A VIBRATION TEST OF A LARGE MODEL STEEL FRAME WITH PRECAST CONCRETE PANEL UNTIL FAILURE

by: R. TAMURA(I), M. MURAKAMI(II), Y. OSAWA(III), N. MIYAJIMA(I) and Y. TANAKA(I)

A 1/3 scale model of a steel frame with precast concrete panel has been tested by application of steady forced excitation using a vibrator of large capacity. The results of a vibration test of apartment houses using this type of structure (Figs. 1, 2 and 3), which was carried out prior to this model test, showed that the concrete panel played important role in the lateral rigidity of the structure during the vibration of small amplitude, and the question has arised for the dynamic behavior of this type of structure during larger amplitude like in case of destructive earthquakes. It was intended in this study to test the model taken out of this apartment building up to the point of failure to get useful information to answer the above-mentioned question. The test specimen was made of steel frames using H-shape and the precast concrete panel connected to the frame by welding and joint mortals (See Figs. 4 and 5).

A free vibration test, a horizontal alternative loading test as well as a forced vibration test were carried out. The test results are shown in Figs. 11, 12 and 13 by a resonance curve with corresponding load-deflection curves. Static and dynamic rigidities, natural period, maximum deflection, damping coefficient etc. are shown in Table 2. The test results are summarized below.

i) The restoring force characteristics varied from a linear spring type to a hard spring type according to the formation of cracks. Finally, the rigidity was suddenly reduced by a destruction of the part where the wall panel is connected and approached to that of the frame (See Fig. 10). Figs. 16 and 19 show the ultimate state of the specimen.

ii) Static and dynamic rigidities were reduced as exciting loads increased, and they coincide with each other at corresponding stages.

iii) The rigidity in the ultimate state was reduced to 1/30, compared with initial rigidity.

iv) The damping coefficient is about 2 to 3% of the critical value except in some case of the hard spring type in which it is 7 to 10%.

v) The load-deflection curves may represented by a linear line approximately at all stages, except in some transient state of the hard spring type.

It is apparent that the participation of concrete panel to the lateral rigidity of the building during small amplitude reduces the maximum deflection of the building considerably.

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Synopsis

This is a paper on a vibration test carried out on a large model until failure, to study the earthquake resistance of a steel frame apartment house constructed with precast concrete wall panels.

The restoring force characteristics move from a linear to a hard spring type according to formation of cracks. Finally, the rigidity is quickly reduced by a destruction of the part where the wall panel is attached. But, the frame again shows linear type rigidity.

The damping coefficient is about 2 to 3%, but in case of the hard spring type some are 7 to 10%.

§ 1 Introduction

This paper summarises results of a test until failure on a large experimental model to study earthquake resistance of a tall apartment building (Fig. 1) made of steel structure with precast concrete wall panels. The building under study is a seven-storied apartment building as shown in Fig. 2. The columns and beams are steel frame with fire proof covering, and precast panels are used as both the walls and the floors.

Vibration generator was installed on the seventh floor, and the results of the vibration tests made are shown in Fig. 3. The results show good agreements with the theoretical rigidity values, assuming both the steel frames and built-in precast concrete panels are exposed as an integral body, which was neglected in the former calculation sheet(1). Because this vibration test was confined to within small amplitude, the results obtained provided insufficient data on which to determine the dynamic behavior of the wall panels when exposed to a destructive earthquake.

In order to have a clarification on this problem, about 1/3 scale model taken out of a part of the apartment building with built-in precast concrete panels was constructed so as to explore the unanswered question by testing the simpler structure of the integrated body of the steel frame and precast concrete. The model was exposed to vibration force generator until the panel was partly fractured.

§ 2 Test Model

The test model has two-span steel frames in longitudinal direction and one-span steel frame in transverse direction with precast concrete panels in the center frame, as shown in Figs. 4, 5, 6, 7.

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Wide flange beams were used as both columns and beams; and the columns were furnished, as shown in Fig. 5, in such a way that concrete covering was provided on the columns, and lath mortar coating on the beams. The columns and beams were connected with high strength bolts in friction type. 80mm thick precast light weight concrete panels were used as the wall, as shown in Fig. 5, and wall and frame were connected by welding the plates anchored on the four corners of panel to the plates welded in corners of the frames.

For the sake of simplification in the present test, the foundation beam of the model was securely fixed onto the laboratory floor.

§ 3 Test Procedure

Vibrational characteristics were measured for each stage of forced vibrations in both transverse and longitudinal directions with different vibration generators in stalled on the roof slab.

On each stage, before and after the forced vibrations were applied, horizontal loading test and free vibration test were made to obtain rigidity, stress and dynamic characteristics.

An overall schedule of the experiments is shown in Table 1.

Forced vibration tests

According to the loading conditions, the vibration generators as given below installed and firmly fixed onto the roof slab, and vibrations were repeatedly generated and applied, and vibration force was exerted alternately in transverse and longitudinal directions:

Vibration Generator	Max. Force	Frequency	Maximum Moment of Inertia	No. of Axis
1 ton vibrator	1 ton	1-50	1,600 kg·cm	2
40 ton vibrator	40 ton	0.237-15	12,000 kg·cm	4
100 ton vibrator	100 ton	1-10	31,000 kg·cm	2

Since the capacities of the vibrators used did not exceed 15 c/s in vibration frequency, an improved glinder-type vibrator was used to study various higher cycle modes during the forced vibration with small amplitude. There are two methods of exerting forced vibrations: One is to increase frequency and repeat the vibrations at points close to the resonance period, while keeping the eccentric mass constant. The other is to exert a constant vibrating force by adjusting the eccentric distance corresponding to the vibrating frequency.

The following instruments were used for each measurements.

Smoked Drum (2 sets)

--- horizontal deflection at both ends of frame and center top

Displacement Meter with Differential Transformer (2 sets)

--- horizontal deflection at the center of the roof slab

6 Channel Electro Magnetic Seismometer (4 sets for horizontal - and 2 sets for vertical - measuring)

--- horizontal and vertical deflection at various points of the roof slab, foundation beam and of areas adjacent to the test model

Registance Wire type Accelometer (0.5g - 6 sets; 1g - 2 sets; 5g - 1 set; 10g - 2 sets)

--- acceleration in each direction at various points of roof slab, beam and foundation beam

Dynamic strain gauge

--- dynamic strains in columns and beams

*Static loading test

Horizontal static load is applied to the center of the roof slab by using 20-ton and 100-ton oil jacks respectively. Load applications were alternately and repeatedly made in the transverse direction. Proof ring was used for the load measurements. Dial gauges were installed at various points of columns, beams and slabs to measure horizontal deflections in various directions. Strain gauges were installed at various points to permit the measuring of strains at each point.

*Free vibration test

Free vibration was initiated through the forced deformation which was caused by pulling the roof horizontally with wire rope. Notched steel bars inserted in the middle of wire ropes were used to determine the force given. Notched bars were designed to be cut off instantly at specified load (5 tons, 10 tons and 20 tons). 20-ton chain block was used for pulling the wire rope. The U-notched steel bars were made of high-carbon steel (S50C) to permit instant breaking off without excessive elongation, and the shapes of notches were made out to correspond to the specified loads.

The measurement method was used in this test as used in the forced vibration test.

§ 4 Test Results

4-1 Static loading test

Figure 8 shows the load-deflection curves in various static loading tests, as arranged according to each test number on the same ordinate (loading). The scale of abscissa (deflection) is in 10/1 with S 73 to S 81, to permit indication of larger deflections.

Static rigidities show marked differences at each stage of the dynamic vibration tests.

The tests are divided into the following three stages.

Stage-I ----- S 4 - S 37
Stage-II ---- S 49 - S 64

Stage-III --- S 73 - S 81

Stage-I indicates linear restoring force characteristics, wherein the elastic rigidities corresponding to the rigidity decrease due to cracks, as well as the initial rigidity, are shown.

Stage-II shows conspicuous restoring force characteristics which are bi-linearly indicated as a compound of characteristics as respectively shown in Stage-I and Stage-III. While the primary rigidity is substantially low, the rigidity begins increasing at a certain loading value (4 tons). This indicates the characteristics of so-called hard spring.

Stage-III shows linear restoring force characteristics in which the rigidity is close to that of the frame after the wall joints breaks down.

4-2 Dynamic vibration test

Figure 9 shows the resonance curves of dynamic vibration tests; natural period on the abscissa and displacement on the ordinate. Full lines indicate curves in the loading from low cycle to high cycle (this will hereafter designated as "Up"), and dotted line indicate curves in opposite loading from high cycle to low cycle (this will be hereafter designated as "Down").

Results of dynamic tests are to be divided into three stages similar to those of static tests. Tests D 41- D 46, corresponding to Stage-I (S 4 - S 37) in the static loading test, show the elastic vibration characteristics, and Tests F 66 - D 72, corresponding to Stage-II (S 49 - S 64) in the static loading test, show elasto-plastic vibration characteristics. The resonance curves of Up and Down differ widely, and the natural periods increase substantially with repeated loading.

Tests D 77 - D 79, corresponding to Stage-III (S 73 - S 81), show that the natural periods are substantially longer, while the resonance curves are linearly indicated vibrations.

4-3 Summary

Figs. 11 - 13 are arranged to make easy comparisons between static characteristics and dynamic characteristics by classifying the test result in each stage and using the common ordinate of deflection and to correspond the load-deflection curves obtained by the static tests and resonance-deflection curves on the same diagrams.

Table 2 shows static and dynamic rigidity, maximum deflection, maximum acceleration, natural period, damping coefficient and rotation angle of member in major tests.

Serial numbers are used to identify the tests, and S represents static loading tests, D dynamic vibration tests, and F free vibration tests.

In the tests S 49 through S 64 indicating a hard spring type, the static rigidity is represented in two ways: primary and secondary rigidities.

Dynamic rigidity is given in $K_1 = 4\pi^2 \frac{m}{T^2}$ calculated from natural period and in $K_2 = \frac{m\alpha max}{\delta max}$ calculated from the acceleration and deflection.

The damping coefficient is given in the hysteresis damping obtained from hysteresis damping curve in static loading test and in dynamic damping ($h = 1/2 \frac{\Delta W}{W_0}$) calculated from resonance curve in dynamic loading test. The angle of rotation of member is calculated from the maximum deflection in the dynamic test.

1) Stage-I (S 4 - D 46)

Fig. 11 shows the data with 40-ton vibration generator with maximum vibrating force of 10 tons at 15 c/s. Decrease in rigidity was observed at about 6 tons as seen in the load-deflection curve, and the initial cracking in wall mortar began at this point. After the D 33 to 36 test (max. 15 c.p.s. - 2 ton), initial bending cracks were found in the column head, and cracks appeared on the four corners of wall panel in the direction of 45 degrees. Cracks in the wall mortar became gradually prominent thereafter.

The tests generally show linear restoring force characteristics, with some decrease in rigidity indicated which is attributable to the cracks.

Vibration in longitudinal direction in the test D 41 through D 46 shows elastic.

2) Stage-II (S 49 - D 72)

Fig. 12 shows the data with 100-ton vibrator actuating constant 3-ton vibrating force in transverse direction. These groups show hard spring restoring force characteristics. As vibration was repeated, overall rigidity became decreasing, keeping same hard spring type characteristics. Shape of the resonance curves shows a prominent change after the D 46 test. Especially, the resonance curves of loading from low cycle (Up) and loading from high cycle (Down) differs from each other and resonance peaks became shifting. The Down resonance curves failed to show clearcut peak and the natural periods decreased rapidly.

In the 1.5-ton constant loading test, D 60, the cracks in outer frame column concrete head became considerably and concrete fragments were observed coming off as the cracks became bigger or smaller during vibration (Fig. 14).

In the 3.0-ton constant loading, in D 72 test, part of wall mortars fell off and concrete fragments around the wall fixing plates came off. Moreover, the cracks of concrete at the bottom of a column in frame 2 became visibly wide (Fig. 15).

3) Stage-III (S 73 - F83)

This stage shows a transition from hard spring type of restoring force characteristics, as shown Fig. 13, due to the abrupt decrease in rigidity by the vibrational loading, to linear restoring force characteristics close to wall-free frame rigidity (wall was disregarded). In the D 79, nearly all the wall mortars dropped off and all the anchor steel bars of wall were torn down. Walls and frames behaved themselves in a utterly unrelated manner during the vibration. Cracks appeared all around the concrete bottom of column and the width of cracks exceeded 5mm. Fracture in the column bottom is shown in the Fig. 16.

F - 82 shows an instance where wall was provided, and F - 83 shows a case where wall was dispensed with. These results were almost entirely identical, and indicate that at this stage the wall had no bearing on them.

The final condition of cracks is shown in the Fig. 19.

Besides these tests, higher cycle vibration test with small amplitude were made by the glinder type vibrator to determine the natural periods, where the locking vibration, twist vibration and etc. in terms of cantilever rotational innertia were also determined.

4-4 Damping

Generally, the damping coefficients were about 2 to 3%, but in the case of hard spring type characteristics with small amplitude they showed relatively large value of approximately 7 to 10%. This is considered attributable to a slip of reinforced bars and mortars in the wall fixing parts that is liable to occur during vibration with small amplitude.

§ 5 Analysis

Figure 10 shows the change in rigidity of center frame in both the static and the dynamic tests. (Test numbers are represented by abscissa and the rigidity by ordinate.)

For the hard spring type characteristics in the tests S 49 through S 63, initial rigidities are indicated in the lower curve and secondary rigidities in the upper curve. Also the theoretical rigidity value is indicated by dotted line on the ordinate.

Initial rigidity represents the theoretical rigidity K_1 obtainable when wall and column are in an integral single unit. The secondary rigidity K_2 represents the case where the wall and the frame behave themselves independently from each other owing to the cracks in the mortars, and the wall is fixed to the frame on four corners only of the latter, thereby limiting their individual deformations. Also the rigidity K_3 represents the case where only the frames on both sides are significant elements, with the wall no longer having bearing on it as the wall to frame joints become complete failure.

Initial rigidity for S 4 agreed well with the theoretical value K_1. By increasing load from S 5 to S 8, the rigidities decreased gradually due to the cracks of the wall mortars. Especially, in dynamic tests D 22 through D 23, the rigidities became as close as K_2 due to the loose mortars. With further increase of load by using heavier vibration generator, the rigidities rapidly decreased and became close to K_3. This does not mean, however, that the wall rigidities were totally lost, but rather the rigidities of frame also decreased due to the bending cracks in the concrete, thereby reducing the overall rigidities to as close as K_3.

The static tests S 49 through 62 show hard spring type characteristics. The overall initial rigidity is close to K_3, and the secondary rigidity is close to K_2. This was because compressed side of concrete work recovered strength when the wall slipped to a certain extent during increased loading.

The rigidities decrease as the cracks widen and effective parts are lost as the tests are repeated.

In the Stage-III D 70 through D 79 tests, the rigidities were exclusively dominated by the frame rigidity and lost the support of wall since wall to frame joints became complete failure. It represents a linear spring type of characteristics. Also, the frame rigidities decreased with the increase of load due to an excess over the shear crack load of the column.

§ 6 Conclusion

As the result of past vibration tests with small amplitude on the apartment building, contribution of the wall to the rigidities, which are usually disregarded in the antiseismic design, was considered to be substantial. Accordingly, a large and simplified structural model taken out of the apartment building was built, and experiments were made to get useful information of rigidity reduction and the dynamic behavior until failure.

The vibrational tests revealed that the natural period in final stage is over seven times larger than the initial natural period. With respect to the restoring force characteristics, linear spring type characteristics appear initially, and turn into hard spring type characteristics due to the fracture of connections of wall and frame, and further, they become linear again corresponding to frame rigidity values.

The damping coefficient is about 3% in the linear region of both initial period and after the wall joints fracture, but it reaches nearly 10% occasionally in the hard spring characteristics region.

On the other hand, the experiments resembled a fatigue test since several ten thousands times of loading was repeated in the forced vibration test.

Damage to concrete parts caused by repeated vibrational loading was observed after the recent Tokachi-Oki earthquake in Japan. The number of repetitions of vibrational load during the earthquake is considered to be most important. It will be necessary in the future to make a comparative study between the data on such a genuine earthquake vibration and the results obtained in our tests.

Experiments are being continued to further investigate the relationship between a structure and the ground on which it rests, particularly to determine effects of vibrations on buildings erected on piled foundations and built in direct surface erection.

§ 7 Acknowledgement

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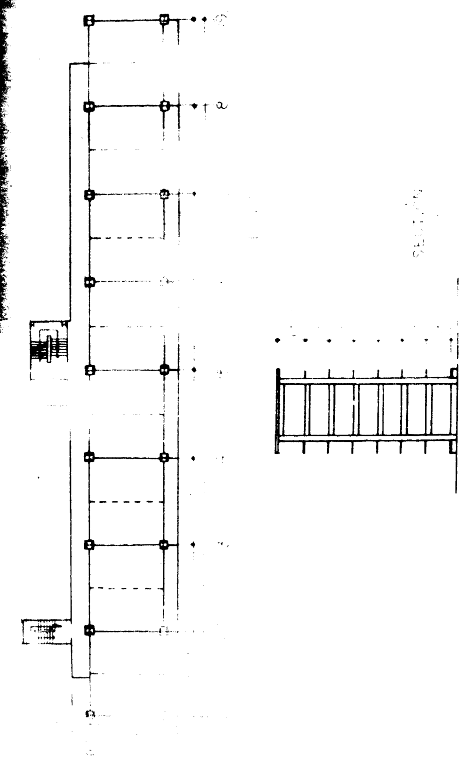


Fig 2

Fig

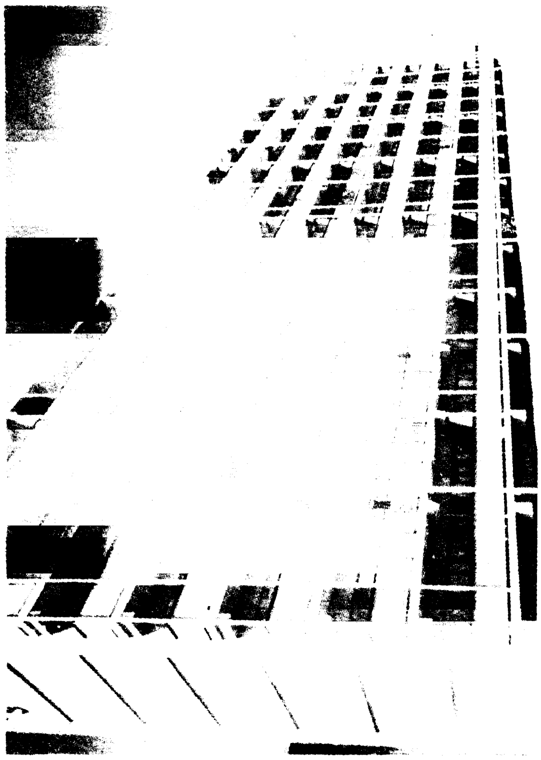


Fig 1

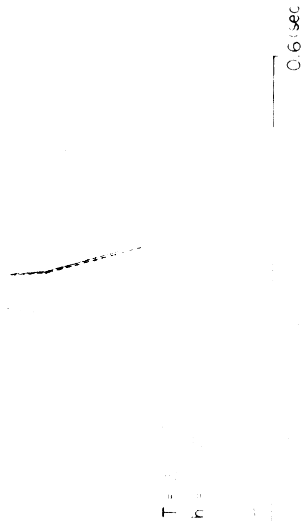


Fig I

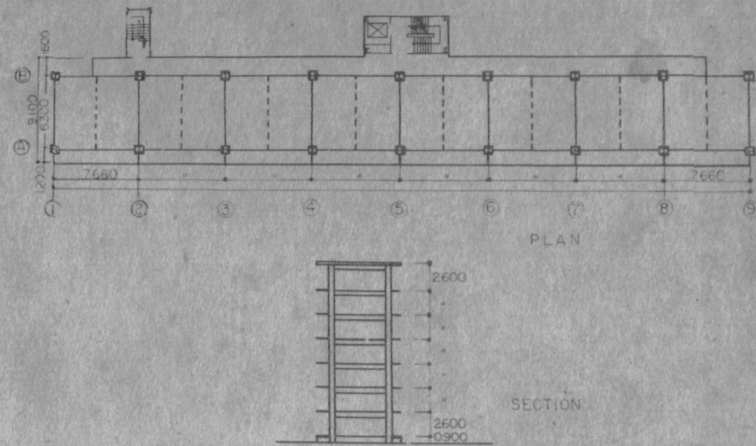
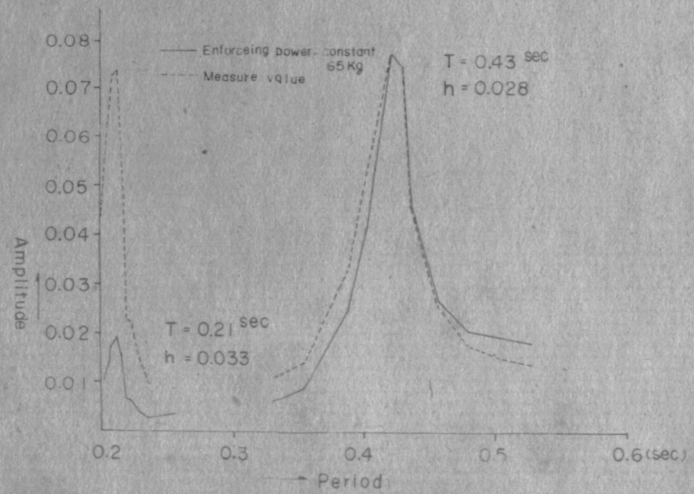


Fig 2



Resonance Curve

Fig 3

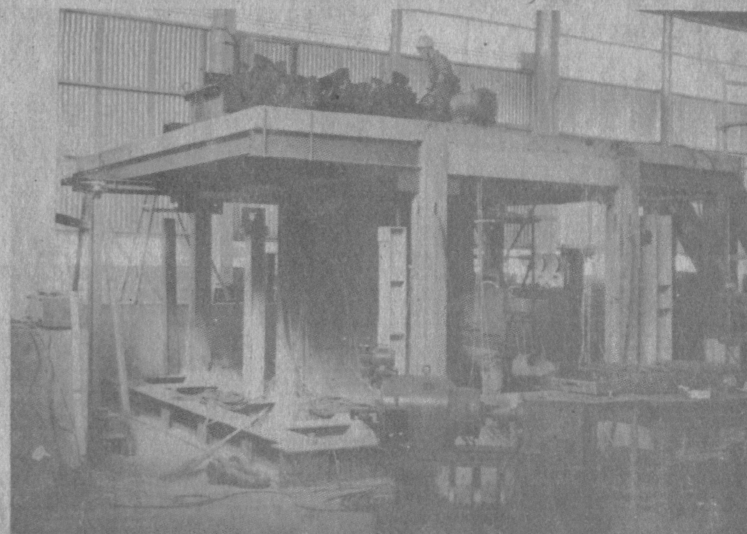


Fig 4

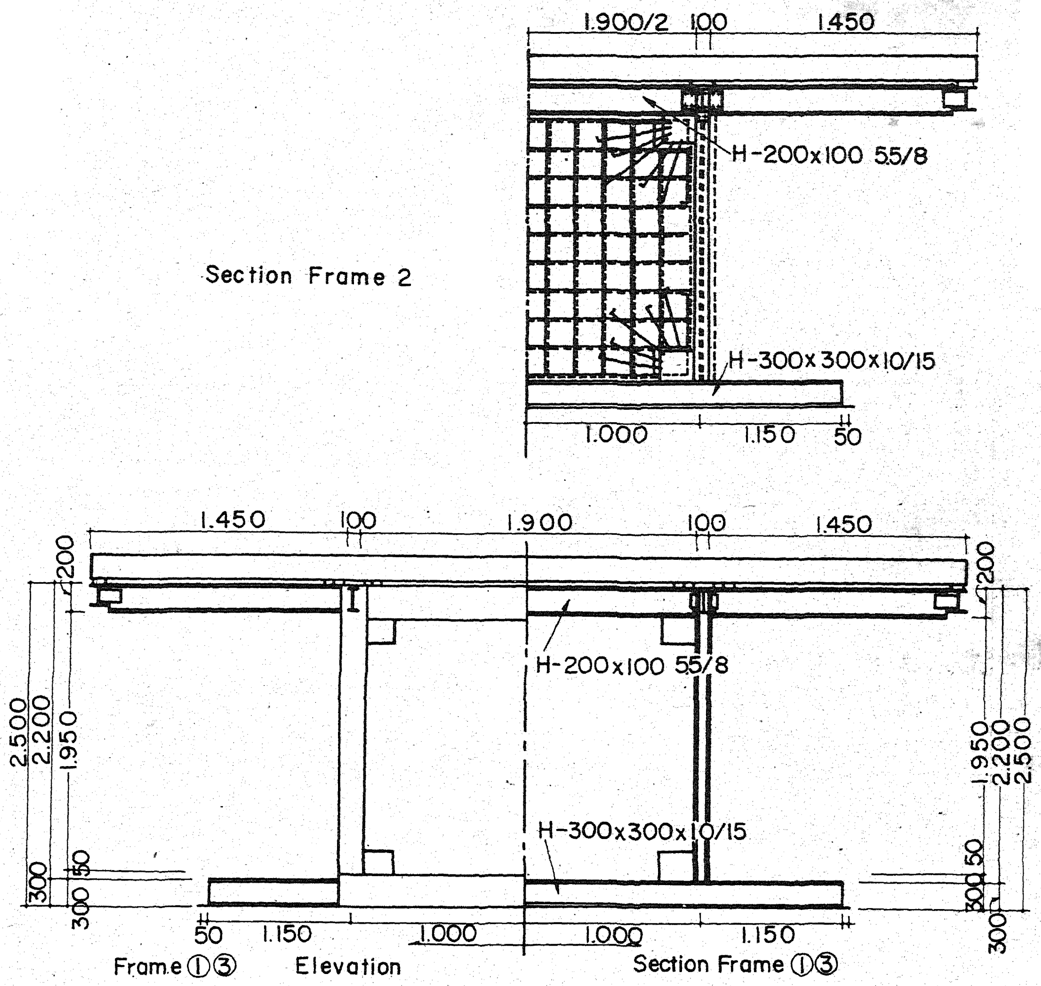


Fig.5

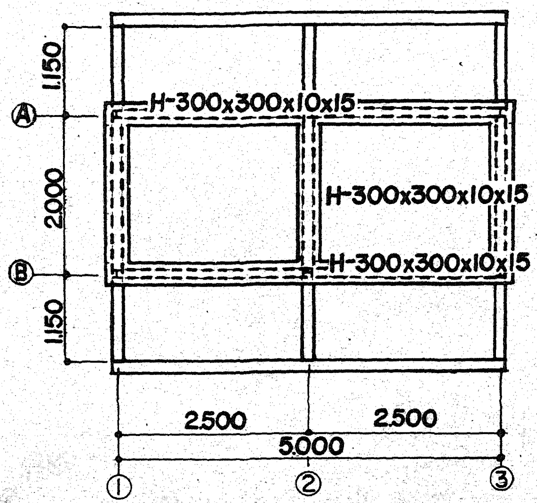


Fig.6. Base Beam Plan

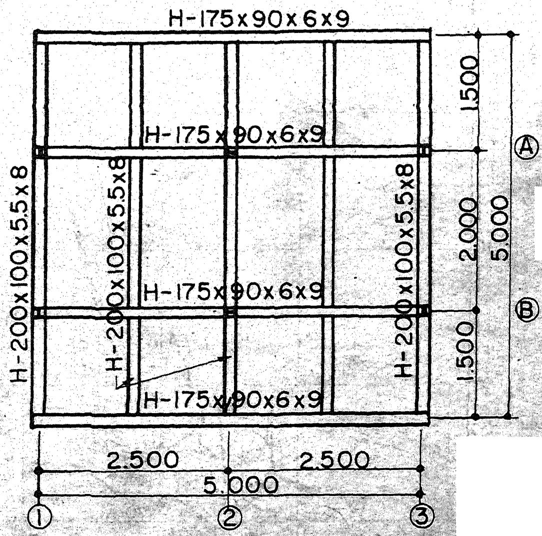


Fig.7 Roof Beam Plan

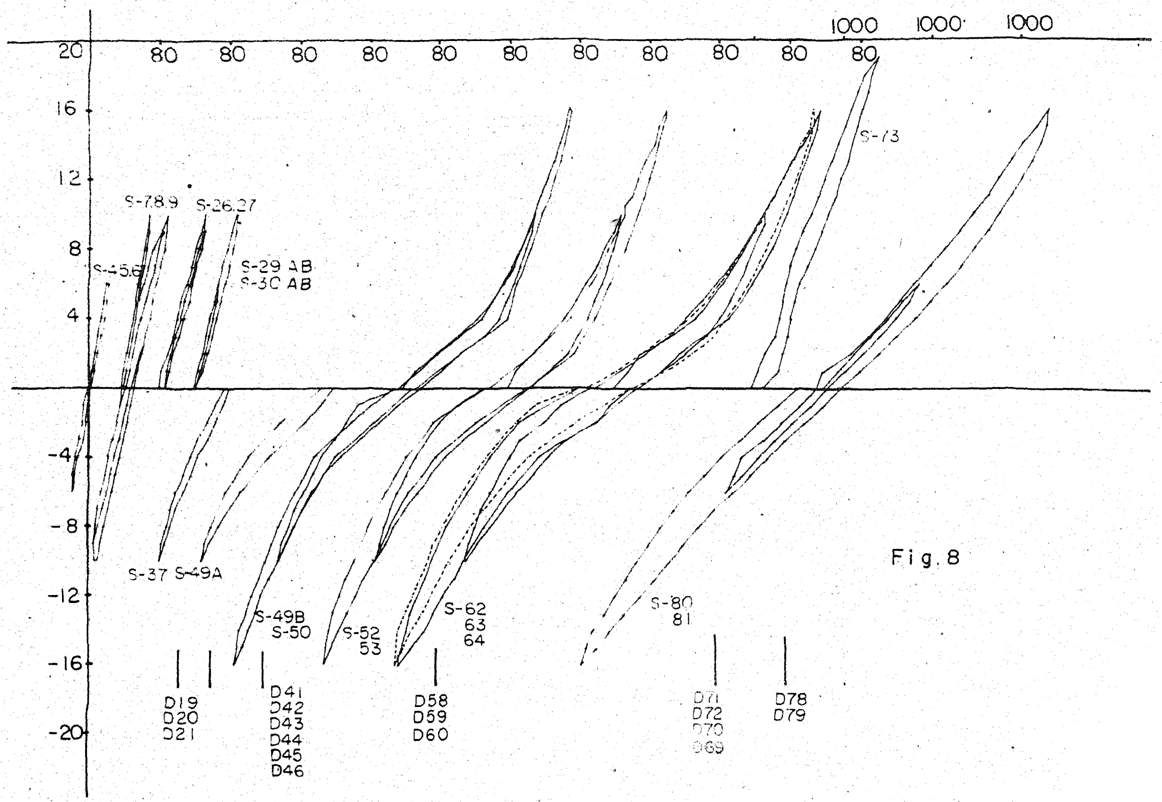


Fig. 8

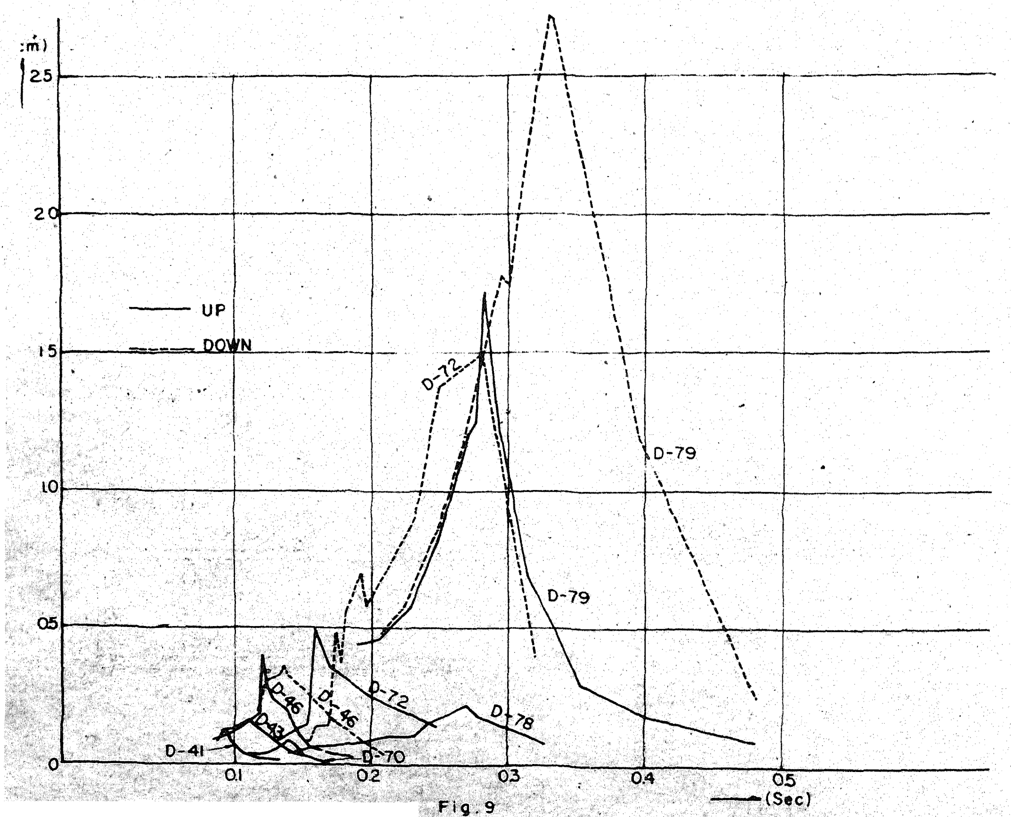


Fig. 9

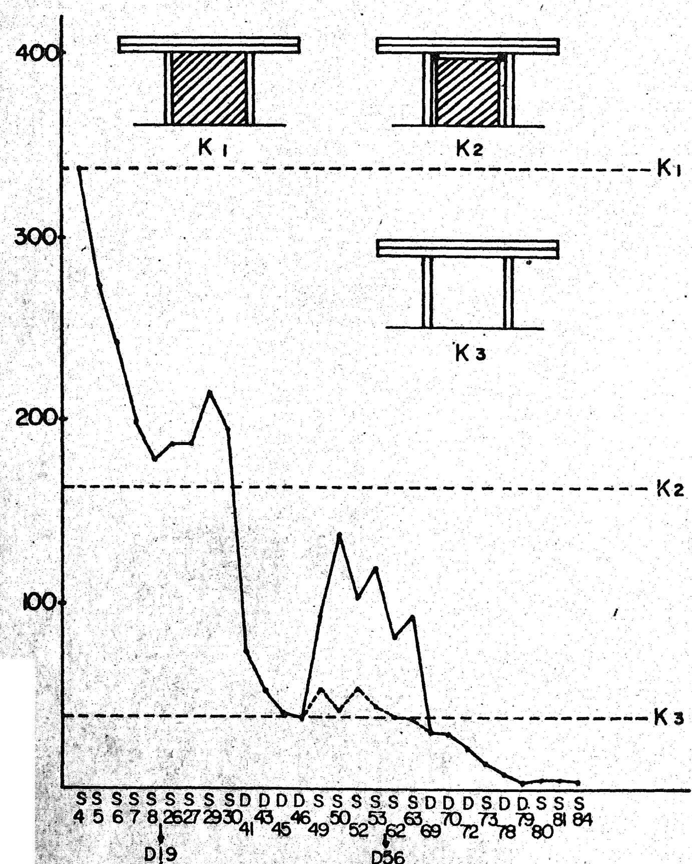


Fig. 10

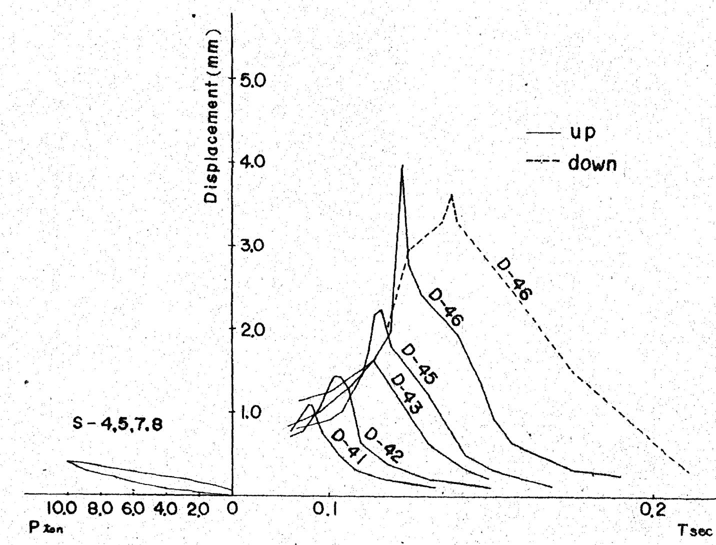


Fig 11

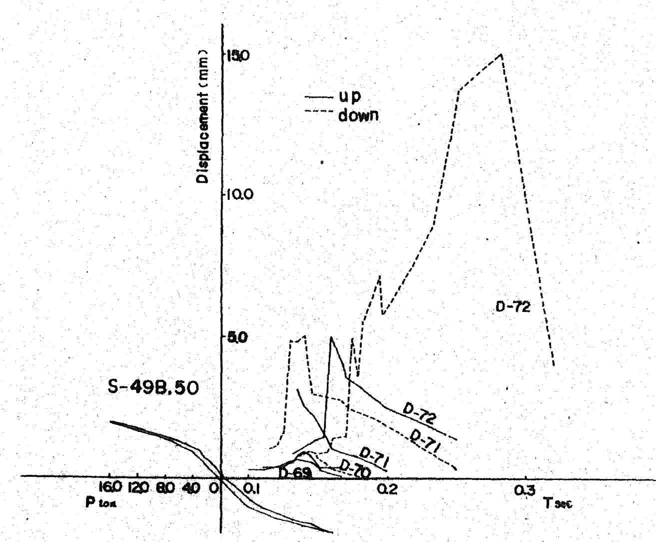


Fig 12

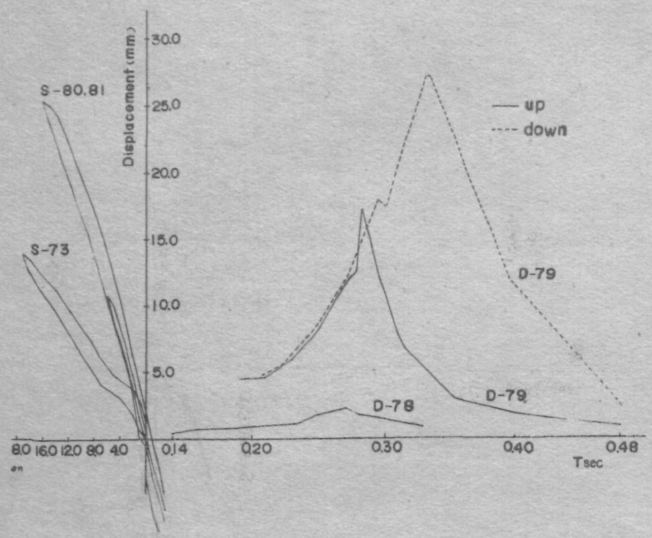


Fig 13

Fig 14

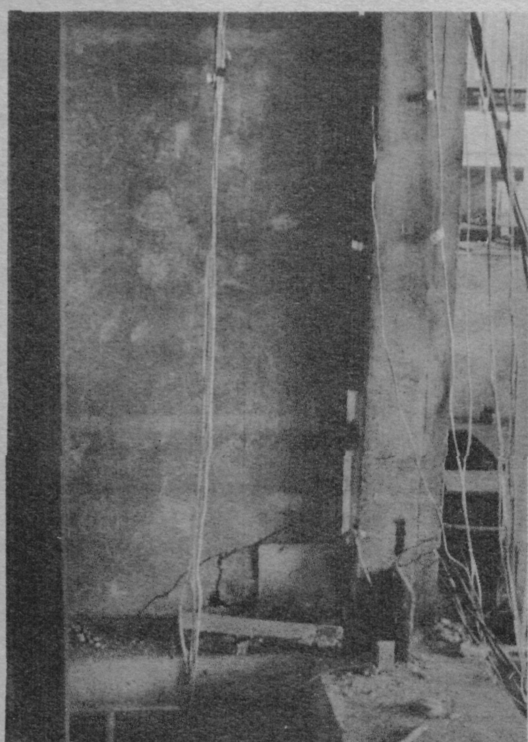


Fig 15

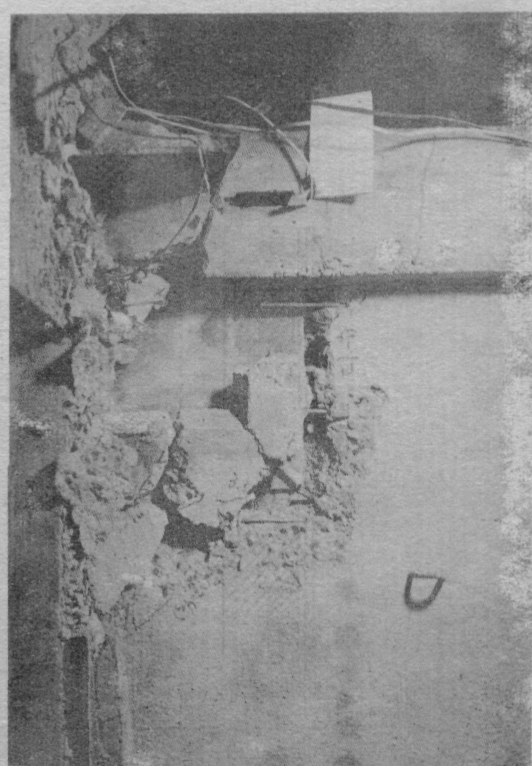


Fig 16

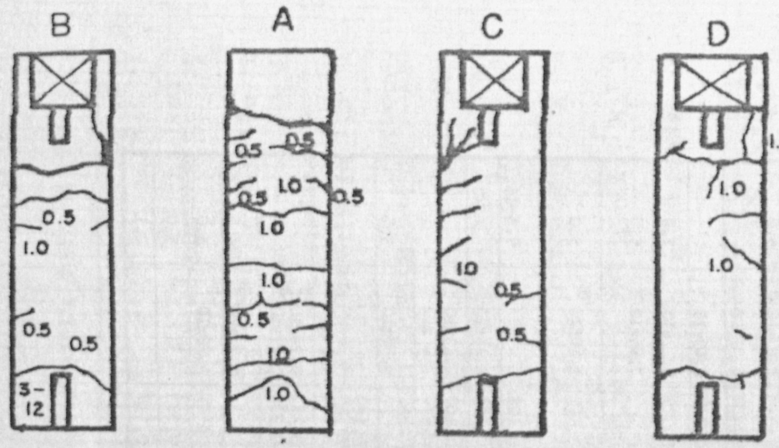


Fig. 17

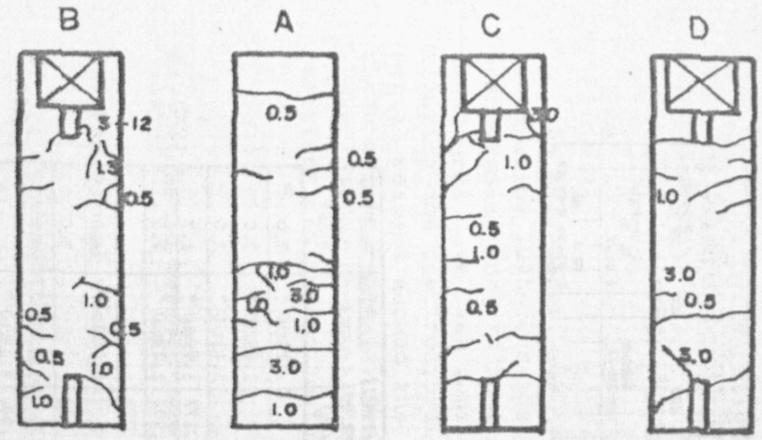


Fig. 18

B-2

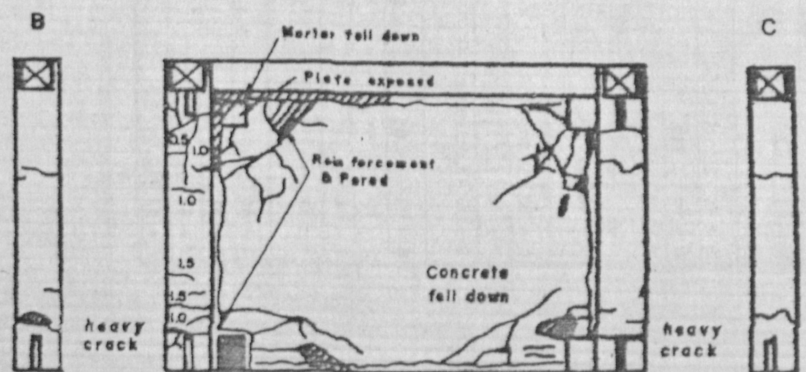


Fig. 19

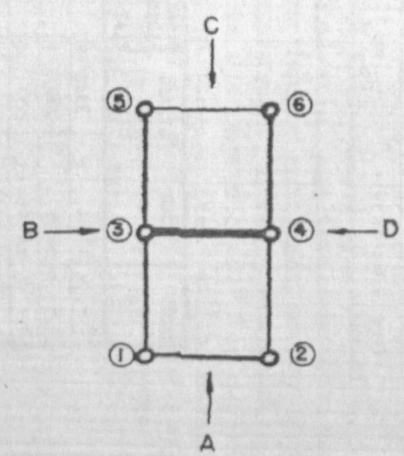


Fig. 20

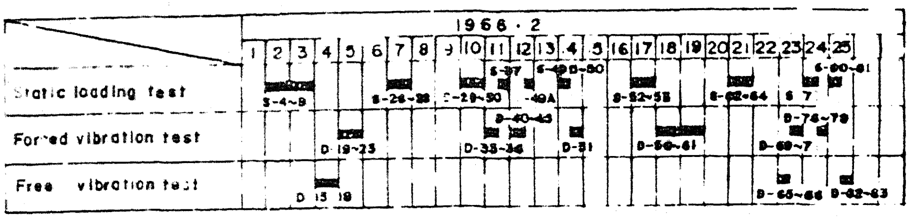


Table - Schedule of the experiment

TEST NO.	LOAD	STATIC RIGI - DITY	DYNAM C RIGIDITY K(t/T K= $\frac{m\ max}{max}$)		δ_{max} (mm)	α max (g)	NATURAL PERIOD (sec)	ROCATION ANGLE	DAMPING COEFFI-CIENT
S - 4	±3.0t	333t/cm			0 09			4.28×10^{-5}	2.0
S - 5	±6.0t	272			0.22			1.05×10^{-4}	2.0
S - 9	±10.0t	333			0.30			1.43×10^{-4}	2.0
D - 19	0.55t(15c/s)		20.6	16.3	0.41	0.042	0.177	1.95×10^{-4}	1.7
D - 21	..84t(")		15.3	14.6	2.58	0.24	0.204	1.23×10^{-3}	1.5
D - 22	0.55t(")		101			0.35	0 080		
S - 26	-10.0t	185			0.54			2.57×10^{-4}	4.0
D - 36	2.0t(15c/s)		74.3			0.40	0.093		3.0
S - 37	-10 0t	125			0.50			3.81×10^{-4}	3.0
D - 41	3.0t('5c/s)		73.1	62.9	1.14	0.45	0.094	5.43×10^{-4}	
D - 43	5.[illegible](")		54.3	35.3	1.69	0.37	0.109	8.05×10^{-4}	7.2
D - 46	10 0t(")		37.5	35.2	3.69	1.14	0.123	1.76×10^{-3}	3.2
S - 49	-10.0t	k=55 k=93			1.50			7.17×10^{-4}	5.0
S - 53	±16.0t	k=42.6 k=121			1.82			8.67×10^{-4}	5.0
D - 56	2.25t(15c/s)		11.2	23.5	0.98	0.14	0.240	4.67×10^{-4}	
D - 60	3 38t(")		6.0	7.0	19.2	0.84	0.322	9.14×10^{-3}	3.8
S - 62	±10.0t	k=39 k=81			1 67			7.95×10^{-4}	5.0
S - 64	±16.0t	k=37.5 k=93			2.30			1.10×10^{-3}	5.0
F - 66	10.0t		35.0				0.128		9.5~16.4
D - 70	5.27t(15c/s)		28.8	26.8	0.93	0.18	0.135	4.43×10^{-4}	
D - 72	3.0t const		21.8 7.8	37.0	5.10 15.12	1.35	0.160 0.270	2.43×10^{-3} 7.2×10^{-3}	
S - 73	+19.0t	13.8						6.58×10^{-3}	3.5
D - 77	5.27t(15c/s)		25.0			0.55	0.150		
D - 78	10.5t(")		7.7	5 2	2.52	0.94	0.270	1.20×10^{-3}	
D - 79	42.5t(")		7.1 5.1	8.3 5 4	17.4 26.6	1.04	0.282 0.332	8.28×10^{-3} 1.27×10^{-2}	
S - 80	±6.0t(")	5.5			11.0			5.23×10^{-3}	4.0
S - 81	±16.0t	6.3			25.6			1.22×10^{-2}	4.0
F - 82	10.0t		4.5				0.353		3.0
F - 83	10.0t		4.7				0.347		2.1

Table 2