On the Aseismicity of Precast Concrete Curtain Wall

By Seiji Watanabe and
Shozaburo Shimaguchi

Precast concrete curtain wall having comparatively large weight has recently been adopted as a member of external wall of a middle- or high-story building, and its aseismicity for expected severe earthquake is required. The problems about the panel of external wall essentially come to the following three points: namely, 1) the strength and rigidity of material of external wall itself, 2) the installing and supporting methods to the construction body and the adaptability as a secondary member, and 3) the effect due to installation of member of external wall on the vibration characteristics of building. Taking these points into consideration combinedly, a dynamic aseismicity test was carried out for an actual-sized panel by constructing a frame on an electro-hydraulic shaking table. As a result, it was clarified that the installing and supporting methods are important for the aseismicity of external wall member, and the installing method determines whether or not the panel acts only as a load or it acts as a body of combined vibration and further presents a damping action. On the basis of these results, the authors attempted to pursue the effects on the dynamic response of the external wall member of precast concrete curtain wall as well as the possibility of utilization for extra-high-story building etc.

ON THE ASEISMICITY OF PRECAST CONCRETE CURTAIN WALL

By

Seiji Watanabe[(I)] and Shozaburo Shimaguchi[(II)]

SYNOPSIS

A precast concrete curtain wall has recently been adopted as the member of the external wall of a middle or high-story building. There are following problems in the aseismicity about the precast curtain walls;

1) The aseismicity of a curtain wall itself.
2) The installing and supporting method to the construction and its safety.
3) The effect due to installation of the member of a external wall on the dynamic properties of the building.

To clarify these points, we carried out the dynamic tests of full-scale panels set in the frame which was fixed on a shaking table. From these tests results, we discuss the damping effects of panels on the construction.

1. HOW TO CONSIDER THE INTERLAYER DISPLACEMENT FOR EARTHQUAKE

As the criterion of the aseismicity of external wall member, let us first consider the allowable value of the interlayer displacement at the time of earthquake. What are generally considered nowadays in the aseismic design of the secondary members of curtain wall are as follows;

For an earthquake of middle intensity frequently experienced, any damage or any disturbance of individual panel is not permissible, but for a big earthquake it is usually considered that, although a fatal damage such as collapse is not allowed, the occurrence of cracks to a slight extent may be allowable. From such a viewpoint, the external members of curtain wall must not be damaged at all when subjected to interlayer displacement up to about 1/300 for earthquake shock. Moreover, the main parts of the member should not be damaged or fallen away by interlayer displacement of about 1/150.

From these considerations, the amount of interlayer displacement caused by earthquake must be within 8-12 mm for reinforced concrete buildings and within 15-25 mm for all-steel buildings.

2. TESTING APPARATUS

As shown in Fig. 1 and 2, mild steel plate springs, the number of which is six on one side and twelve on both sides, are fixed on an electro-hydraulic shaking table of large type (4,366 x 3,166 mm) and a slab with steel enforcement (2,500 x 3,500 x 300 mm) is supported at the upper ends of the plate springs.

As the plate springs which is fixed on the shaking table are allowed to displace only in the direction of the long side of slab, the vibration of the slab is limited to occur only in the direction of the long side of the slab.

(I) Ohbayashi-gumi Engineering Research Laboratory, Chief of Structural Dynamics Sec.

(II) Ohbayashi-gumi Engineering Research Laboratory

For the member of a curtain wall lateral displacements are generally considered as a problem, so that a panel under test is set parallel to the longitudinal side of a slab. At the same time to examine the transverse displacements another test piece is mounted in the direction at right angles to the longitudinal side of the slab.

The vibration characteristics of the testing apparatus are as follows;

i) The electro-hydraulic large type shaking table has the characteristics such that the ampitude is _10 cm up to 0-0.5 c/s, 30 cm up to 0.5 - 5.0 c/s and constant up to 5 - 50 c/s.

ii)	Total weight of frame: Total weight of concrete: Total weight of steel material:	8,581.6 kg. 2,844.0 kg. 5,737.6 kg.
iii)	Weight of upper slab:	3,748.0 kg.
iv)	Mass: $m = W/g$	4.97 kg sec^2/cm.
v)	Spring constant of plate spring:	$k = 244.0$ kg/cm.
vi)	Proper circular frequency:	$w = k/m = 5.38$ rad/sec.
vii)	Proper frequency:	$f = w/2 = 0.856$ c/s.
viii)	Proper period:	$T = 1/f = 1.17$ sec.

The resonance curve of the frame is shown in Fig. 3, from which the measured natural frequency is obrained as f = 1.1 c.s.

3. TEST PIECE

As test pieces the shock beton panels of window type and wall type of different shapes and dimensions were used.

A couple of the test pieces are mounted on the both sides so as to have a load without unbalance. The test pieces and their mounted states are shown in Figs. 4-10.

4. TESTING METHOD

The test was made by the following two methods;

1) To obtain the frequency chatacteristic, we varied the frequency with the displacement of the shaking table constant.

2) To obtain the large interlayer displacement necessary to check the strength of the panel itself, with the frequency of the shaking table fixed at the vicinity of the resonance, we varied the forced displacement so that the interlayer displacement was increased.

5. <u>TEST RESULTS</u> (Description of vibration recording)

From the results of the shaking table test mentioned above for the system of the frame-precast curtain wall the following facts were clarified:

I. On the aseismicity of panel itself;

Since the panel is able to endure a considerably large displacement after occurrence of cracks, destructive phenomena of the panel itself were not observed.

II. On the mounting method of panel;

In the case of the mounting method shown in the mounting diagram of the C type panel, the bolts were broken down by fracture. In other types a large displacement caused no fault except slip.

III. On the dynamic characteristics obtained from the shaking table experiment;

In Figs. 11 and 12 are shown the examples of the acceleration response of the frame and panel in the cases of mounting the A type and the B type panel, and in Fig. 13 and 14 are shown the strain at each point of panel.

Fig. 11. Acceleration response when A type panel is mounted.
Fig. 12. Acceleration response when B type panel is mounted.
Fig. 13. Strain response at each point of A type panel.
Fig. 14. Strain response at each point of B type panel.
Fig. 15. Distribution of strain of A type panel.
Fig. 16. Distribution of strain of B type panel.
Fig. 17. Dynamic hysterisis loop when A type panel is mounted.
Fig. 18. Dynamic hysterisis loop when B type panel is mounted.
Fig. 19. Measured point of panel.

6. <u>DISCUSSION OF RESULTS</u>

In the acceleration responses shown in Fig. 11, Fig. 12 and the strain responses shown in Fig. 13, Fig. 14 the responses decrease abruptly in the vicinity of the resonace point. It indicates in the case of the strain, the decrease of the external force applied to the panel, and about entire frame, the application of some other external forces. As the reason of these appearances, we can infer the slip will be caused by the supporting parts of panel the top of which was tighten by bolts in the slit holes. In other words, some action like coulomb friction in the part of bolts are caused.

According to this observation, we could say, the force to cause the slip in the vicinity of the resonance point corresponds to the first peak of strain response curve, and an equivalent coulomb friction caused in the slit holes work as the external force to give the smallet strain to panel through the top supporting point.

Then the calculation of these values in the point of static view with strain are given Table 2.

(Table 2)

From Table 2, it is seen that interlayer displacement about Type A and Type B causing the slip, shows equal value in each panels, but size of Coulomb friction shows different value by the rigidity of each panel. The strain responses are shown in Fig. 13, Fig. 14. It is a problem to be notice that Type B in Fig. 14 shows the distribution similar to crack occurrence. About the panel Type C which have smaller value of moment of intertia and smaller reinforcement ratio, crack is occurred before the slip appeared and as the panel, after crack occurrence, can follow large deformation, there is no slip to be observed on the same test piece. By these observation, it is able to consider that the panels where crack does not appear against the external force by causing the slip at the part of the support give large Coulomb friction force to the construction. Being difficult to explain the phenomenon that responses become larger over the resonance point by the reason that the supporting part of the panel is fixed again once the slip occurred, now we consider about the case that the relative movement between the frame and the panel is over the slip space, and the case that frame and panel begin to move separately. About those problems, we'll try to discuss after for they would be appear as the different problems in the real construction. It is the most important point that in Type A and B, there caused the slip in the supporting parts at large transformation which is about 7-8 m/m in interlayer displacement. (and the force is 1.3-1.5 t.)

According to this experimental result, when supporting methods are slip-occurred type, the panels which are designed so as to protect the crack up to 7-8 m/m interlayer displacement bear the large earthquake considerable. Then about Coulomb friction caused by the slip we'll try to discuss about how effective to the real construction.

7. METHOD OF THEORETICAL ANALYSIS

As the first step of the application of the results of this experiment to an actual construction, we have simulated the test construction by the following mechanical model as shown in Fig. 21.

The differential equations of this system are as follows;

$$m_1(\ddot{x}_1+\ddot{y}_0)+C_1\dot{x}_1+C_3\dot{x}_3+k_1x_1+k_3x_3\pm C_3'=0$$

$$m_2(\ddot{x}_2+\ddot{y}_0)+C_2\dot{x}_2+C_3\dot{x}_3+k_2x_2-k_3x_3\pm C_3'=0$$

In which k_1, k_2, k_3 are spring constants and C_1, C_2, C_3 are viscous damping coefficient and $\pm C_3'$ is a Coulomb friction force which appears in the range $|x_3|>x_r$, and the sign agree with the sign of the velocity $\dot{x}_3$. x_1 is a relative displacement between a base and mass m_1 , and x_2 is a relative displacement between a base and mass m_2, and x_3 is a relative displacement between mass m_1 and mass m_2 .

The value of the equivalent Coulomb friction force in this experiment is largest in Type A and slip occur at 7-8 mm displacement. The solutions are given useing analog computer through the conditions that the value of the equivalent Coulomb friction force is 390 kg/1 panel and the slip occur at 7-8 mm displacement. Fig. 22 shows a graphical representation of the response of the frame m_1 .

Fig. 20, which was given by J. P. Den Hartog, shows the magnification factor of displacement of a single degree of freedom system with combined viscous and Coulomb damping, where parameter x_f/a is the ratio of the Coulomb friction force to the amplitude of a harmonic external force.

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(1) Den Hartog, J.P.: Forced vibrations with combined viscous and Coulomb damping. Pil. Mag., ser. 7, vol. 9, p. 801, (1930)

(2) Jacobsen, L.S.: *Steady forced vibration as influenced by damping.* Trans. ASME, APM-52-15, (1930)

(3) Den Hartog, J.P.: Forced vibrations with combined Coulomb and viscous friction. Trans. ASME, APM-59-9, (1931)

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Tentatively, as the measure of damping effect of Coulomb friction force on the response of the structural frame, we use the parameter x_f/a of the above mentioned single degree of freedom system. In our case, the values of parameter x_f/a at resonance are given as Table 3. In Table 3, these values are comparatively large, hence the damping effect of Coulomb friction force from the panel on the response of the frame may be great.

However these values of parameter x_f/a is the values for the comparatively light experimental model frame. On the other hand lateral seismic force to full scal structure is very large, and the parameter x_f/a being small, damping effect on real frame during earthquakes may not be expected. However such a treatment for the damping effect of panel-frame Coulomb friction force on the real structural frame is very questionable.

In the damage investigation of the Tokachi Earthquake (1968) there are no damage in only a part including the precast concrete curtain wall of a three-story school construction (in Hachinohe). It is difficult to solve these problems but that will be developed by dynamical view.

8. CONCLUSION

From the results of this experiment, following aspects about the problems of the aseismicity of a precast curtain wall have been clarified;

1) The panel itself is not caused the large damage after crack has occured.

2) The supporting methods which have a slip mode are the effective methods to reduce the response of a panels at the vicinity of a resonance frequency.

3) In the case of the design of the panel itself, it is desirable that the panel should be reinforced by placing the effective arrangement of bars on the parts of column so that the panel could endure up to 7-8 mm interlayer displacement without any crack.

4) It may be recommendable that for soft type buildings supporting parts of a curtain wall have a mechanism of slip with Coulomb friction while for rigid type buildings mechanism of slip without friction.

There having been no reasonable codes about the supporting methods of a curtain wall, we hope our work will serve the decision of the supporting method.

9. ACKNOWLEDGMENT

The autors wish to thank Mr. Morohashi of the Shook Beton Co. for his assiduous cooperation in carrying out the present experiment.

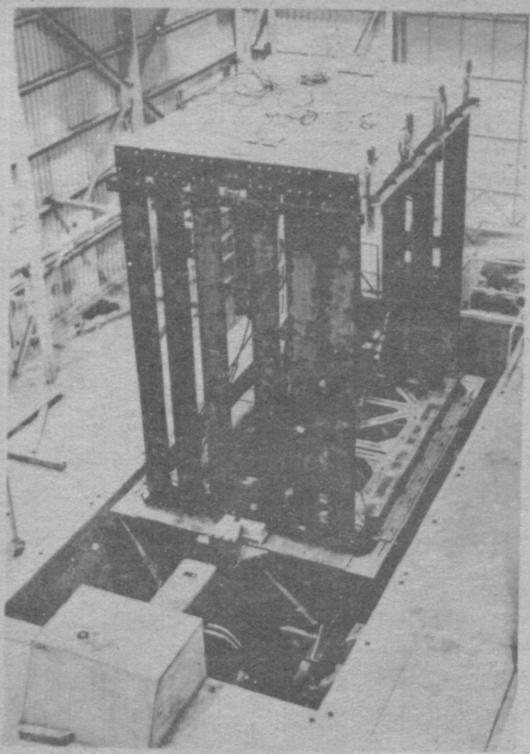

Fig-1 TESTING APPARATUS

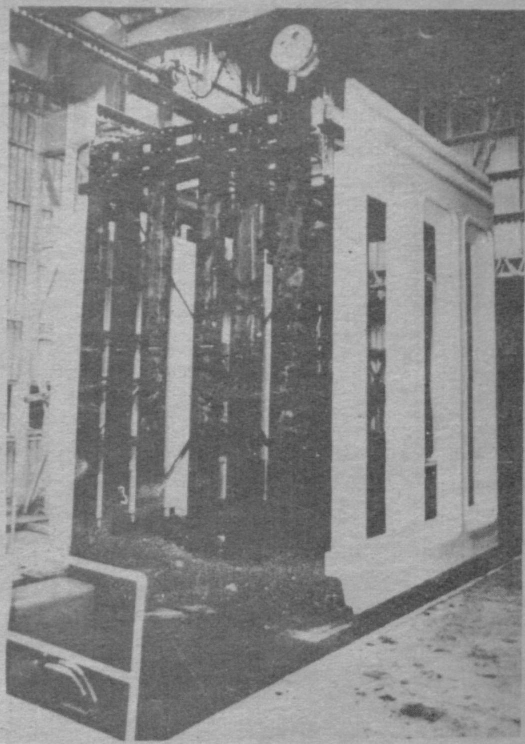

Fig-2 TESTING APPARATUS

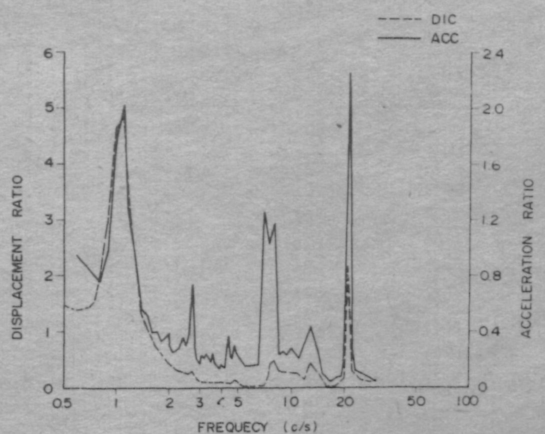

Fig-3 THE RESONANCE CURVE
OF THE FRAME

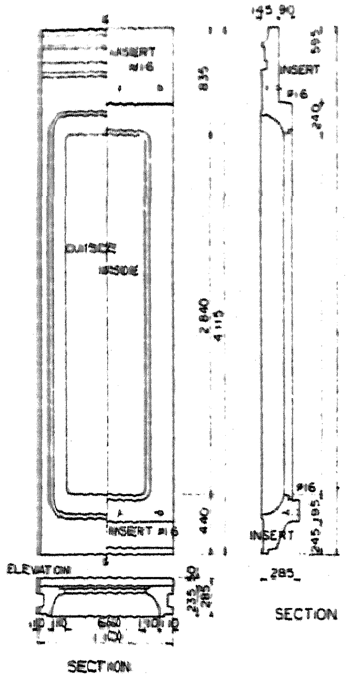

Fig-4 A TYPE PANEL

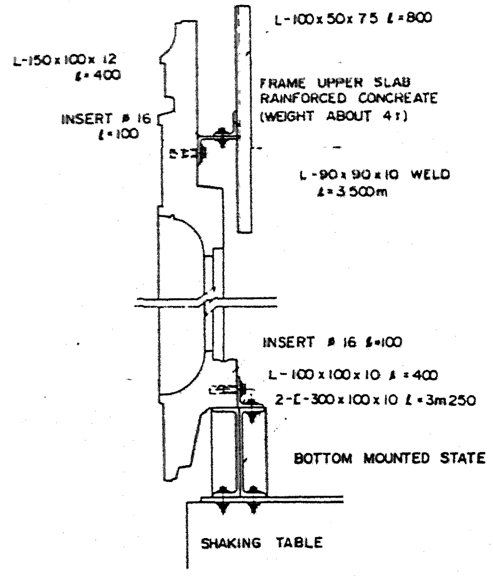

Fig-5 A TYPE PANEL MOUNTED STATE

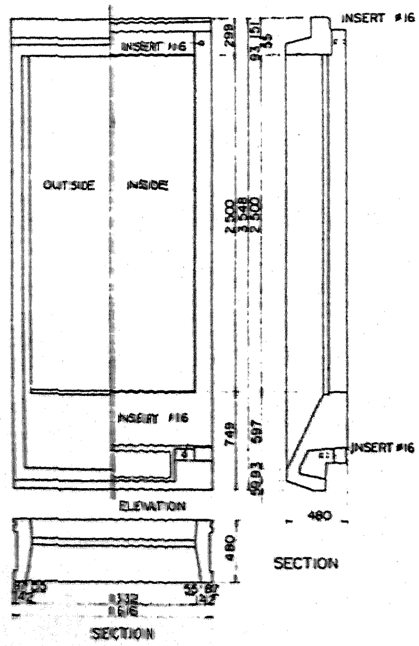

Fig-6 B TYPE PANEL

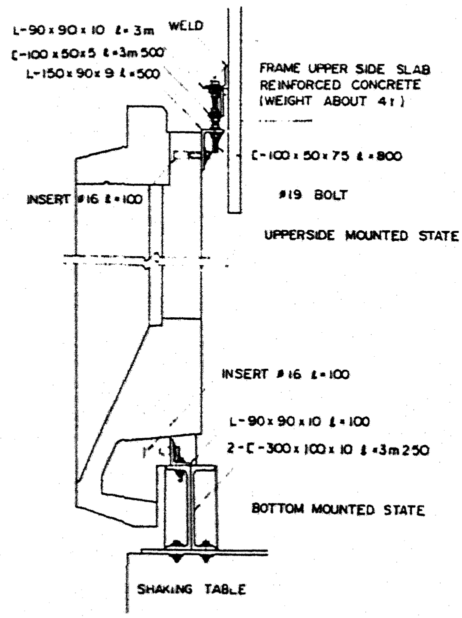

Fig-7 B TYPE PANEL MOUNTED STATE

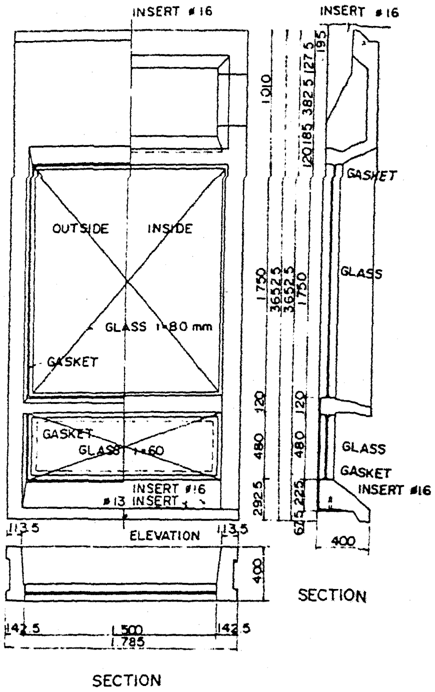

Fig-8 C TYPE PANEL

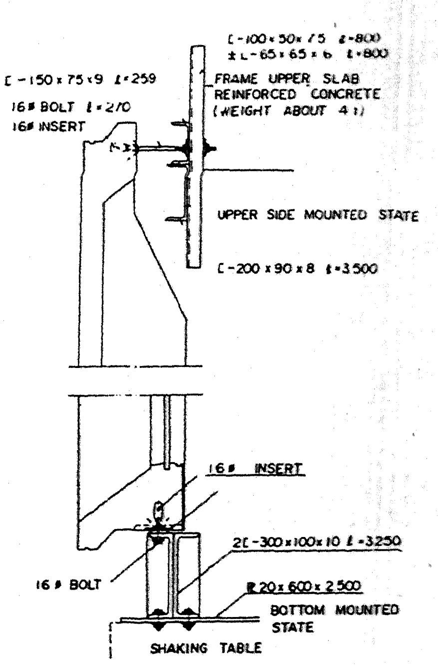

Fig-9 C TYPE MOUNTED STATE

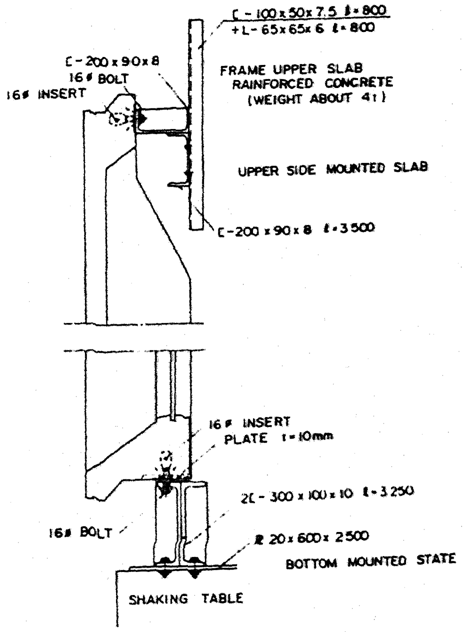

Fig-10 C TYPE PANEL MOUNTED
STATE 2

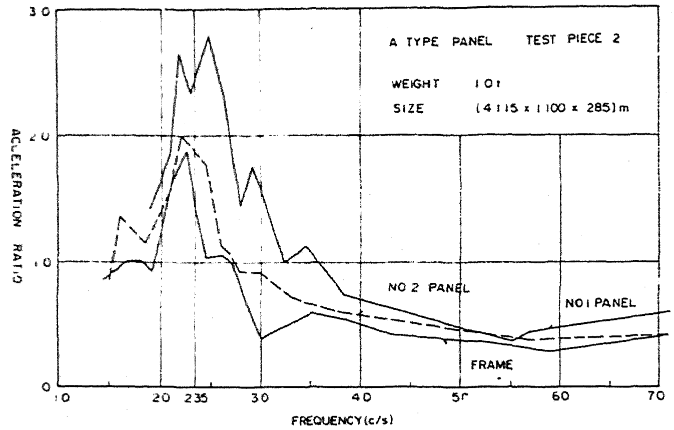

Fig-11 ACCELERATION RESPONSE WHEN A TYPE PANEL MOUNTING IS MADE

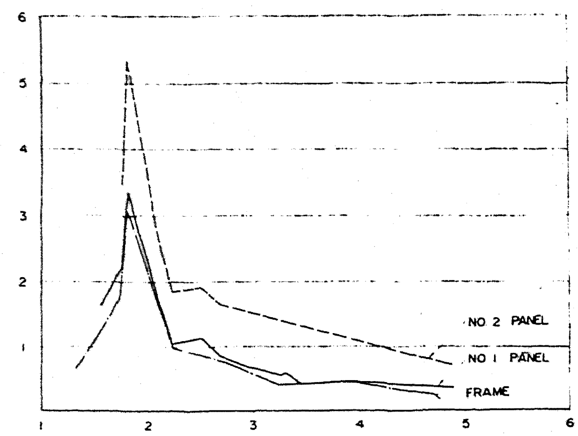

Fig-12 ACCELERATION RESPONSE WHEN B TYPE PANEL MOUNTING IS MADE

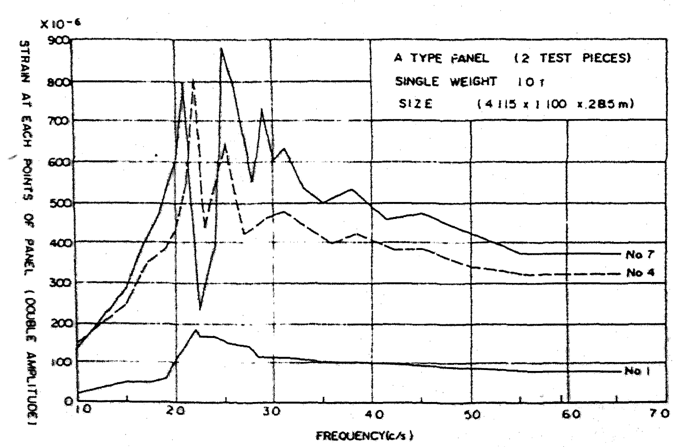

Fig-13 STRAIN RESPONSE AT EACH POINT OF A TYPE PANEL

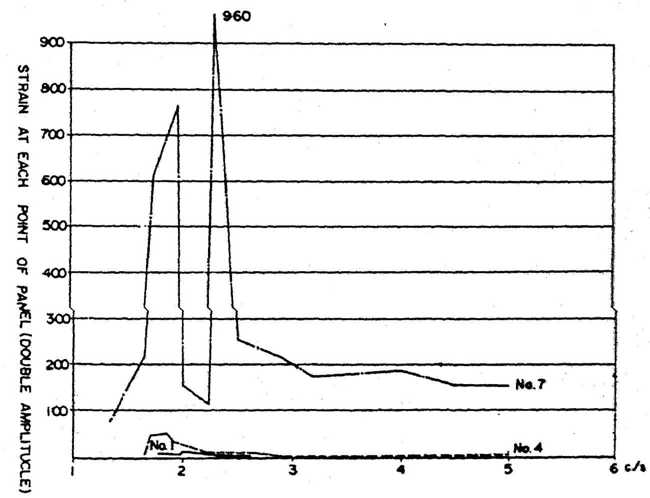

Fig-14 STRAIN RESPONSE AT EACH POINT OF B TYPE PANEL

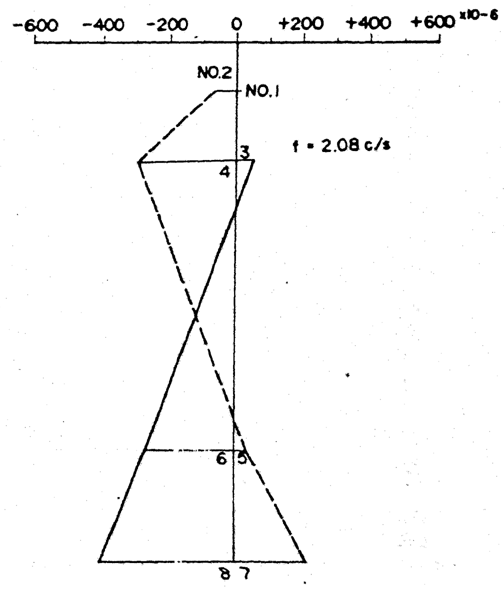

Fig - 15 DISTRIBUTION OF STRAIN OF A TYPE PANEL

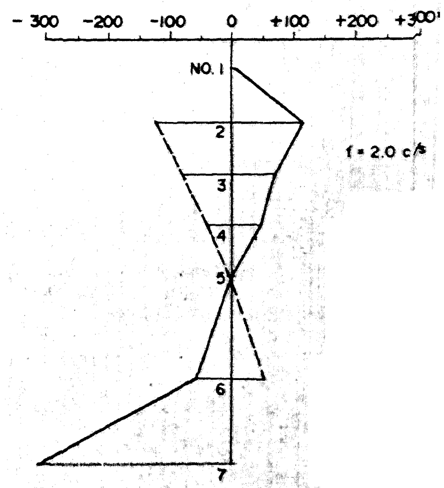

Fig-16 DISTRIBUTION OF STRAIN OF B TYPE PANEL

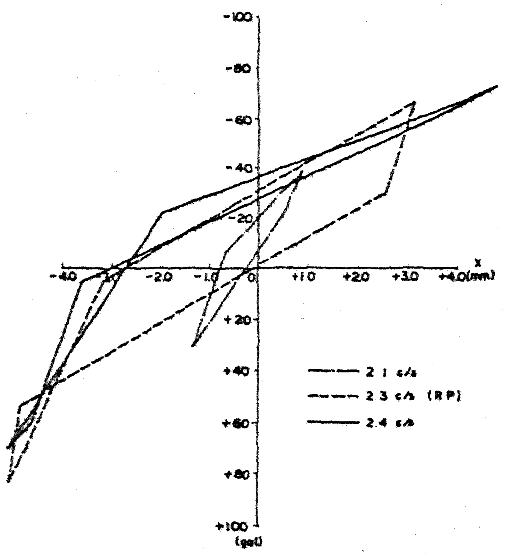

Fig-17 HYSTERESIS LOOPS OF A TYPE PANEL

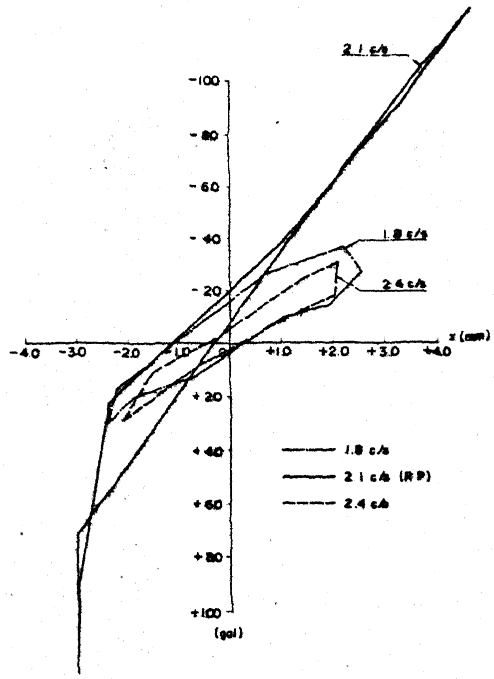

Fig-18 HYSTERESIS LOOPS OF B TYPE PANEL

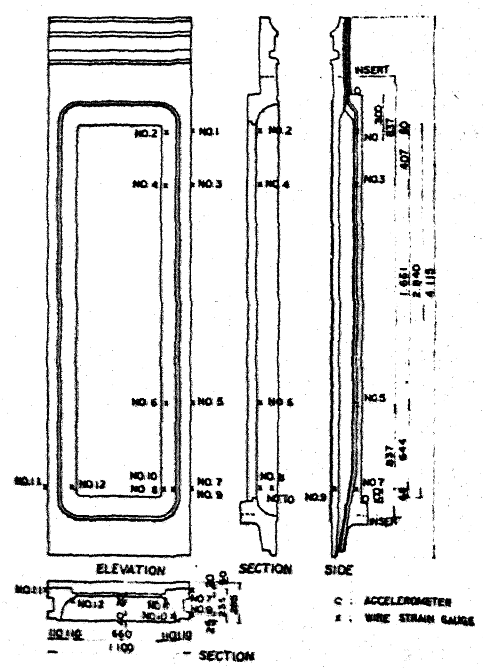

Fig-19 MEASUREMENT POINTS OF A TYPE PANEL

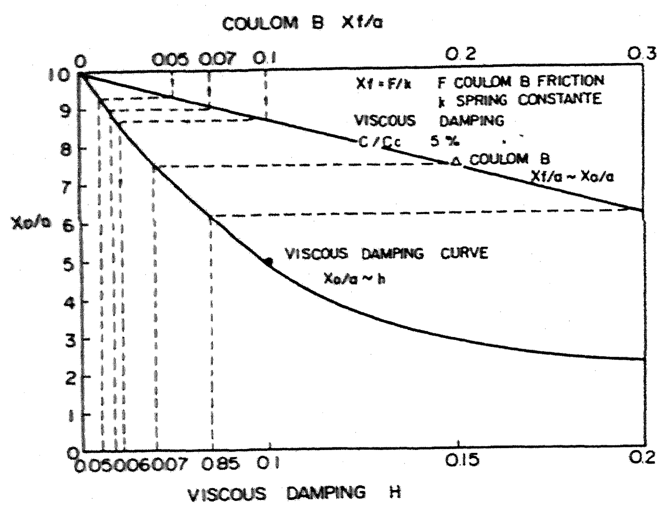

Fig 23

TYPE	SECTION	I cm^4	A cm^2	pt %
A	220, 220, 2-D13, D16	8054	297.3	1.33
B	480, D13, φ6, φ9, φ9, D13	5494	486.8	0.8
C	400, φ6, D13, D13, φ6, φ6, 142	4950	449.5	0.6

TABLE 1

	I cm^4	NO.7 STRAIN	EXT. FORCE OF PEAK VALVE P kg	TOP DISPLACEMENT PEAK VALVE δ cm	COULOMB FRICTION
A TYPE	8054 (1.33%)	400×10^{-6}	1280	0.721	390 kg
B TYPE	5484 (0.8%)	380x10	1400	0.785	270 kg
C TYPE	4950 (0.752)				

C TYPE IS NOT SLIP

TABLE 2

	T s/s	h	m	k kg/cm	a0 cm	a cm	Xf/a
FRAME ONLY	1 1	0.056~0.09	4 97	244	—	—	—
A TYPE PANEL MOUNT	2 35	0.092~0.135	5 99	1280	0 2	5 5	0 58
B TYPE PANEL MOUNT	2 14~2 95	0.116~0.123	6 34	1120	0 1	8 8	0 326

TABLE 3

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	W	k1	Xf = F/k	S	Xf/a
R	241	69 4	8 6/69 4=0.124	2 145	0.058
9	135 6	69 4	0.124	2 145	0.058
8	132 4	78 0	0 111	2 475	0.045
7	116 9	88 0	0 098	2 475	0.040
6	116 9	95 5	0 09	2 435	0 037
5	121 0	104 5	0 083	2 435	0 034
4	160 3	122 0	0 07	2 390	0 03
3	156 2	126 0	0 059	2 390	0.025
2	168 2	213 0	0 04	0 76	0 053
1	400 1	—	—	—	—

TABLE 4