the steel frame and such vibration was found to be identical with the deflection computed by assuming that the steel frame alone had been subjected to vibration.

Under vibration in direction of the D axis, the resonances similar to those observed under vibration along the X and Y axes were found to occur one after another, each time being accompanied by some torsion.

Deflection of precast concrete panels was absorbed by metal fastenings as long as the deflection was insignificant. When the deflection became great, the entire panel system tended, reaching resonances at different frequencies, to cope with the frame deflection as was intended by the structural design.

Generally, the damping constants became smaller as the resonant frequencies became higher, and the constants for the vibration along the X and D axes were found to be two to three times as large as those for the vibration along the Y axis.

The metal fasteners proposed for actual use were found to be capable of coping with the frame deflection under the vibration in all three directions, and they were proved by the tests to meet the deflection requirements as prescribed by the structural design.

7. ACKNOWLEDGEMENT

The authors would not have been able to carry out the tests reported in this paper with satisfactory results without kind assistance given by the representatives of the Sanwa Bank, Limited, the owner of the building and helpful suggestions rendered by Prof. Hajime Umemura of Tokyo University. To them, the authors wish to express their sincere thanks. The authors are also grateful to Messrs. S. Watanabe, S. Shimaguchi and N. Konoue of Ohbayashi-Gumi, Ltd. for their assistance during the tests.

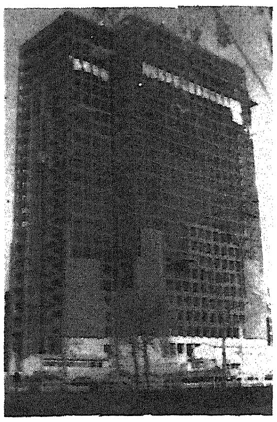

Fig. 1 Overall View

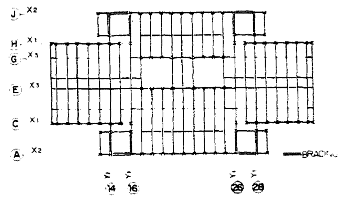

Fig. 2 Beam Plan

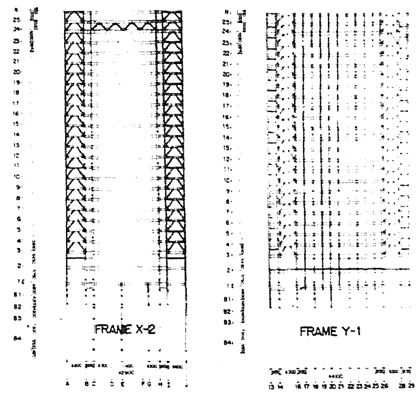

Fig. 3 Framing Elevations

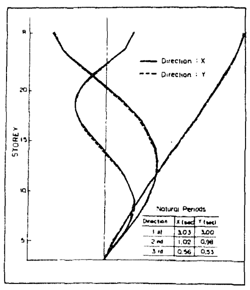

Fig. 4 Natural Periods

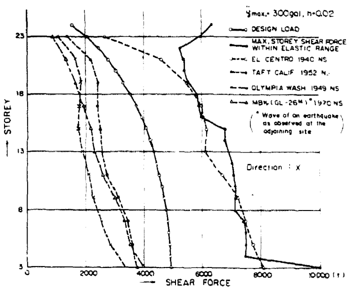

Fig. 5 Design Load

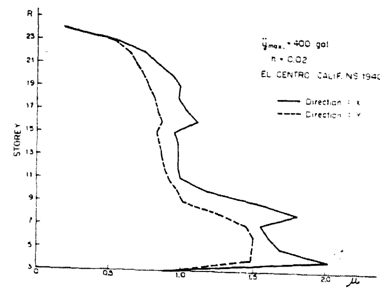

Fig. 6 Elastic Plastic Response Analysis

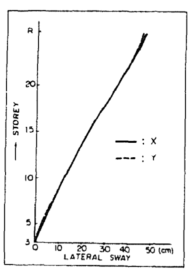

Fig. 7 Lateral Sway under Design Load

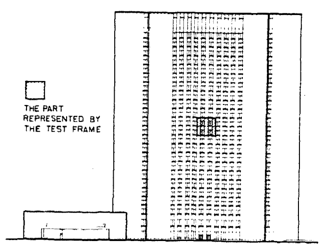

Fig. 8 North Elevation

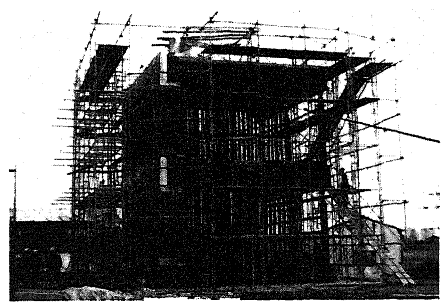

Fig. 9 Test Model

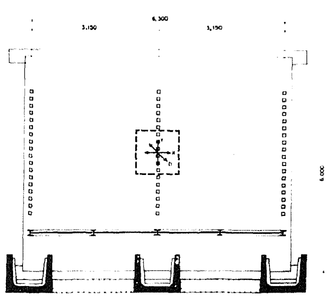

Fig. 10 Plan

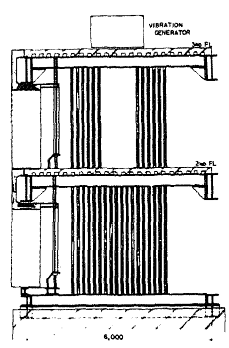

Fig. 11 Section

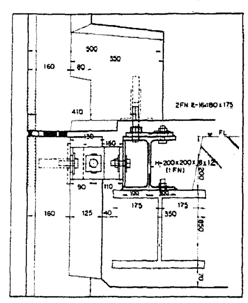

Fig. 12 Column Panel Fastening

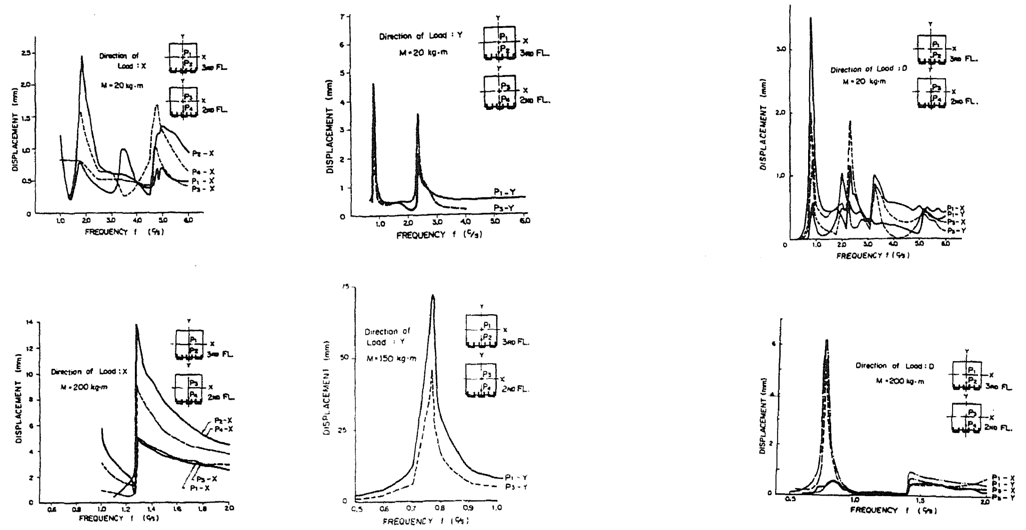

Fig. 13 Resonance Curves

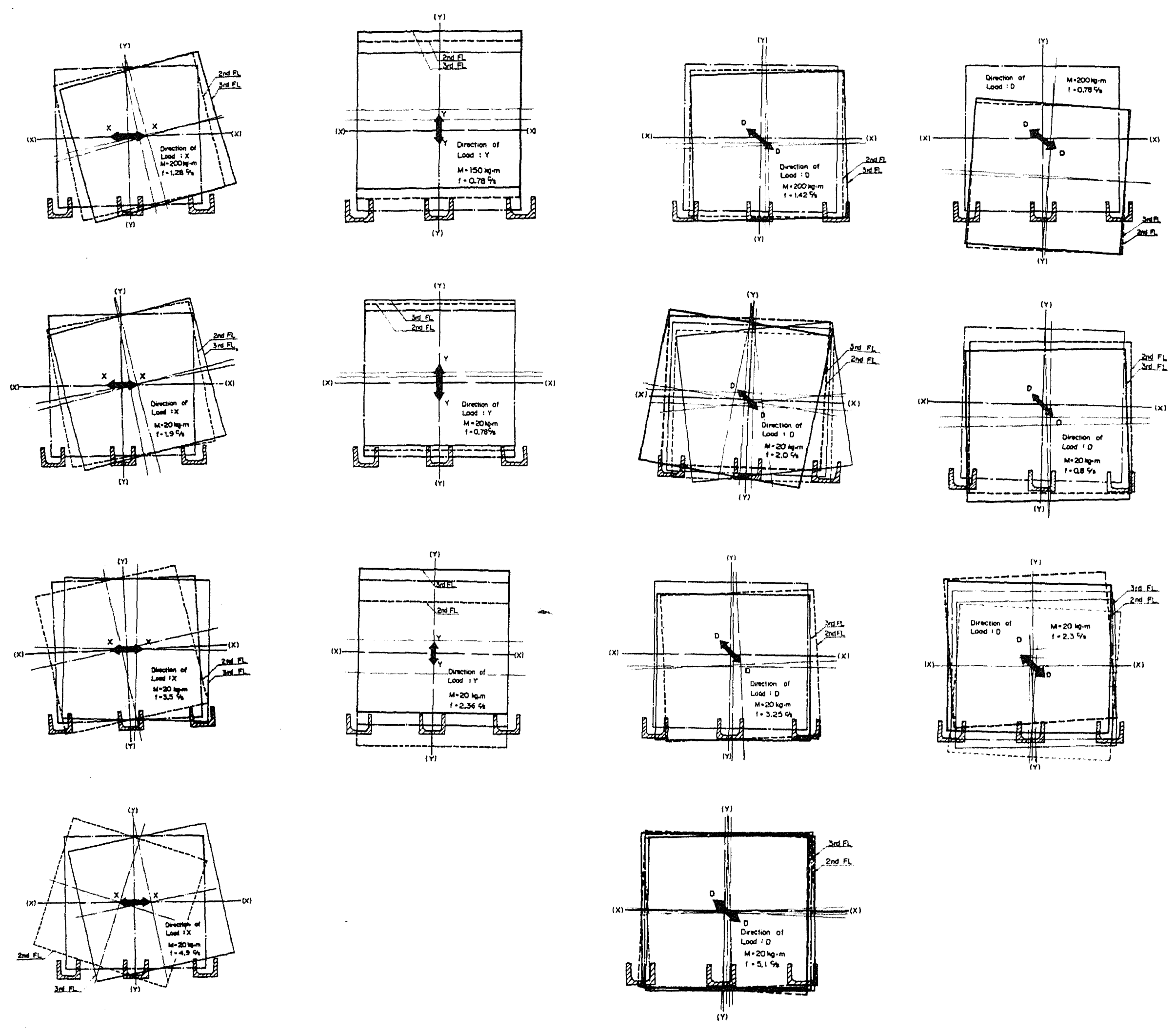

Fig. 14 Modes of Deflection

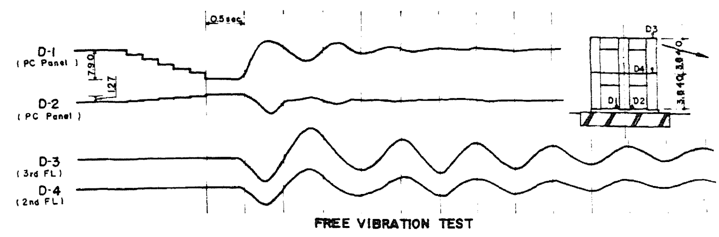

Fig. 15 Results of Free Vibration

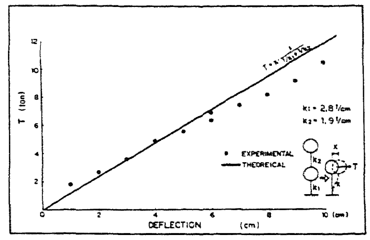

Fig. 16 Load-Deflection Diagram

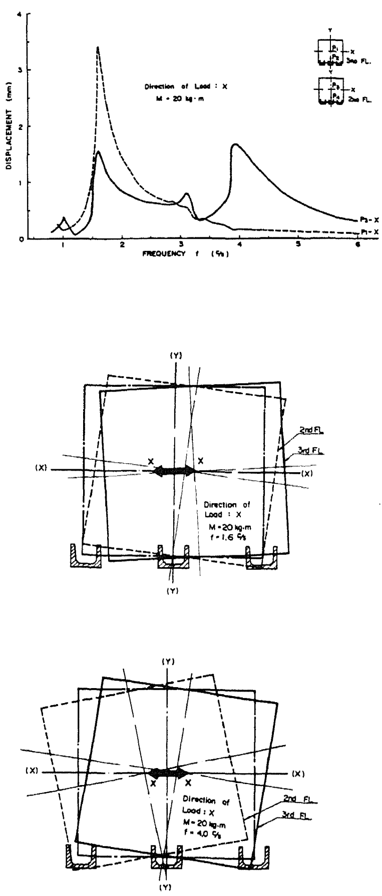

Fig. 17 Test at Stage 1

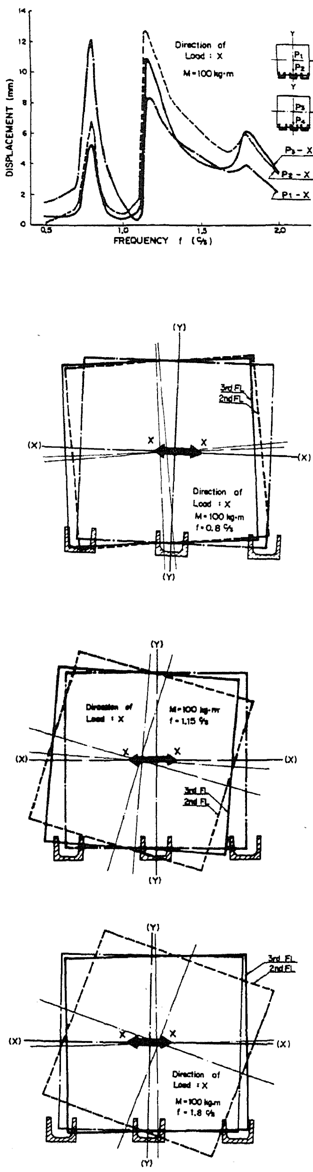

Fig. 18 Test at Stage 2

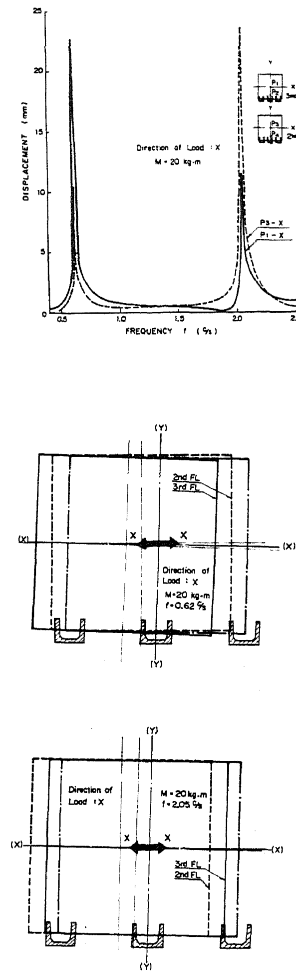

Fig. 19 Test at Stage 3

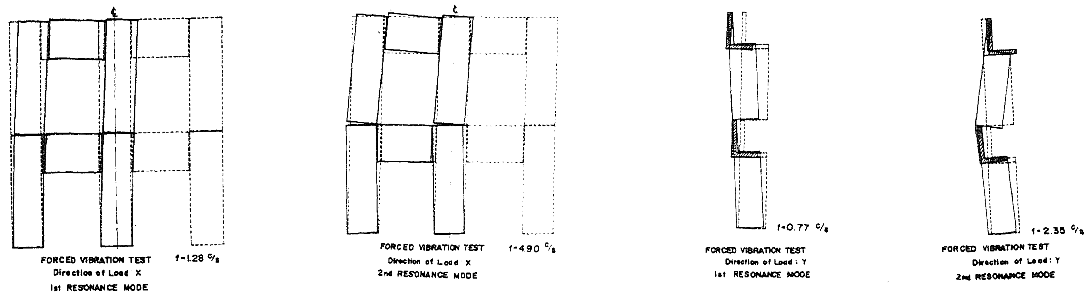

Fig. 20 Modes of Deflection of Panels

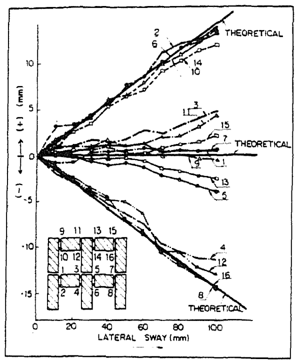

Fig. 21 Joint between Beam and Column Panels

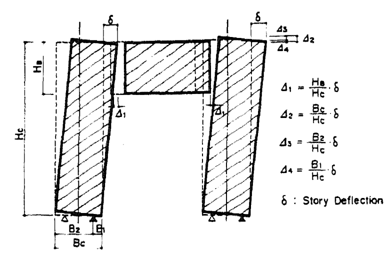

Fig. 24 Theoretical Mode of Deflection of Panels

Fig. 25 Column Base during Test

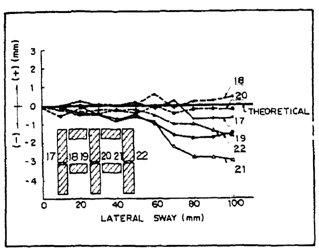

Fig. 22 Joint between Column and Column Panels

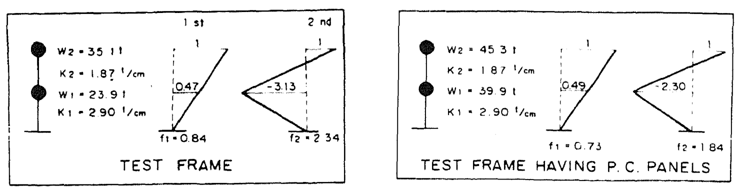

Fig. 26 Theoretical Values

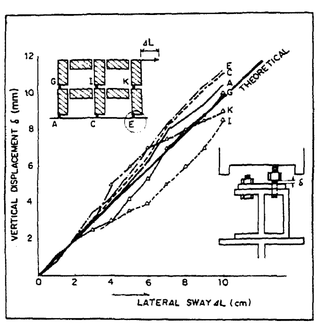

Fig. 23 Displacement of Column Panel

Table 1 Summary of Test Results

DIRECTION OF LOAD	FREQUENCY (c/s)	DAMPING COEFF. (%)	M (kg·m)	MODE
X	1.28	9.0	200	1st (X) + Torsion
	1.90	7.6	20	1st (X) + Torsion
	3.50	6.9	20	2nd (X) + Torsion
	4.90	5.8	20	1st (X) + Torsion
Y	0.78	1.6	150	1st (Y)
	0.78	4.3	20	1st (Y)
	2.36	1.8	20	2nd (Y)
D	0.78	1.8	200	1st (Y) + Torsion
	0.80	7.8	20	1st (Y) + Torsion
	1.42	—	200	1st (D)
	2.0	6.0	20	1st (X) + Torsion
	2.3	2.7	20	2nd (Y) + Torsion
	3.25	6.2	20	2nd (X) + Torsion
	5.1	5.9	20	1st (X), 2nd (Y)