Experiment on Behaviour of Prestressed Concrete Four Storeyed Model Structure under Lateral Force.

By

Kiyoshi Nakano*

Synopsis

A four storeyed model structure of prestressed concrete has been subjected by horizontal forces. The forces have been applied at each floor level by hydraulic jacks. Measurements were performed on horizontal deflections, period of natural vibration at each loading steps, and strains of concrete. The test results would serve for clarifying load-deflection characteristics of prestressed concrete building structures.

1. Introduction

The present experiment has been performed as a synthetical test for a series of research on "design method of joint panel of prestressed concrete structures".

The author believes that structural soundness of assembly-type construction depends mainly on proper design of joint panel and structural joint. Provided that the above proper design has been done, prestressed concrete structures would have ample resistance both for vertical loading and earthquake. Then, prosperous application of prestressed concrete may be expected as a powerful means of rationalization of building industry.

This experiment has been done in accordance with the following plan:

(i) Columns and beams of the specimen are both prestressed. But, beams which are perpendicular to the direction of loading are made of ordinary reinforced concrete.

(ii) In joint panels, prestress is introduced. The type of joint panels is the following four kinds:

Type 1. r - (width of column)/(width of beam) - 10cm/10cm - 1, PC bars are arranged in double both for column and beam.

Type 2. r - 12/10 - 1.2, PC bars are arranged in double for column, and in single for beam.

Type 3. r - 15/10 - 1.5, PC bars are arranged in double for column, and in single for beam.

Type 4. r - 20/10 - 2, PC bars are arranged in double both for column and beam.

(iii) Type of floor slabs is the following four kinds:

* Building Research Institute, Ministry of Construction

Roof slab

Three precast reinforced concrete slabs are adhered by epoxy mortar on beams' surface. Openning between precast slabs are also filled with epoxy mortar.

4th floor slab

Three precast reinforced concrete slabs are jointed by ordinary reinforced concrete procedure. Connection of the above three slabs are attained by welding two steel plates which have been inserted and anchored in each precast slabs.

3rd floor slab

The same as the roof slab. But, halving joints are prepared for connection of slabs and beams.

2nd floor slab

The same as the 4th floor slab. But, halving joints are prepared for connection of slabs and beams.

(iv) The same amount of horizontal forces have been applied to each floor levels. Considering the fact that actual seismic forces change its direction, alternative loadings have been applied, that is, the specimen has been loaded from north side and south side.

(v) Stress has been put on measurement of ductility. So that, load-deflection diagrams under lateral force were one of the main items of measurement.

(vi) Observation of cracking in joint panel.

(vii) Observation of failure, particular to three dimentional specimen.

(viii) Measurement of strain of concrete.

(ix) Examination of rigidity of assembly-type floor, and its effect on rigidity of frame.

(x) Examination of decrease of rigidity of frame in accordance with increment of load.

2. General description of specimen

(2.1) Preparation of specimen

The specimen is shown in Fig. 1. Elements of the specimen were prefabricated at the Kamonomiya factory of the PS Concrete Co. These elements were assembled at testing room of the Building Research Institute.

Sequence of assemblage is as follows:

(i) Prestressing of columns.
(ii) Placing beams between two columns, and casting joint concrete between beam and column.
(iii) After hardening of the joint concrete, the beams were prestressed.

Thus, a pair of prestressed concrete two dimentional frame has been prepared on the floor of the testing room. Grouting has then been done.

(iv) The above two frames have been errected by crane one by one, and anchor plates of columns were fixed to the double deck of the testing room. The fixing was performed by four Ø 20mm PC bars per column. These PC bars had previously been tensioned by 15 tons per bar, and been anchored to the deck. After fixing the anchor plates to the bars, space between the anchor plate and the deck was packed by dry mortar, and the anchor plates were anchored to the deck using nuts.

(v) Reinforced concrete beams which are perpendicular to loading direction, were hoisted one by one, and were shored in final position. Reinforcement which had projected from the beam end was welded with mild steel bars inserted in the column. After that, concrete was cast.

(vi) Hoisting of slab elements, and connection of slabs and beams, were performed as mentioned in 1. (iii).

Prestressing force is as follows:

Columns:
4-Ø12mm, 6 tons per bar. Total prestressing force is 24 tons.

Beams of 2nd and roof floor:
2-Ø18mm, 12 tons per bar. Total prestressing force is 24 tons.

Beams of 3rd and 4th floor:
1-Ø18mm, 12 tons per bar. Total prestressing force is 12 tons.

(2.2) Qualities of concrete, mortar, and grout.

(2.2.1) Concrete. Mixing proportions are shown in Table-1. Test results of mechanical properties are shown in Table-2, and Table-3.

(2.2.2) Epoxy mortar. Epoxy mortar was used for adhesion of roof slab and 3rd floor slab. Epoxy resin was also used for adhesion of precast slabs each others. Mechanical properties of these epoxy mortar and resin are shown in Table-4.

(2.2.3) Grout. Grout used was pure cement paste. The cement was ordinary portland cement. Water cement ratio was 45.2%. Test results by 4 x 4 x 16cm test pieces showed bending strength of 17.6 kg/cm^2, and compressive strength of 388 kg/cm^2 (mean value of 9 test pieces).

(2.3) PC steel and mild steel.

PC steel bars fabricated at the Koshuha Netsuren Co., were used. Diameter of the PC bars were 12, 18, and 20mm. Mechanical properties of these bars are shown in Table-5. Mechanical properties of ordinary steel bars are shown in Table-6.

3. Method of loading and measurement

Horizontal loads have been applied to each floor by hydraulic jacks at the midle point of two columns.

Measurements were performed on the following items:

(1) Horizontal deflection at each floor levels.
(2) Strain of concrete was measured by SR-4.
(3) Period of natural vibration of the specimen was measured by accelerometer. For the purpose of exciting natural vibration of the specimen, the specimen was at first puiled by wire rope, and then, a thin wire which connected the wire rope and the specimen, was cut off by a shears.

Loading stages are shown in Table-7.

4. General characteristics of the specimen

(4.1) Bending resistance of members.

The results of calculations are shown in Fig. 2.

(4.2) Stress and deflection of the frame in elastic range.

Characteristics of the frame are shown in Table-8.

Moment diagram of the frame, corresponding to fest end moment of 100 kgm at each beam ends, is shown in Fig. 3.

Resisting moments which correspond to the state of decompression are as follow:

G1-G4:	120kg/cm^2	x 667	800 kgm.
C4 :	120kg/cm^2	x 667	800 kgm.
C3 :	100kg/cm^2	x 800	800 kgm.
C2 :	80kg/cm^2	x1000	800 kgm.
C1 :	60kg/cm^2	x1334	800 kgm.

Supposing that each beam ends are subjected to the same quantities of fest end moment, the fest end moment which corresponds to the state of decompression is 800/88 x 100 910 kgm, (See Fig. 3). This moment corresponds to vertical load of 8.4 tons per floor, or 10.2 tons per floor. The former is calculated on the basis of uniform distribution of the load to the beam, and the latter is calculated on the basis of triangular distribution of load. From the above calculations, we may assume that design horizontal force of this frame corresponding to k 0.2g is between (8.4 x 0.2)/2 to (10.4 x 0.1)/2.

Moment diagram of the frame corresponding to 1.0 ton's horizontal force per floor is shown in Fig. 4.

Calculated joint translation angles, and panel joint angles of the frame corresponding to 1.0 ton's horizontal force per floor are shown in Fig. 5. The unit of figures in Fig. 5 is 1/6 EK. Supposing that E 3 x 10^5kg/cm^2, and K_o EJ/ 116cm^2, deflections of each floor levels

are calculated as indicated in Table-9.

It may be seen from the above Table that the ratio of deflection do not change practically whether slabs co-operation of slabs decrease deflection about 26% compared with the deflection where slabs do not co-operate.

(4.3) Stresses corresponding to design load under seismic force.

Based on the calculations in the above section (4.2), following combinations of load are calculated.

(a) n (G P) 1.5 K Mr

where, n 1.0 or 1.2, according to direction of seismic force
G stresses due to dead weight of the frame
P stresses due to live load. Here P is considered as zero.
K seismic stress
M_r resisting moment of members. (See (4.1))

K $(M_r - n\,\overline{G \quad P})/1.5$
P_h K/K_o

where, K_o is stresses corresponding to horizontal force of 1 ton per floor.

(b) P_h' K'/K_o is calculated by the same method as shown in (a). But, here, P_h' is stresses corresponding to the stresses at the time of decompression.

The values of P_h and P_h' are shown in Fig. 6.

From Fig. 5, the design horizontal force is estimated as 1.18 tons, that is a little higher than the values calculated in (4.2), 0.85-1.04 ton For the present experiment where P 0, the value of P_h has been calculated as 2.3 tons.

(4.4) Period of natural vibration of the specimen.

(a) Unit elastic force of the frame.
Unit elastic forces, C_{ij}, horizontal forces which are necessary to cause unit horizontal deflection (1cm) at each floor levels, are shown in Fig. 7.
The weights of each floors are also indicated in Fig. 7.

(b) Equilibrium of forces. Vibration equation.

Particle 4:	$W_4/g\ y_4$	$C_{14}y_1$	$C_{24}y_2$	$C_{34}y_3$	$C_{44}y_4$	0
Particle 3:	$W_3/g\ y_3$	$C_{13}y_1$	$C_{23}y_2$	$C_{33}y_3$	$C_{43}y_4$	0
Particle 2:	$W_2/g\ y_2$	$C_{12}y_1$	$C_{22}y_2$	$C_{32}y_3$	$C_{42}y_4$	0
Particle 1:	W_1/gyy_1	$C_{21}y_1$	$C_{21}y_2$	$C_{31}y_3$	$C_{41}y_4$	0

Solving the above equation we obtain, circular frequency n, and period of natural vibration T, as follows:

$_1n$ 70, $_2n$ 206, $_1T$ 0.09 second, $_2T$ 0.03 second.

(c) Spring constants and weights of each floors.
Spring constants obtained from actual loading test, and weights of each floors are shown in Fig. 8. In the above mentioned loading test, a are 0, 0.5 t, and 1.0 t.

Equilibrium of each particles is as follows:

$$
\begin{array}{llll}
M_1 & y_1 & & K_2 \; y_2 \; K_1 \; y_1 \\
M_2 & (y_1 \; y_2) & & K_3 \; y_3 \; K_2 \; y_2 \\
M_3 & (y_1 \; y_2 \; y_3) & & K_4 \; y_4 \; K_3 \; y_3 \\
M_4 & (y_1 \; y_2 \; y_3 \; y_4) & & -K_4 \; y_4
\end{array}
$$

Where, y_i is absolute deflection of each floors.

Solving the above equation, we obtain the following values:

$_1n$ 63.3, $_1T$ 0.0992 second.

5. Consideration of the test results.

(5.1) Resistance against horizontal force.

In this experiment, horizontal forces were applied up to the moment when the frame entered yielding state, but were not applied up to collapse of the frame. Then, it is not clear the relation between design horizontal force and actual ultimate horizontal resistance of the frame. But the following facts may be concluded:

(1) This frame resist safely against a horizontal force which is about 2.6 times of design horizontal force.

(2) According to the measurement by SR-4, at the time of maximum load, strains of concrete exceed 0.2% at the point marked as 0 (see Fig. 6), and exceed 0.1% at the points marked as x in the Figure.

Consequently, it is seen that considerable numbers of plastic hinges might have been formed in the frame. Then, it may be said that properly designed prestressed concrete structures can behave as high degrees statically indeterminate structures.

(5.2) Maximum deflection, and ductility.

Load-deflection diagrams of each floors are shown in Fig. 9

From these diagrams the following facts may be concluded:

(1) Coincidence of measured deflections with calculated deflections is limited only to low loading stage. That is because, rigidities of G2 and G3 are lowered at the early stage of loading due to crackings.

In Table-10, comparison between measured deflections and calculated deflections is shown.

As indicated in the above table, the coincidence is fairly well up to the loading of 1 ton. At the loading stage which is a little higher than the design horizontal force, the ratio of measured and calculated deflection is 0.58-0.65. That means the fact that at the loading stage, rigidity of the frame decreased about 40%.

(2) The diagrams shown in Fig. 9, show the fact that, load deflection curves have no breaking point, and rigidity of the frame lowers continuously, in accordance with increase of load. Recovery of deflection is remarkably well, then absorption of energy is quite small.

(3) Measured maximum relative deflections at each floor levels are as follows:

1st storey:	29.2mm
2nd storey:	27.8mm
3rd storey:	21.1mm
4th storey:	8.6mm

These values, corresponding to 0.9/100 - 2/9/100 of storey height, are considerably greater than the value of relative deflection which is ordinary allowed for buildings. The deflection of 1st storey, 29.2 mm, exceed theoretical maximum deflection of single storeyed specimen which has been tested by the author[1]). Then it may be said that the present type of prestressed concrete frame has sufficient ductility.

(5.3) Dynamic behaviour of specimen.

The values of fraction of critical damping h, and damping rationd, calculated from the load-deflection diagrams shown in Fig. 9, are indicated in Table-11.

The periods of natural vibration for the first mode, measured by a test are shown in Table-12. This test has been performed as follows: At first the specimen was pulled by chain block, and then wires connecting the chain block and the specimen was cut off, thus, natural vibration of the specimen was excited.

Time-acceleration diagrams were recorded during the natural vibration, using accelerometers settled on the roof slab of the specimen. The test was performed after each loading stages.

From the above mentioned results and the results of (4.4), the following facts may be concluded:

(1) As to the period of natural vibration, the above three values of period ($_1T$ 0.09 second, the value obtained by elastic calculation, $_1T'$ 0.099 second, the value obtained by calculation using measured spring constants of the frame, $_1T''$ 0.097 second, the value obtained from the test result of natural vibration of the frame), coincide fairly well.

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1) BRI Occasional Report No. 21, September 1964.

(2) As to the period of natural vibration of second mode, there is no test result. But according to the calculation, (see (4.4) (e)), ratio of ${}_1T/{}_2T$ 0.3. Then, it is seen that this frame has characteristics of shear vibration type.

(3) The period of natural vibration of the first mode, measured after each loading stages are as shown in Table-12. From the above Table, it is seen that the period do not change practically up to the loading stages which do not exceed the design horizontal force, and that the period become longer about 10% after the loading of 2.17 times (5/2.3) of the design horizontal force and about 30% after the loading of 2.6 times (6/2.3) of the design horizontal force. That is, increase of the period of natural vibration of prestressed concrete structures, in accordance with progress of partial ruptures, is very small.

(4) Damping factor, and fraction of critical damping are calculated based on the static loading tests. Judging from the load-deflection diagrams (see Fig. 9), damping of the frame is practically zero up to the loading stage of P_h 3.5 tons. As shown in Table-11, the fraction of critical damping is 0.07 after the loading of 2.17 times of the design horizontal force. Then, it is obvious that the damping of prestressed concrete structures is considerably less than that of reinforced concrete structures. It is seen from Table-11 that the values of h are different for each storeys. That means the fact that degree of damage is different for each storeys.

(5.4) Behaviour of slabs under lateral force.

As to the precast slabs used for the frame, the following facts may b be said:

(1) There is no practical difference between the joint method of ordinary reinforced concrete procedure, and that of epoxy mortar joint.

(2) Three precast concrete panels have behaved monolithically up to the final loading stage.

(3) Transmission of horizontal force from the slabs to the frame are satisfactory. But, it has been cleared that deformation of the slabs have the same tendency of that of the frame, and considerable bending deformation of slabs has measured.

(4) Cracking patterns of slabs and the frame arround the slabs, are different according to the joint methods. As to this point further consideration is required. (See Fig. 10 and Fig. 11).

(5.5) Strain of concrete.

As a result of the measurement of strain of concrete by SR-4, the followings may be concluded.

(i) Initial cracking load of the frame nearly coincide with calculated value.

(ii) Panel joint having single PC bar lose rigidity of joint in accordance with increment of load.

(iii) Capacity of energy absorption of joint concrete is small, both for the joint with single PC bar and for the joint with double PC bars. That is, strain of concrete which has increased plastically restores fairly elastically.

6. Conclusion

By the above mensioned experiment, several problems on structural design of prestressed concrete have been revealed.

(1) Statically indeterminate structure having sufficient ductility may be designed provided that design of panel joint is properly performed.

(2) Special attention should be paid for seismic response of prestressed concrete structures, which have different dynamic characteristics to reinforced concrete structures. The main point of difference is that, in reinforced concrete structures, deflection characteristics enters considerably into plastic range when structures are subjected design horizontal force, on the contrary, in prestressed concrete structures, deflection characteristics do not enter into plastic range provided that they are not subjected to horizontal force of several times of the design force.

As to seismic responce, it is generally assumed that prestressed concrete structures may be subjected to the greater seismic force than that of reinforced concrete structures, because fraction of critical damping is smaller for prestressed concrete structures than for reinforced concrete structures. Then, in actual earthquake, it may be assumed that there is sufficient possibility that prestressed concrete structures also enter into plastic range. In the other words, it may be considered that equivalent static lateral force, corresponding to the same ground motion, should be greater for prestressed concrete structures than for reinforced concrete structures. Then, it is only apparent properties that prestressed concrete structures have elastic deflection characteristics up to several times of the design horizontal force. Then, practical criteria for judging soundness of prestressed concrete structures should be ductility in plastic range, and relative deflection at the moment when structures enter into plastic range.

(3) Ductility of the frame have been sufficient as mentioned in (1). But, this fact should not be applied unconditionally. To assure ductility of structures, the followings are paramountly necessary: (i) to perform proper design of joint panel, (ii) to avoid premature formation of plastic hinge in column.

(4) As the relative deflection, it should be kept in mind that there is an inherent minimum deflection for each structures which should be considered in case of earthquake design. The above minimum deflection has an important meanings when designing interior and exterior finishing of buildings. From the stand point of ductility, it is preferable that relative deflection is great as possible. But, excessive deflection, at the time when structures enter into plastic range, make it difficult the design of exterior and interior finishings. In this frame, relative deflection is about 1.5 cm per storey. That is about twice of ordinarily permitted maximum deflection for steel skeleton structures in Japan.

That means that, earthquake damage of prestressed concrete structures having ordinary finishings may be greater than that of reinforced concrete, at least as to interior and exterior finishing. In case of prestressed concrete structures having many shear walls, the above presumption should not be applied. It may be generally concluded that in designing prestressed concrete structure, method of fixing of exterior and interior finishing materials should be carefully examined.

(5) Residual deformation of joint panel is smaller than that of reinforced structures. The above fact correspond to the smaller capacity of energy absorption and the amaliness of damping capacity. The above properties are disadvantageous for finishing materials, as deflection become great due to the greatness of seismic response. But after ceasing of earthquake, damage of structures may be smaller than reinforced concrete structures, because residual deflection is smaller for prestressed concrete structures, Then, it may be said that, properly designed prestressed concrete structures having proper finishings, may be suffered smaller damage than reinforced concrete structures, against the same intensity of earthquake. In the other words, it may be said that, more economical design is possible by prestressed concrete structures than reinforced concrete structures, against the same degree of damage.

(6) The above conclusions are based on the assumption that structures behave monolithically, and that prestress maintained permanently. Then, the author would like to call special attention on the following points.

(i) Sufficient rigidity of slabs. This is necessary to make clear structural design, and especially important for assembly-type structures. In the present experiment, precast concrete slabs of 5 cm thickness were used. According to the test results, the slabs have behaved monolithically, but there occured considerable amount of tensile strains in the slabs, so that method of junction of slabs to frame should be designed and executed carefully.

(ii) Rupture of prestressing tendons should be carefully avoided, as the rupture lowers reliability and safety of structures mortally. Then it is desirable that, PC steel having sufficient elongation should be used for the part which suffers large bending deformation, and that grouting should be done arround prestressing tendons.

(iii) Tortional effect of members is sometimes necessary to consider. In the present specimen, remarkable cracks have been observed in a reinforced concrete beam which is perpendicular to the loading direction.

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Fig. 1-a. General aspect of specimen (1).
(East and west side)

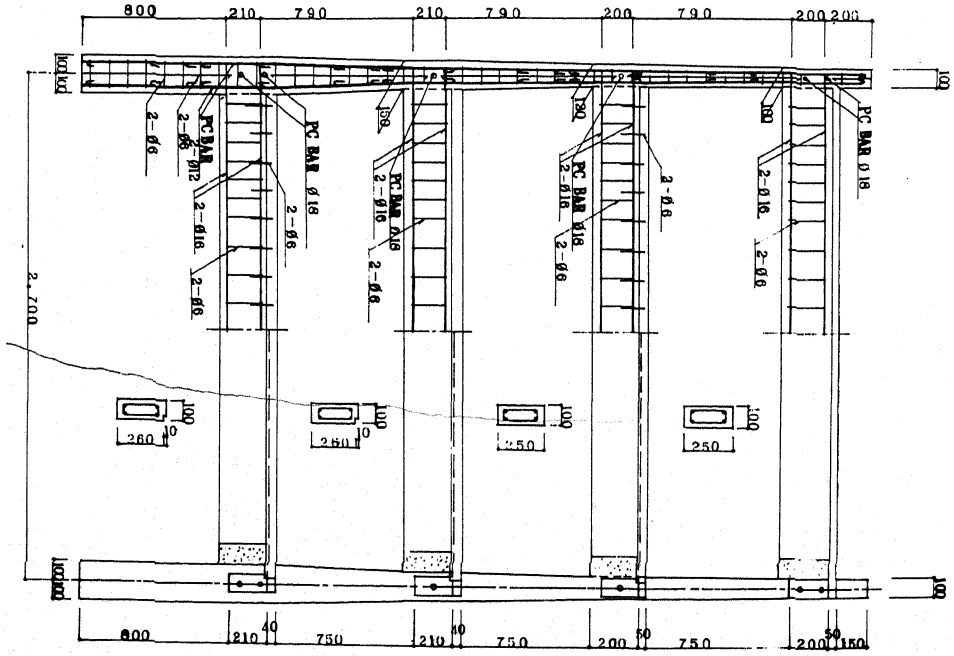


Fig.1-b General aspect of specimen (2).
(North and south side)

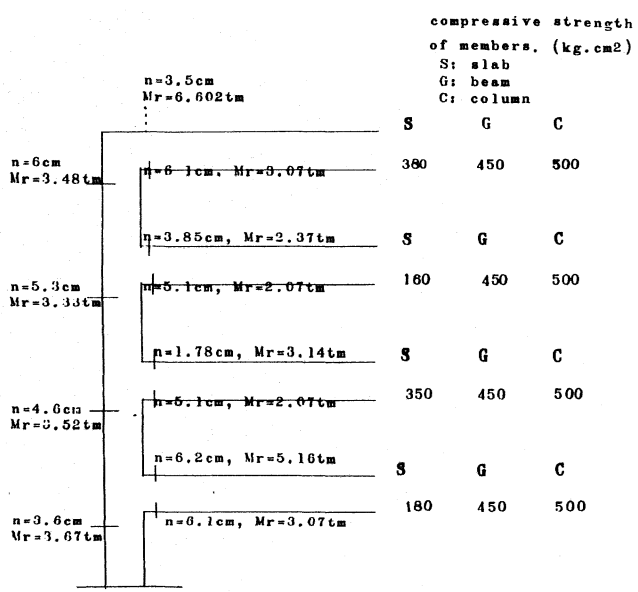


Fig. 2 Bending resistance of members

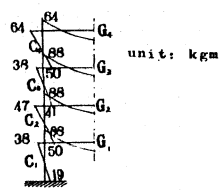


Fig. 3 Moment diagram corresponding to fest end moment of 100 kgm at each beam ends.

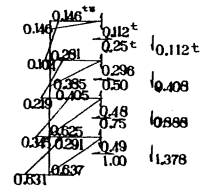


Fig. 4 Moment diagram corresponding to 1.0 ton's horizontal force per floor.

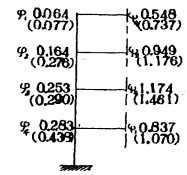


Fig. 5 Joint translation angle φ_s and panel joint angle φ_s.

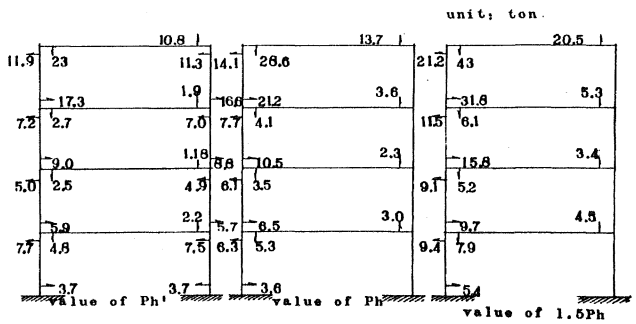


Fig. 6 Horizontal forces corresponding to various combination of load.

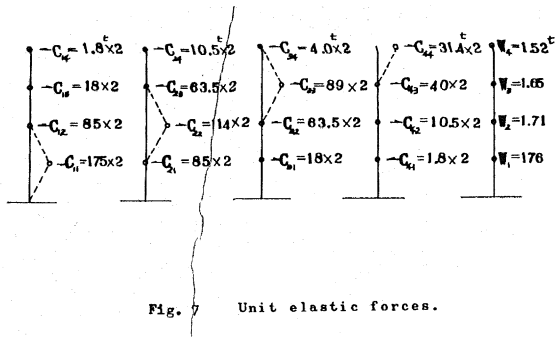


Fig. 7 Unit elastic forces.

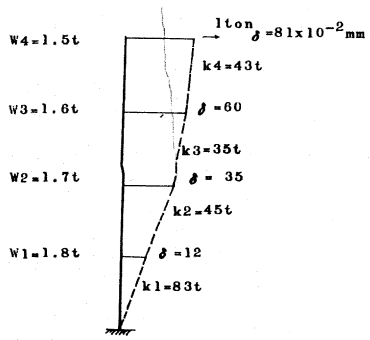


Fig. 8 Spring constants of the frame.

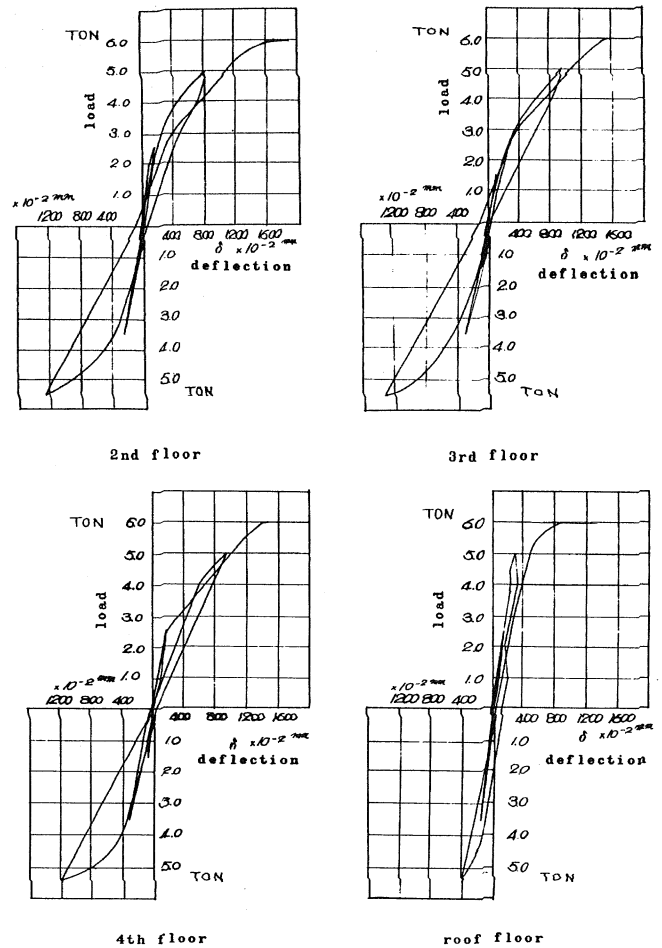


Fig. 9 Load-deflection diagrams.

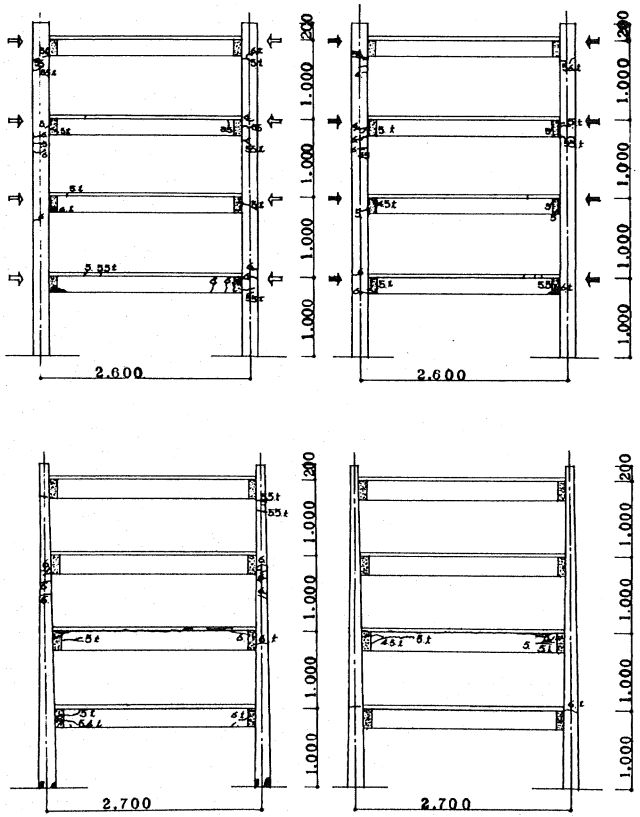


Fig. 10 Cracking pattern of the frame(1).

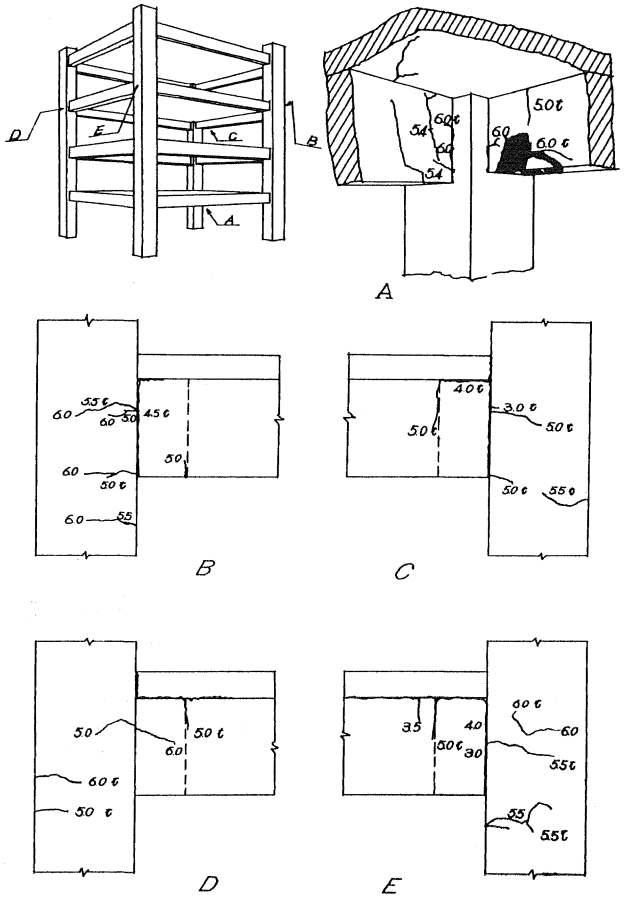


Fig. 11 Cracking pattern of the frame (2). (details.)

Table-1. Mixing proportion of concrete.

position in the specimen.	cement. (kg)	sand (kg)	gravel. (kg)	w/c ratio (%)
		per M^3		
reinforced concrete beam	Asano Rapid Hardening Cement. 300	river sand 842	crashed stone 1118	60.4
PSC column and beam	480	803	1039	36.0
slab	350	845	1100	52.0
PSC beam joint (in situ)	480	797	1030	36.0
RC beam joint (in situ)	425 390	764 910	1010 905	55.0 55.0
slab joint (in situ)	340	990	880	55.0

Table-2. Test results of concrete (1).
(φ 10x20cm test piece.)

position in the specimen.	compressive strength. kg/cm^2	tensile strength* kg/cm^2	Young's modulus** kg/cm^2	test age days
slab joint (1)	313 --- 348	--- 29.0 ---	--- --- 244,000	17 17 17
slab joint (2)	302 --- 308	--- 24.9 ---	--- --- 255,000	17 17 17
PSC beam joint	700 656 --- 777 357	--- --- 42.7 --- ---	--- --- --- 445,000 284,000	28 28 28 28 40 hours
RC beam joint (1)	518 --- 505	--- 40.6 ---	--- --- 339,000	21 21 21
RC beam joint (2)	318 --- 288	--- 31.4 ---	--- --- 231,000	21 21 21
precast slab	134 --- 134	--- 21.6 ---	--- --- 163,000	42 42 42
precast column and beam	499 466 ---	--- --- 38.4	--- 358,000 ---	45 45 45

Remarks: * Tensile strength by Brazil test.
** Gauge lenth is 100mm.

Table-3. Test results of concrete (2).
(Compressive strength of test cylinders which have been cut off from the specimen)

position in the specimen.	diameter x height (cm)	compressive strength (kg/cm^2)
2nd floor slab	4.4 x 5.3 4.4 x 5.8 4.4 x 5.7	197 365 164
3rd floor slab	4.4 x 5.5 4.4 x 5.3 4.4 x 5.5	334 345 390
4th floor slab	4.4 x 5.6 4.4 x 5.3 4.4 x 5.5	154 164 184
roof floor slab	4.4 x 5.4 4.4 x 5.4 4.4 x 5.5	359 418 332
RC beam (2nd floor)	4.4 x 9.1 4.4 x 9.0	184 177
PSC beam (2nd floor)	4.4 x 9.5 4.4 x 9.0	505 415
column (1st storey)	6.8 x 14.0 6.8 x 13.9	462 573

Table-4. Test results of epoxy mortar and resin.

position in the specimen.	diameter x height (mm)	bending strength (kg/cm2)	compressive strength (kg/cm2)	Young's * modulus (kg/cm2)
between slabs	30 x 60 30 x 60 30 x 60	--- --- ---	684 651 646	52,600 --- ---
between slabs	15 x 30 15 x 30 15 x 30	--- --- ---	703 678 684	47,800 --- ---
slab and beam	15 x 30 15 x 30 15 x 30	--- --- ---	698 616 633	64,900 --- --- .
between slabs	prisme 40 x 40 x 160	301 268 226 313 339 353	794 789 770 770 753 776 793 795 789 795 789 795	

* Gauge length was 67 mm. Measured by SR-4.

Table-5. Mechanical properties of PC bars.

nominal diameter. (mm)	diameter. (mm)	maximum load. (kg)	maximum stress. (kg/mm2)	elongation* (%)
12	10.8	13,100	143.0	5.75
12	10.8	13,300	146.0	5.75
12	10.9	13,300	142.5	6.00
12	10.8	13,300	145.0	5.75
12	10.9	13,350	143.0	5.75
18	16.9	28,550	127.3	6.50
18	16.9	28,450	127.0	6.00
18	16.9	28,500	127.1	6.00

* Gauge length was 200mm.

Table-6. Mechanical properties of ordinary steels.

nominal diameter. (mm)	diameter. (mm)	yield stress. (kg/mm2)	maximum stress. (kg/mm2)	elongation.* (%)
6	5.85	---	63.2	14.6
6	5.90	---	63.4	12.5
6	5.85	---	65.3	12.5
6	5.90	---	62.3	14.6
6	6.00	---	61.3	14.6
6	5.80	---	63.3	14.6
6	5.90	---	65.3	12.5
6	5.90	---	64.5	14.6
6	5.90	---	60.1	12.5
6	5.60	54.7	64.1	27.1
6	5.80	55.3	61.7	22.4
6	5.80	56.6	63.1	---
16	16.05	30.2	44.3	31.2
16	16.00	30.2	44.2	31.2
16	15.90	31.7	47.3	32.1

* Gauge length was 8 times of diameter.

Table-7. Loading stage.

No. of test.	loading direction.	loading stage (t)	measurement
1	N	0.5-1.0-1.5	horizontal deflection, strain of concrete, period of natural vibration.
2	S	0.5-1.0-1.5-2.0-2.5	ditto
3	N	1.0-2.0-2.5-3.0-3.5	ditto
4	S	1.0-2.0-3.0-3.5-4.0-4.5-5.0	ditto
5	N	1.0-2.0	horizontal deflection, strain of concrete.
6	N	1.0-2.0-3.0-4.0-5.0-5.5	horizontal deflection, strain of concrete, period of natural vibration.
7	N	1.0-2.0-3.0-4.0-5.0-5.5-6.0	ditto

Table-8. Characteristics of the frame.

member.	dimension. (cm x cm)	cross sectional area. A (cm3)	moment of inertia. I (cm4)	length. L (cm)	K=I/L (cm3)	k
column C1	20 x 20	400	13,300	92.5	144	1.24
column C2	$\frac{20\times(20+15)}{2}$	350	11,600	100	116	1.00
column C3	$\frac{20\times(15+12)}{2}$	270	8,960	100	89.6	0.77
column C4	$\frac{20\times(12+10)}{2}$	220	7,300	100	73.0	0.63
beams G1-G4	10 x 25 (10 x 20)		22,700 (13,000)	260 (260)	87 (50)	0.75 (0.43)

Remarks: Effective breadth of beam is assumed as 40cm(=6t+b). Where, t is thickness of slab.
Figures in ( ) correspond to the case when slabs do not cooperate with the beams.

Table-9. Horizontal deflection corresponding to 1 ton's lateral force per floor.

storey	relative deflection (mm)	ratio of relative deflection	absolute deflection (mm)	ratio of absolute deflection
1	0.402 (0.514)	1 (1)	0.402 (0.514)	1 (1)
2	0.564 (0.701)	1.4 (1.4)	0.966 (1.215)	2.4 (2.4)
3	0.456 (0.564)	1.1 (1.1)	1.422 (1.779)	3.5 (3.5)
4	0.263 0.354)	0.65 (0.7)	1.682 (2.133)	4.15 (4.2)

Remark: Figures in ( ) are correspond to the case where effective width of slabs for G3 and G1 is zero.

Table-10 Comparison of measured and calculated deflections.

(x 10^{-2}mm)

position	0-1.5 ton (N-S) at 1.0 ton			0-1.5 ton (N-S) at 1.5 ton			0-2.5 tons (S-N) at 2.5 tons		
	meas.	cal.	ratio	meas.	cal.	ratio	meas.	cal.	ratio
1	48	40	0.83	103	60	0.58	173	100	0.58
2	52	57	1.10	121	80	0.71	232	143	0.62
3	45	46	1.02	77	67	0.87	180	115	0.64
4	25	26	1.03	43	37	0.86	101	65	0.65

Table-11. Damping characteristics.

loading stage	1st storey		2nd storey		3rd storey		4th storey	
	d	h	d	h	d	h	d	h
0-3.5t	1.2	0.029	1.37	0.049	1.23	0.032	1.07	0.010
0-5.0t	1.56	0.071	1.60	0.074	1.33	0.045	2.72	0.047
0-5.5t	1.53	0.067	1.51	0.066	1.40	0.052	1.31	0.042

Table-12. Period of natural vibration for the first mode.

loading stage	0-1.5t	0-2.5t	0-3.5t	0-5.0t	0-5.5t	0-6.0t
Period (sec.)	0.097	0.103	0.103	0.110	0.120	0.130

EXPERIMENT ON BEHAVIOUR OF PRESTRESSED CONCRETE FOUR STOREYED MODEL STRUCTURE UNDER LATERAL FORCE

BY K. NAKANO

QUESTION BY: S. INOMATA - JAPAN

1. Is the residual deformation of a P.C. frame greater or smaller than that of a R.C. frame?

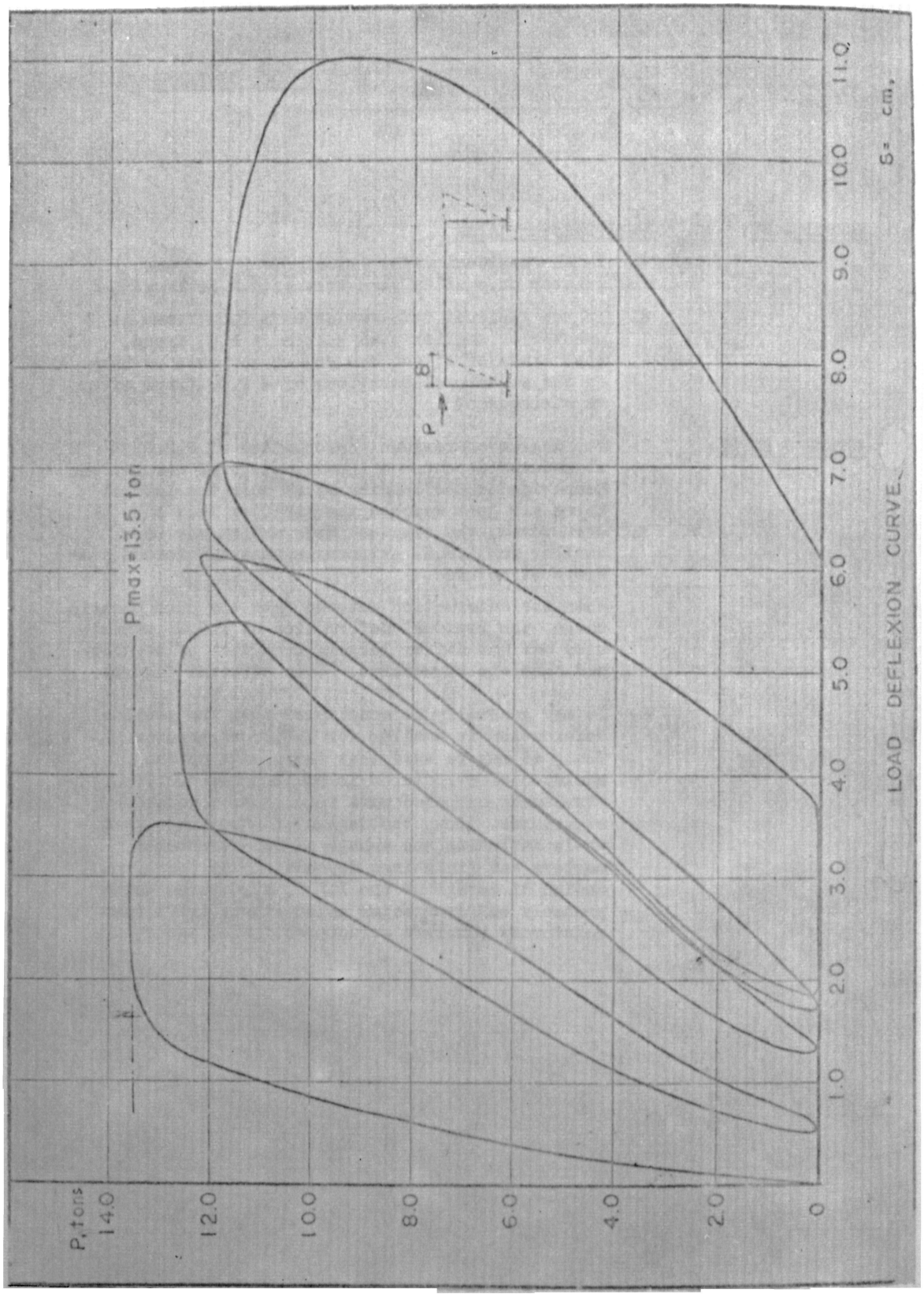
2. If the residual deformation of a P.C. frame is greater or smaller than that of a R.C. frame, what kinds of effect can you expect this to have on the structural behaviour of a P.C. frame after an earthquake?

AUTHOR'S REPLY:

1. The degree of residual deformation of P.S.C. structures depends on loading stage of structures. Comparing at the loading stage when the lateral force has just reached the yielding load of structures, the residual deformation may be smaller for P.S.C. structures than reinforced concrete structures.

   When the deformation entered into the pure plastic range, the residual deformation of P.S.C. structures may not differ largely from that of reinforced concrete structures. (see attached figure).

2. It may generally be considered that the residual deformation is smaller for P.S.C. structures. Then, it may be said that the damage of the skeleton of structures is smaller for P.S.C. structures compared with reinforced concrete structures. But, the damage of structures as a whole including the damage of non-structural members and finishings depends on the proper design of detail of the P.S.C. structures which probably suffer greater inter-storey drift than reinforced concrete structures during earthquake.


P, tons
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
δ = cm.
P max = 13.5 ton
P
δ
LOAD DEFLEXION CURVE