報　告

Abstract on Structural Behaviour of an 8-story Precast Prestressed Conrcete Housing Structure

By Shin Okamoto*

1. Introduction

In the Building Research Institute, the procedure method of housing structure which consists of precast reinforced concrete elements assembled with post-tensioning method has been developed since 1965. During these four years, several pilot constructions have been performed ; including a housing for B. R. I. officials (8 units, 4 stories) publicly operated apartment houses in Kanagawa Prefecture (4 buildings ; 24 units, 4～5 stories and an apartment house for a private firm (30 units, 5 stories). In this year, we have plans on hand for construction of a 8-story apartment house of 104 units with this prefabricating system. This paper mainly concerned with the experiments on a 4-story full-size structure under the action of simulated earthquake load. This test had been carried out in the Large-Size Structures Testing Laboratory at the B. R. I. to provide information to aid the design of the 8-story apartment house, and to obtain the safety factor against failure and cracking under seismic force.

2. Outline of the 8-story apartment house

(1) Basic composition of the structure

This housing structure consists of following precast reinforced concrete structural elements;

1) L, $+$, T and I shaped walled columns : 18 cm thick
2) Girders; 30 cm in width, 74 cm in depth for 2nd to 5th floor and 69 cm in depth for 6th to top floor
3) Slab; plan dimensions of 3.6 m by 2.7 m and 12 cm thick

An outline of the composition of this structure is summarized in **Fig. 1** and **Fig. 2** and some details of elements are listed in **Table. 1**.

The vertical connections between columns and girders are made by post tensioning method. The prestressing bars anchored in underground girders cast in-situ are extended through columns and girders with coupling at each floor level. These bars are tensioned and grouted after completion of assembly works of 5th floor and then extended to the upper floors. The joints between floor slabs and between floor slabs and girders are made by welding the steel plates anchored in each element as shown in Fig. 1 E.

(2) Assumption in the stuctural calculation

The above housing structure has been designed on the basis of simplified assumptions as mentioned below.

1) The vertical distribution of the design seismic force coefficient is based upon the Japanese code as shown in Fig. 3.
2) Structural calculation of the longitudinal frame works.

a) The joint between the columns and girders, so called connection pannel is assumed to have the same rigidity and load carrying capacity as the joint of monolithic reinforced concrete.

b) The stress analysis for the frames has been performed with a modified elastic frame theory under the action of above mentioned design seismic force ; the structural members are idealized into the elements with rigid zones considering the deformation due to bending moments and shear forces.

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Fig. 1 Outline of 8-story precast prestressed concrete housing structure

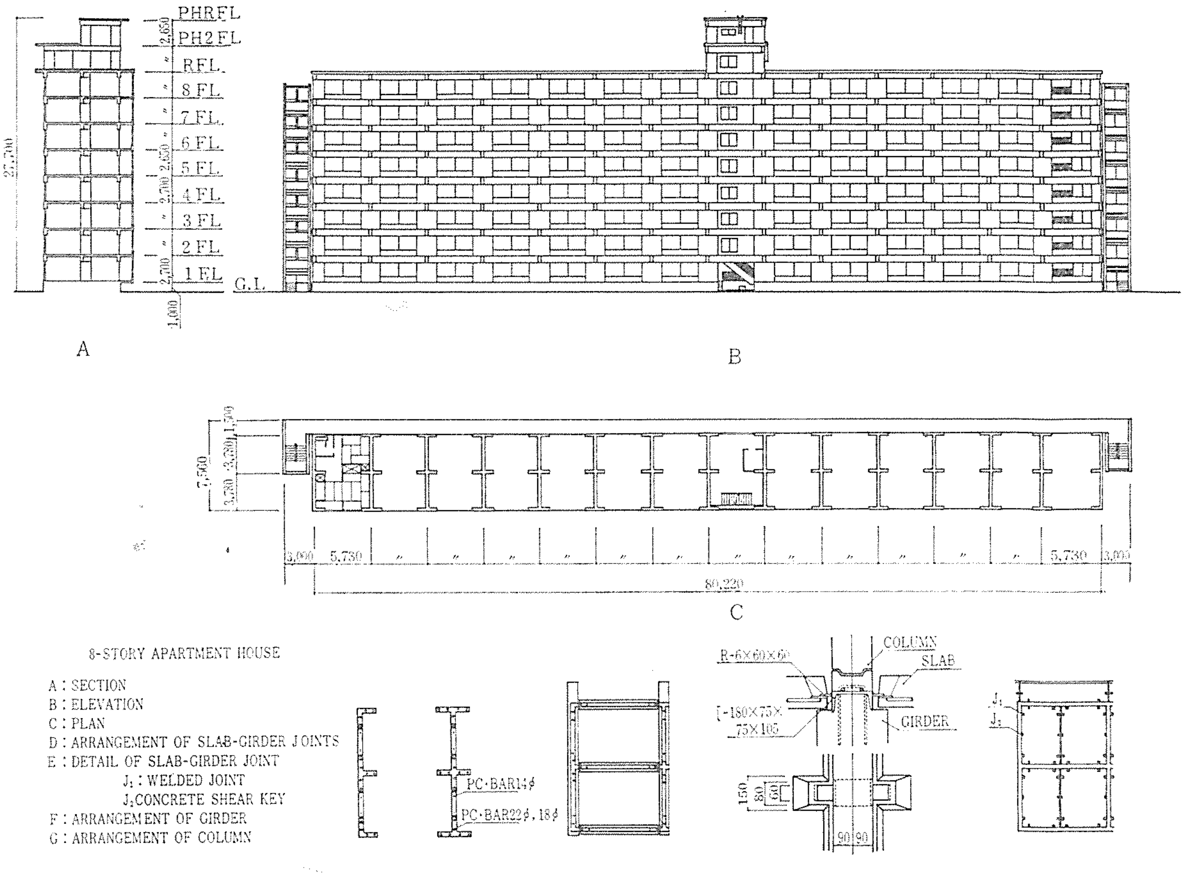


Fig. 2 Column assembly

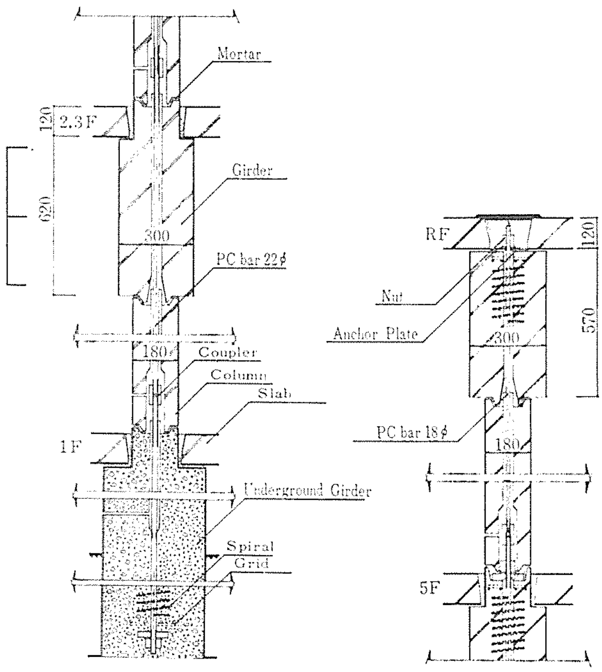
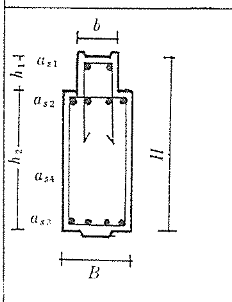


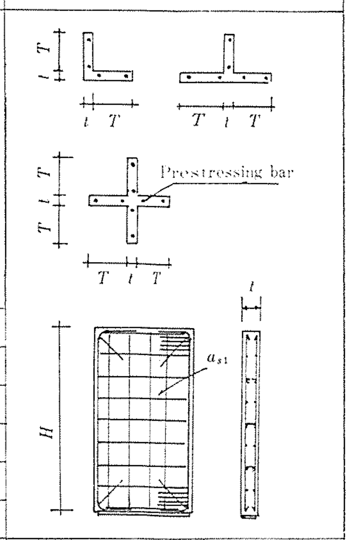
Table 1 Details of elements of 8-story housing structure

Girder



Floor	R～6F	5～2F
B	300	300
b	180	180
H	690	690
h_1	120	120
h_2	570	620
a_{s1}	2−D16	2−D16
a_{s2}	4−D19	4−D25
a_{s3}	4−D19	4−D25
a_{s4}	9φ @250	9φ @250

Column



Floor	8～5F	4～1F
T	900	900
t	180	180
H	1940	1940
a_{s1}	2-9φ @ 225	2-9φ @ 120
Prestressing bar	18φ	22φ

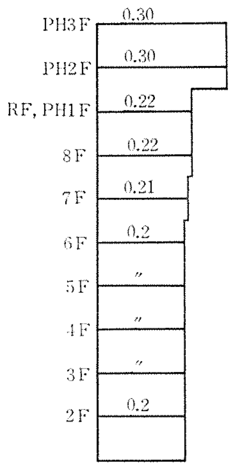
c) The girders have been designed so as to resist design stress as ordinaly reinforced concrete beam.

d) The safety for the flexural failure of columns as ordinally prestressed concrete members has been checked against 1.5 times design seismic force.

3) Structural calculation of the transverse frame works.

The transverse frame works have been considered as monolithic shear wall. However the concentration of shear force to the girder has been considered for the condition after the loss of load carrying capacity of vertical joint between I shaped walls and $+$, L and T shaped walls.

Fig. 3 Distribution of design seismic force coefficient



(3) Some other details of structure

The above mentioned structure has following structural properties

1) Mean shearing stress of the lst floor column is 8.13 kg/cm^2 for longitudinal direction and 5.84 kg/cm^2 for transversal direction ; Mean shearing stress are defined by Q/A_w, where Q corresponds to the base shear force under the action of the design seismic force and A_w to the total sectional area of the lst floor column as rectangular section excluding the sectional area perpendicular to the considering direction.

2) The steel ratio of prestressing bars is 0.39 percent for lower 4 stories and 0.26 percent for upper 4 stories.

3) The ratio of lateral reinforcement of column is 0.63 percent for lower 4 stories and 0.31 percent for upper 4 stories.

4) The average normal stress of the column due to effective prestressing force is about 24 kg/cm^2.

3. Test on a 4-story full-size structure under horizontal static and dynamic force.

(1) Purpose of the test

The main purpose of the present test is to obtain the safety factors of the 8-story housing structure against failure and cracking under seismic force and to provide information about the load deflection characteristic which can be used in a non-linear dynamic analysis of earthquake response.

(2) General description of the test structure

1) The test building was a 4-story frame structure with plan dimension of 4.59 m×11.7 m and consisted of full size structural elements almost same as those of lower 4 storys of 8-story housing structure. A plan and elevation of the test structure are shown in **Fig. 4** and **Fig. 5**. The footing of the test structure was cast in-situ and anchored to the double-decked

Fig. 4 Plan of test structure

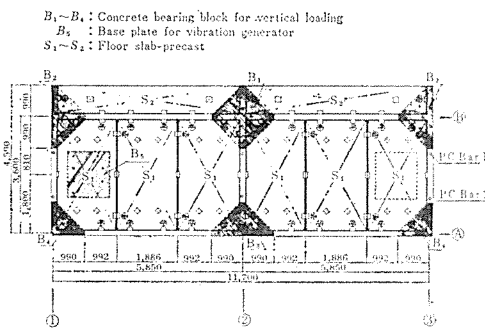


Fig. 5 Elevation and some details of test structure

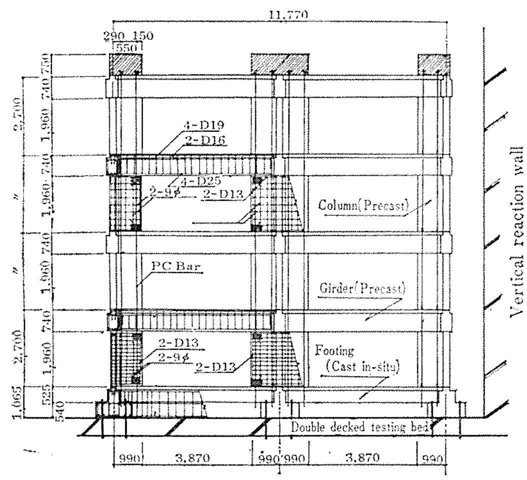
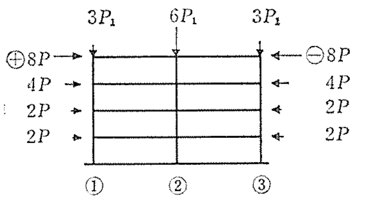


Table 3 Schedule of loading

Loading	Maximun observed horizontal load P^* (ton)	Loading stage	Vertical load $P_1{}^{**}$(ton)
D 1			30 or 0
S 1			30
S 2	± 6	±0.8 D	30
S 3	± 9	±1.2 D	30
S 3′	±18	±2.4 D	30
D 2			30 or 0
S 4	+18	+2.4 D	① 30~10, ② 30, ③ 30~50
S 4′	−18	−2.4 D	① 30~50, ② 30, ③ 30~10
S 5	+27	+3.6 D	30
D 3			30 or 0
S 6	−27	−3.6 D	30
S 7	+29.3, −30	+3.91 D, −4.0 D	30
S 8	+28.9, −26	+3.86 D, −3.47 D	30
D 4			30 or 0



* : Horizontal load per one jack
** : Vertical load per one jack

testing bed with 87 prestressing bars of 27 mm in diameter. The prestressing bars for column assembly were anchored in this footing.

2) Properties of material used

The mechanical properties of concrete, joint mortar, mild steel reinforcements and prestressing bars are shown in **Table 2**.

3) Test procedures

The tests had been carried out under horizontal static and dynamic loading in accordance with the schedule of **Table 3**. The loadings of S 1~S 8 and D 1~D 4 in **Table 3** indicate the static and dynamic loading tests, respectively. The loading stage, D corresponds to the design horizontal load, that is P=7.5 tons, which is determined by equalizing the mean shearing stress of 1st story column of the test structure in loading stage of D to that of the 8-story housing structure under the action of design seismic force. The vertical load which simulated the dead and live load of the upper 4 floors of 8-story building had been also applied during horizontal loading.

Vertical loading devices : The vertical loading was performed by 12 hydraulic hollow ram jacks positioned on the concrete bearing blocks as shown in **Fig. 6 (a)**. The prestressing bars were anchored in the footing through holes in the each floor. The oil pressure of each jack was lead to the accumulator to eliminate the change of pressure due to vertical displacement of test structure and to hold constant pressure during lateral loading.

Horizontal loading devices : The horizontal loading was performed by sixteen bilaterally operative oil jacks attached to the vertical reaction wall. The horizontal load had been applied both in pulling and pushing directions against the vertical wall.

Dynamic loading devices : Two vibratoin generators were used for longitudinal forced vibration test.

A block diagram of vertical loading devices and the distribution of horizontal loads are

Table 2 Properties of materials used

(1) Concrete

Elements	Strength (kg/cm²)		Elastic modulus (10^4 kg/cm²)	
	Comp.	Tens.	$E_i{}^*$	$E_{1/3}{}^{**}$
L, +, T shaped column	369	34.9	29.7	28.3
Girder	405	33.8	30.5	27.4
Floor slab	387	27.8	32.4	30.3
I shaped wall	449	36.8	34.0	31.5
Footing (Cast in-situ)	301	28.7	—	—

* Initial tangent modulus
** Secant modulus

(2) Joint mortar between column and girder

Joint	Strength (kg/cm²)			
	Cured in air		Cured in water	
	Bending	Comp.	Bending	Comp.
Top floor	57.4	350	90.4	546
3 rd floor	54.2	398	76.5	471
Mean	55.8	374	83.5	509

(3) Mild steel reinforcement

Diameter (mm)	Yield strength (kg/cm²)	Tensile strength (kg/cm²)	Elongation (%)
D*16	3 600	5 300	25
D 25	3 700	5 600	24

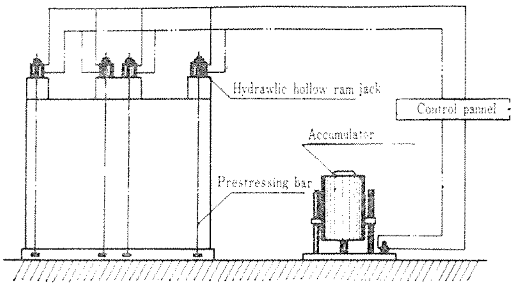
* Deformed bar

(4) Prestressing Bar

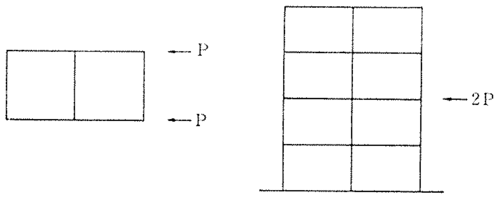
Diameter (mm)	Yield strength (kg/cm²)	Tensile strength (kg/cm²)	Elongation (%)	Elastic modulus (10^6kg/cm²)
D14*	13 660	14 630	8.7	2.08
22 ϕ**	12 860	13 970	8.7	2.045

* Deformed bar,
** Round bar

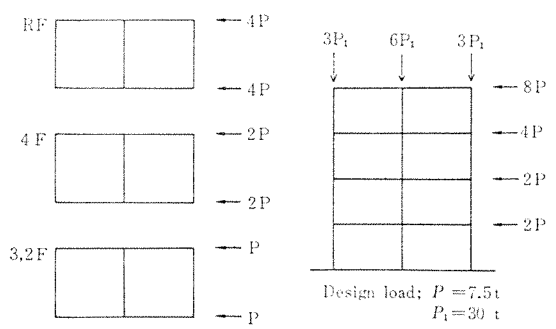
Fig. 6 Details of loading



a) Block diagram of vertical loading devices



b) Distribution of horizontal loads for test S 1



c) Distribution of horizontal loads for test S 2～S 8

shown in **Fig. 6**.

(3) Test results

1) Behaviour of the test structure under static loading

Crack patterns of *B* frame at the loading stage of 1.2 *D* and at the end of test are shown in **Fig. 7** and **8**, respectively.

The total horizontal displacements measured at 2 nd and top floor level are plotted in **Fig. 9** and **10**, respectively, against the applied load. The displacement, δ in these figure indicates the mean value of 4 readings obtained at each floor level with 4 dialgauges and includes the horizontal slippage of joint between columns and girders. This horizontal slippage measured at the bottom of 1st floor column is shown in **Fig. 11**.

The load-displacement curves indicate a fairly linear relation up to 1.2 *D*.

Fig. 7 Crack pattern of *B* frame at 1.2 *D*

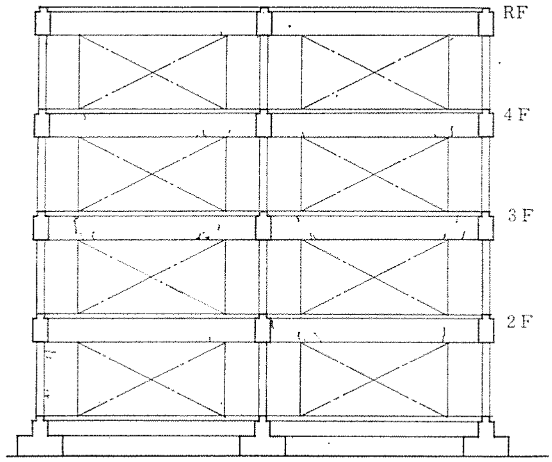
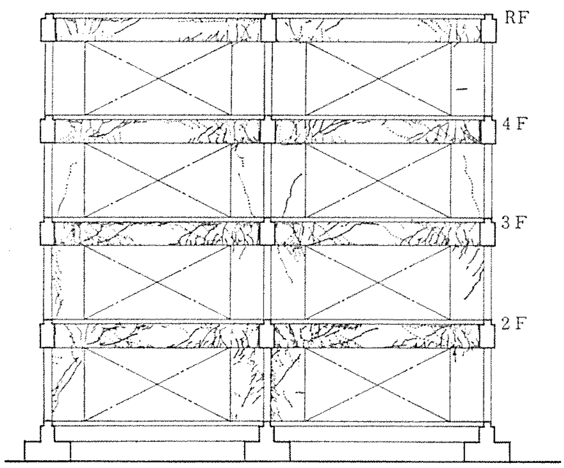


Fig. 8 Crack pattern of *B* frame at the end of test



At this loading stage, no cracks of columns but slight bending cracks of girder were observed and the horizontal slippage was only 0.3 percent of total displacement. The de-

Fig. 9 Relation between applied horizontal load and total displacement at the 2nd floor

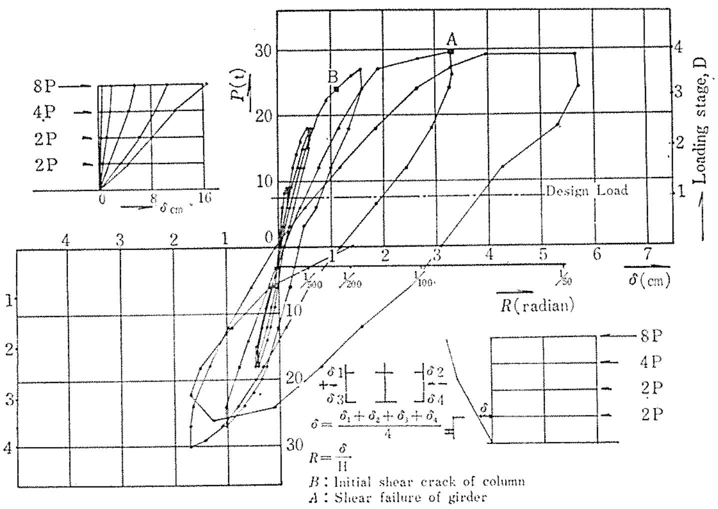


Fig. 10 Relation between applied horizontal load and total displacement at the top floor

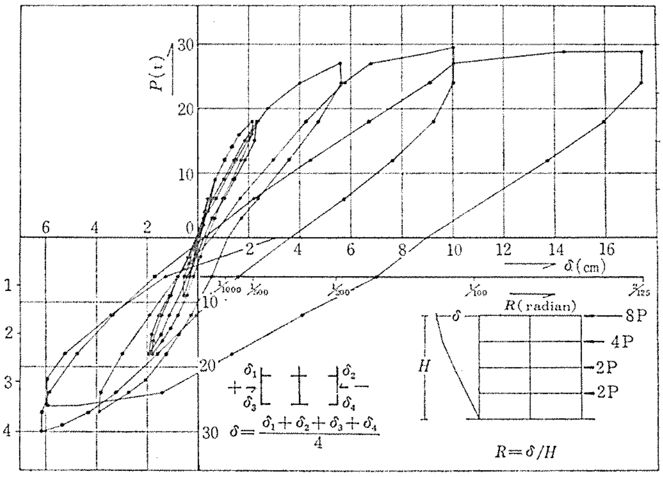
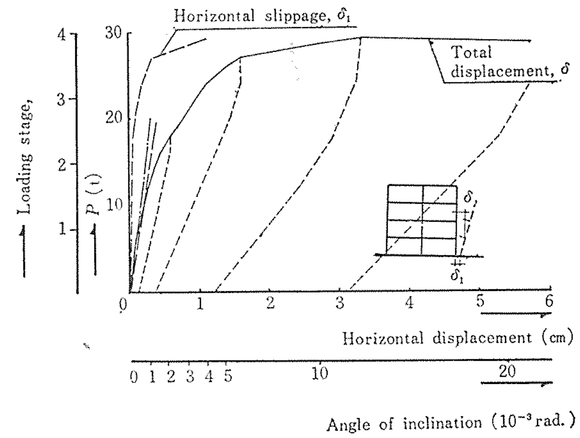


Fig. 11 Relation between applied horizontal load and slippage measured at the bottom of the 1st floor column



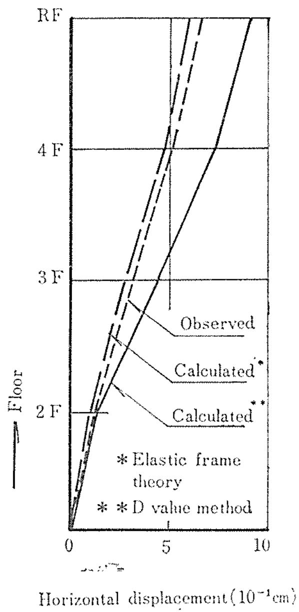
flection predicted by modified elastic frame theory compared very well with the experimental value at this initial loading stage as shown in **Fig. 12.**

The initially linear stiffness of the test structure decreased gradually beyond 1.2 D with gradual increase of horizontal slippage and development of additional bending cracks of girders and columns.

The relative story displacement was 1.72 times the calculated value with elastic frame theory, while the horizontal slippage was 0.9 percent of total displacement at 2.4 D. The maximum crack width at this loading stage was about 0.3 mm at the tensile face of 2nd floor girder.

The bending cracks of girders developed into diagonal shear crack at 2.9 D, while ini-

Fig. 12 Comparison of observed vertical distribution of horizontal displacement and calculated one at the loading stage of 1.2 D



tial diagonal shear crack of columns appeared at 3.2 D or at a mean shearing stress of 27.91 kg/cm^2 in the 1st floor column which corresponded to 7.5 percent of compressive strength of concrete. From the measurement of steel strain of the girders the yielding of tensile steel reinforcement of 2nd and 3rd floor girder started at the loading stage of 3.0 D. The stiffness of the test structure decrease rapidly with the increase of the horizontal slippage, the occurance of above mentioned diagonal shear cracks and the yielding of the tensile steel reinforcements of girders after 3.6 D.

The maximum positive load carrying ca-

Photo 1 Whole view of test structure.

Photo 2 Shear failure of girder

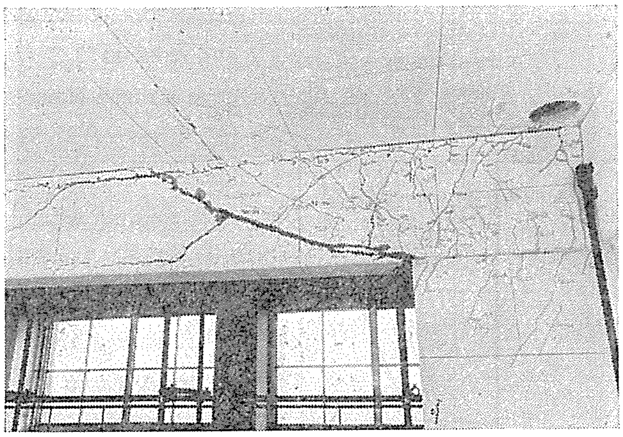
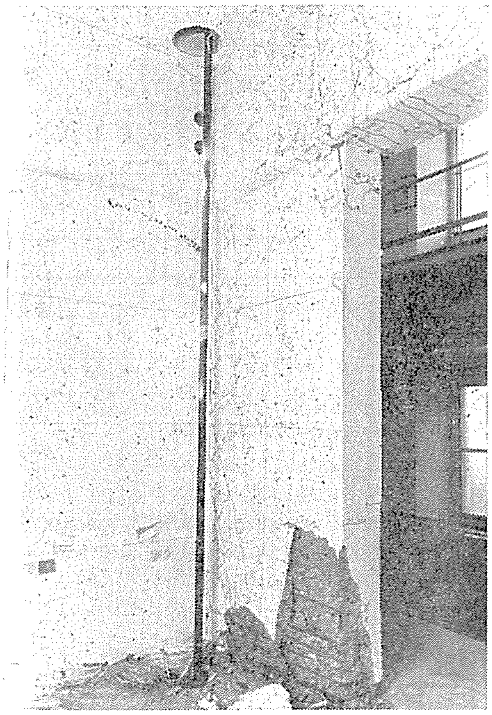


Photo 3 Crack pattern of 1st floor column at the end of test



pacity of the test structure was about $3.9\,D$ or the mean shearing stress of $31.45\,\text{kg/cm}^2$ which corresponded to 8.5 percent of compressive strength of concrete. The final failure mechanism was the shear failure of 2nd and 3rd floor girders as shown in photo 2. Further repeating of load did not cause any considerable decreasing of load carrying capacity.

The angle of inclination, $R=\delta/h$ indicated in **Fig. 9** and **10** was approximately 12×10^{-3} radian at the maximun positive load and 23.8×10^{-3} radian at the final maximum load.

The slippage increased rapidly from the value of 11 percent of total displacement at $3.6\,D$ to 21 percent at the maximum load.

The closely spaced lateral reinforcements at the corner of columns were very effective to prevent the development of shear crack and to avoid the crushing of concrete inside these reinforcement as shown in photo 3.

2) Distribution of shear force

The shear stress obtained by strain measurement of each column are summarized in **Table 4**. The shear force, Qi' shared in each column was obtained by multiplying the A_e as shown in **Fig. 13** by measured shearing stress, τ as shown in **Table 4**. The ratio of total shear force obtained by strain measurement, $\Sigma Qi'$ to the total shear force applied to the test structure, ΣQi, are also listed in this Table. If the column behaves as a member having rectangular section, the ratio, κ will be 1.5. The ratio, κ increase from the value of 1.39 at $1.2\,D$ to 1.46 at $3.6\,D$ with increase of applied shear force. This indicates that the sectional area of column perpendicular to the loading direction make some contribution to bear the shear stress, especially, in the initial elastic range. However, this contribution was about 8 percent of total shear force at $1.2\,D$ and only 2 percent at $2.4\,D$.

3) Distribution of strain due to bending.

In **Fig. 14**, the strain calculated using modified elastic frame theory is compared with measured strain at the loading stage of $0.8\,D$. From this figure, it can be concluded that the modified elastic frame theory can be applicable to such a precast prestressed frame structure with sufficient accuracy within the range of linear stiffness of structure.

4) Calculation of failure moment of column for bending

In **Fig. 15**, the failure moment of T shaped column for bending as a prestressed concrete member in conjunction with axial force is shown with the typical strain distributions in prestressing bars obtained by test.

This figure indicates how the failure mo-

Table 4 Measured shear stress

Flame	Position of strain measurement	P=6 t τ (kg/cm²)	P=6 t Q_i' (ton)	P=12 t τ (kg/cm²)	P=12 t Q_i' (ton)	P=18 t τ (kg/cm²)	P=18 t Q_i' (ton)
A	1	5.01	9.74	8.62	16.06	10.42	20.26
	2-1	11.04					
	2-2	12.53					
	2-mean	11.78	41.68	25.3	90.17	40.53	144.45
	3	6.94	13.49	13.0	25.27	17.21	33.46
	$(\Sigma Q_i')_A$		64.9		131.50		198.17
B	1	5.45	10.61	9.10	17.64	14.08	27.37
	2-1	11.55					
	2-2	12.67					
	2-mean	12.11	43.16	26.11	93.06	42.69	150.00
	3	7.56	14.61	13.73	26.69	23.25	45.2
	$(\Sigma Q_i')_B$		68.3		137.39		222.57
ΣQ_i			133.28		268.89		420.74
$(\Sigma Q_i')_B/(\Sigma Q_i')_A$		1.05		1.05		1.12	
$\kappa=\Sigma Q_i'/\Sigma Q$		1.39		1.40		1.46	

$Q_i'=\tau\times A_e$

τ : Shear stress calculated by strain measurement

Fig. 13 Position of strain measurement for the calculation of shear stress

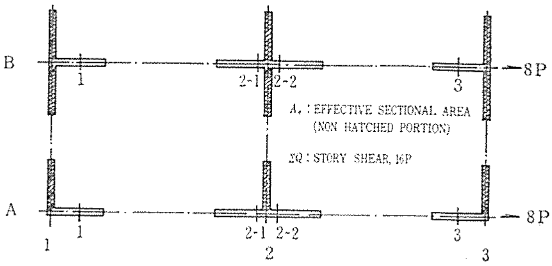
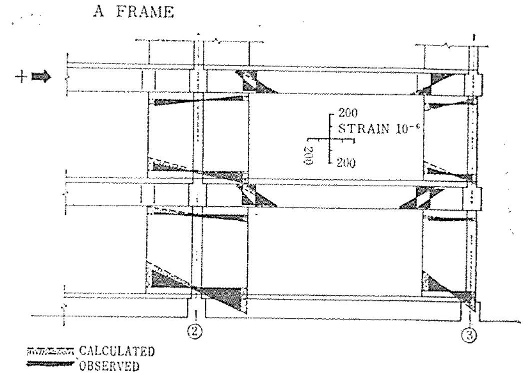
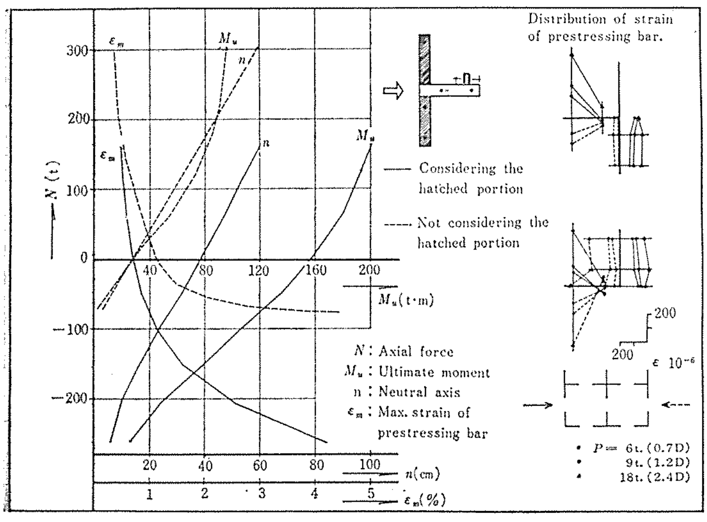


Fig. 14 Distribution of strain due to bending at the loading stage of 0.8 D



ment of column for bending is affected by the assumption of effective width of flange. Though the strain of prestressing bars in flange fell to 80 percent of web strain, it can be presumed from the test results that the strain of prestressing bars in the flange exceeded the yield strain at the ultimate stage. Therefore for the calculation of failure moment of column for bending, the yield stress of prestressing bars in the flange would be applicable.

Fig. 15 Relation between axial force and ultimate moment of column



5) Results of forced vibration tests

The mode, natural period and critical damping obtained by a total of 14 steady- state vibration tests are summarized in **Table. 5**. Test $D1$ was carried out before applying initial static horizontal load; while test $D4$ after reached at ultimate failure. It can be seen from this figure how the natural period and damping increase with the decrease in stiffness of structure ; The natural period at the test $D1$ is 0.123 sec. ; increasing 0.28 at the test D 4. The natural period and mode predicted from the analysis of structure are shown in **Fig. 16** in order to compare with the results from dynamic tests. In this analysis spring constant of structure were calculated ;

Case 1, from the load-deflection properties obtained by static loading test up to 0.8 D.

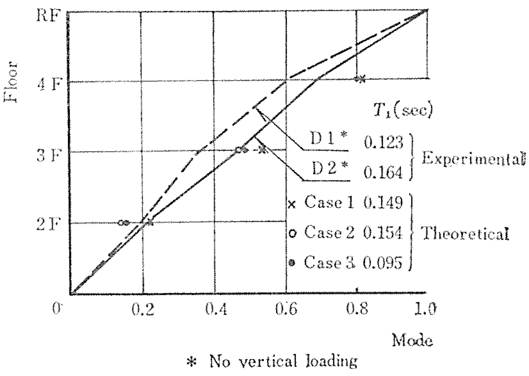
Case 2, from the load-deflection properties obtained by modified elastic frame analysis. In this case, the sectional area of T, L and + shaped walled columns perpendicular to the loading direction was neglected for the calculation of rigidity of columns.

Case 3, from the load-deflection properties

Table 5 Dynamic test results

Test No.	Vertical loading	Exciting Moment (kg·m)	Natural Period (T_1sec)	Critical Damping (%)	Remarks
D-1	× ○	3	0.123 0.125	3.0 3.6	Before horizontal Static loading
D-2	× ○	3	0.164 0.164	2.4 1.4	After loading of 2.4 D
D-3	× ○	3	0.173 0.172	9.3 8.7	After loading of 3.6 D
D-4	×	3 9 12 18	0.270 0.286 0.294 0.294	5.4 6.2 5.8 11.1	After failure
	○	3 9 12 18	0.283 0.286 0.286 0.294	6.1 7.4 5.4 7.0	

Fig. 16 Natural period and mode comparing the observed values with the calculated ones



obtained by modified elastic frame theory. In this case, the sectional area as mentioned above was considered.

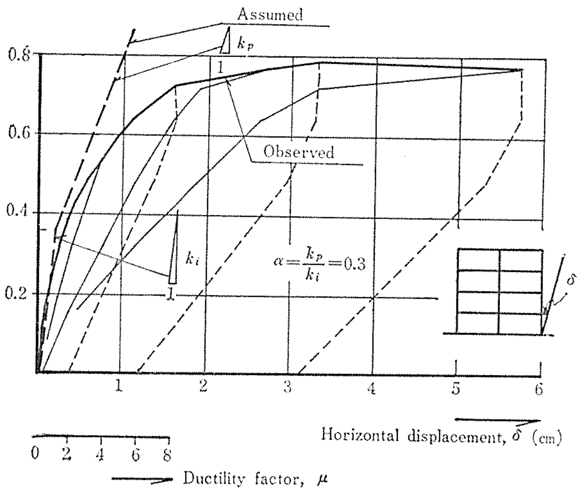
The natural period measured at the test D1 shows the mean value of the calculated results of Case 2 and Case 3, while the natural period at the test D 2 is almost same as the calculated value of case 2. This indicates that the flange of the column wake little contribution to the rigidity of the structure as a whole after application of horizontal load beyond the design sysmic load.

4. Results of dynamic analysis

1) Earthquake response of the longitudinal flume works of 8-story housing structure

A shear type model having eight lamped mass and bi-linear restitutive characteristics was considered for the dynamic analysis of longitudinal frame works of 8-story housing structure. The comparison of assumed bi-linear restitutive characteristic and experimental one is made in **Fig. 17**.

Fig. 17 Bi-linear restitutive characteristic used in the earthquake response analysis of longitudinal frame work



The reduction factors of spring constant, $\alpha=k_p/k_i$ and yield point were determined by referring to the test results. The earthquake response is discussed using three ground acceleration record modifying the maximum intensity to 300 gal. The calculated results are shown in **Fig. 18**. The maximum ductility factors is about 3.0 for El Centro earthquake and the base shear coefficient did not exceed 0.5 in any case. The restitutive characteristics obtained from the test indicate that this structure shows the steady behaviour in such a range of base shear and ductility factor. Therefore it may be predicted that this frame structure will be safe for the earthquake of ground acceleration of about 300 gal.

2) Earthquake response of the transversal shear walls

A combined bending and shear type model having eight lamped mass and linear restitutive characteristic were considered for the earthquake response of transversal shear wall.

In this analysis, the parametric considerations for the rocking motion of foundation were performed using three ground acceleration record modifying the maximum intensity to 200 gal. The story shear coefficients for base and top story are plotted in **Fig. 19**

Fig. 18 Results of earthquake response analysis of longitudinal frame work

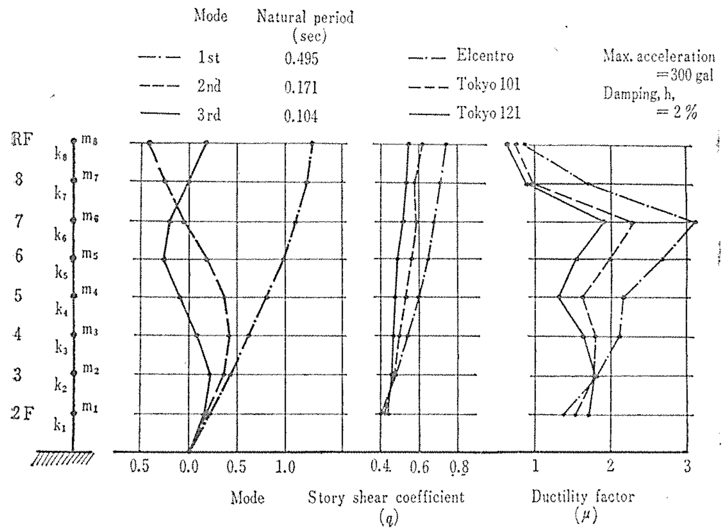
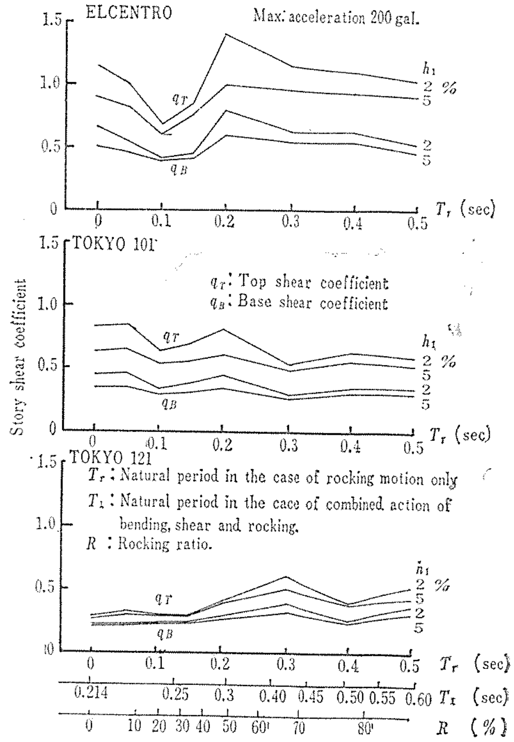


Fig. 19 Results of earthquake response analysis of transversal shear wall



against T_r, T_i and R; T_r corresponds to the natural fre- quency due to rocking motion only obtained under the assumption that the shear wall behave as a rigid body; while T_1 to the natural frequency due to combined action of rocking, shear and bending.

The ratio of the deflection at the top of the structure due to rocking motion only to the total deflection is defined by rocking ratio, R in **Fig. 17**. The probable ratio, R for hard, medium and soft soil condition which corre-spond to the earthquake record of Elcontro, Tokyo 101 and Tokyo 121 respectively would be in the following range.

For hard soil :

$R \leqq 30\%$, that is to say $T_r \leqq 0.15$

For medium soil : $R \leqq 50\%$, $T_r \leqq 0.20$

For soft soil : $R \leqq 80\%$, $T_r \leqq 0.45$

The results of the response analysis show that the base shear coefficient q_B is in the range of 0.2 to 0.5 except in the case of Elcentro earthquake.

5. Concluding remarks

This investigation was concerned with the study of structural behaviour of the 8-story precast prestressed concrete housing structure using 4-story full-size model and earthquake response analysis. The investigation indicates that the above mentioned structure has a sufficient safety in connection with the load carrying capacity and deformation characteristics. For further details of the test results and analysis, the reader, refers to a BRI Research Paper which will be published in the near future.

Acknowledgements

This study was carried out as the project of the committee on housing structure in Building Research Institute. The author thanks the Committee members for providing help and suggestion to carry out this study.

(1970.7.10・受付)