RESEARCH AND DEVELOPMENT ON PREFABRICATED REINFORCED CONCRETE STRUCTURAL SYSTEM

by

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SUMMARY

Research and development on prefabricated reinforced concrete structural system for buildings has been developed in TODA CONSTRUCTION CO., LTD., since 1974, and resulted in the highly aseismic 11-storied apartment house in 1979.

In this paper, the structural design method is introduced first. Secondary the results of the dynamic analysis in longitudinal direction and of the actural scale experiment of columns are discussed.

INTRODUCTION

In such a country as Japan where severe earthquakes occur so often, it is a difficult problem to make high rise reinforced concrete buildings highly aseismic. Therefore, almost all of the reinforced concrete buildings are lower than 20 meters. (about 6-storied)

On the other hand, it is becoming a seriously social problem in Japan that the skillfull laborers on site work are becoming shorter and older.

In order to settle these problems, the research and development project on prefabricated reinforced concrete structural system has been systematized in TODA CONSTRUCTION CO., LTD.

PROFILES OF THE 11-STORIED APARTMENT HOUSE

The profiles of the 11-storied apartment house are; 1) This building is 11-storied without basement and its eaves height, 30.7m. 2) The height and area of the typical floor are 2.7m and 973.0m^2 respectively. 3) It consists of 10 bays with equal span length (6.0m) in longitudinal direction and 3 bays (span length of central bay is 4.15m and of both sides, 4.2m) in transverse direction. 4) All of the members of superstructure are prefabricated, but footing beams and piles are the cast-in-place concrete. 5) The dimension and compressive strength of concrete of members are shown in Table 1. 6) The plan and section in longitudinal direction are shown in Fig. 1 and 2 respectively.

BUILDING COMPONENTS, CONSTRUCTION, JOINTS AND REINFORCEMENTS

The main building component in longitudinal direction was + shape (shown in Fig. 3) and in transverse direction, the shear wall with girder. (shown in Fig. 4)

The constructional process of building is shown in Fig. 5.

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The joints were made as follows; 1) NMB splice sleave was adopted to the joint between column and column. 2) Cadweld was adopted to the joints between column and girder in transverse direction, and between girder and girder in longitudinal direction. 3) The joints at bottom and both sides of shear wall were made by wet joint with concrete.

In order to reduce the number of joints of longitudinal reinforcing bars and to keep the sufficient accuracy on site work, large diameter deformed bars were arranged.

STRUCTURAL DESIGN

The structural design method of this building consisted of two different concepts. One was the moment resisting frame in longitudinal direction, which resisted to the destructive earthquakes by energy absorption in its inelastic characteristics. (i.e., ductile frame) And, the aseismic design was done depending on the flow-chart shown in Fig. 6. The other was the shear walled frame in transverse direction, resisting to them by its ultimate strength.

The structural analysis subjected to seismic force was made by slope-deflection method under following assumptions; 1) Characteristics of all of the prefabricated reinforced concrete members were assumed as the same of the cast-in-place reinforced concrete. 2) Rigid zone was assumed to be located at the each end of columns and girders. 3) Walled frame was transformed into equivalent braced frame. 4) Deflections due to bending moment, shear force and axial force were considered to columns, deflections due to bending moment and shear force to girders and deflection due to axial force to braces. 5) Effects of stiffness of columns and girders by longitudinal reinforcing bars were considered, but of girders by slabs were neglected.

ESTIMATION OF THE ULTIMATE STRENGTH

The unit model was employed to estimate the ultimate strength in longitudinal direction, which was the simplified model of structure consisting of infinite number of bays with equal span length. (shown in Fig. 7) Using this model and process shown in Fig. 6, the base shear coefficients at the ultimate strength subjected to two different seismic force distributions (uniform and inverse triangular) were obtained. These base shear coefficients, longitudinal reinforcing bars of columns and girders, and the distributions of bending moments, shear forces and yield hinges are shown in Fig. 8. Besides, the base shear coefficient obtained by moment distribution method was 0.383. The base shear coefficients obtained could exceed 0.3 set as the target base shear coefficient in this aseismic design. Web reinforcements of columns were calculated by the equation in A.I.J. standard enough to resist the largest shear force obtained and of girders, by equating the flexural capacity of girders in A.I.J. standard.

DYNAMIC ANALYSIS

The model employed in longitudinal direction is shown in Fig. 9. The parabolic flexibility distribution was assumed as the flexibility of members to consider the movement of inflection point of columns. The N-S component of El-Centro 1940, E-W of Taft 1952, N-S and E-W of Hachinohe 1968 in which the maximum acceleration was set equal to 450 gals for elasto-plastic response analysis were employed as earthquake excitations. The damping ratios of structure and swaying were assumed to be 3% and 10% respectively. Degrading Tri-linear Type was adopted to the force-restoring characteristics on the each end of rigid zones and the spring stiffness of swaying. (shown in Fig. 10)

The natural periods and participation vectors obtained are shown in Fig. 11.

The distributions of plastic hinges, bending moments and shear forces obtained by elasto-plastic response analysis are shown in Fig. 12. By E-W component of Hachinohe, 15 plastic hinges were formed at the end of girders from 2nd to 9th floor, but no hinge was formed at the end of columns. The maximum base shear coefficients, story drifts, ductility factors and ratios of the bending moment responsed to the ultimate one of columns are shown in Table 2.

Judging from these results obtained, it has proven that the frame in longitudinal direction has the highly aseismic ability. And, it has also proven that the frame in transverse direction has the aseismic ability enough to resist to the destructive earthquakes by elasto-plastic response analysis. (omitted in this paper)

ACTURAL SCALE EXPERIMENT OF COLUMNS

There are very few experiments of column having D51 deformed bars as longitudinal reinforcements in Japan. Furthermore, there is no experiment of column including joint.

The six actural scale specimens of column having D51 without or with joint were experimented to prove their highly aseismic ability at the structural testing laboratory, University of Tokyo, in 1976.

1) Specimens

The columns of 2nd, 5th and 9th story were selected as the specimens, representing the stress distributions of the lower, middle and upper stories respectively. Web reinforcement of each specimen was obtained by equating the flexural capacity of column in A.I.J. standard, assuming the inflection point was remained at the ultimate strength of unit model. The items and compressive strength of concrete of specimens, the characteristics of reinforcing bars and the examples of specimens are shown in Table 3, 4 and Fig. 13 respectively.

2) Loading apparatus and schedule

Loading apparatus is shown in Photo 1. Loading method was very similar to one proposed by Dr. Ohno and slightly modified to be able to move the inflection point arbitrarily as shown in Fig. 14. The ratio of P_1 and P_2 was controled as shown in Fig. 15.

The alternative cyclic loading was adopted. Loading schedule was decided as follows in consideration of the stress distribution at the ultimate strength of unit model. 1st cycle; ① The inflection point was kept at the center of height. ② The maximum shear force adopted was set to the larger ultimate shear force of unit model. 2nd cycle; ① The inflection point was kept as the same of the ultimate strength of unit model due to uniform distribution. ② The maximum shear force adopted was the same above. 3rd ∿ 5th cycle; ① The inflection point was kept the same above. ② After confirming that the tensile reinforcing bar had reached its yield point by the measurement of strain, it was repeated 3 times being controled by $2\delta_y$ of deflection. (δ_y; yield deflection) 6th ∿ 8th cycle; ① The inflection point was kept the same above. ② It was repeated 3 times being controled by $4\delta_y$. 9th cycle; ① The inflection point was kept the same above. ② More than $8\delta_y$ of deflection was enforced in positive loading.

3) Result and discussion of the experiment

a) Result and discussion of cracking pattern and failure mode

The cracking patterns and load-deflection relationships of specimens are shown in Fig. 16 and 17 respectively.

The cracking patterns observed on the specimens up to 2nd cycle basically consisted of a few number of flexure and flexure-shear cracks. And, short transverse splitting cracks along joint occured on the specimens with joint. As the load and deflection continued to increase beyond yield deflection, the cracks occured before grew in size, and a number of new cracks due to flexure, flexure-shear, shear and bond splitting along the tensile reinforcing bars formed, and longitudinal cracks formed in the compressed concrete. Finally, the shell concrete in the compressive zone near the end of specimens spalled off. When the direction of loading was reversed, the same sequence of events occured.

All of the specimens were failed by crushing of compressive concrete at the end without significant deterioration and never failed by bond split failure which had been worried.

b) Result and discussion of strength and ductility.

The results of cracking, yield and ultimate strength obtained and their ratios to those calculated, the bond stresses of D51 obtained by strain distribution and the ratios of the maximum relative deflections to the clear span length are shown in Table 5, 6 and 7 respectively.
① The averages of cracking strength due to flexure and shear were larger than those calculated and the average due to flexure-shear, smaller. The yield and ultimate strength were larger than those calculated. ② The bond stresses of D51 obtained extremely exceeded the allowable bond stress of D51 (0.8 times to the allowable bond stress in A.I.J. standard), but no effect by bond splitting cracks was observed on the failure mode and load-deflection relationship. ③ In spite that web reinforcement ratio of the specimens of 2nd story exceeded the limit in A.I.J. standard ($p_w \leqq 1.2\%$) and became 1.7%, the strains observed exceeded the yield strain. This result indicated that web reinforcements such as large ratio could show fully to play their role. ④ As shown in Fig. 17 and Table 7, all of the specimens had the sufficient ductility.

CONCLUSION

11-storied apartment house, one of the R & D on prefabricated reinforced concrete structural system for buildings, has been developed through the dynamic analysis and actural scale experiment of columns and resulted in the aseismic enough to resist the destructive earthquakes. And now, the R & D of this system to office and school buildings has been continued.

ACKNOWLEDGMENT

The authors wish to thank Dr. Hajime UMEMURA, emeritus professor of University of Tokyo, Dr. Yasuhisa SONOBE, professor of Tsukuba University, Dr. Hiroyuki AOYAMA, professor of University of Tokyo and Mr. Kazuhiko ISHIBASHI, instructor of Chiba Institute of Technology for their useful advice and criticism throughout this R & D.

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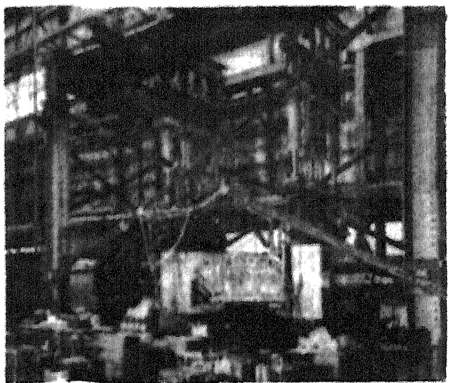

Photo.1 Loading apparatus

Table 2 Results of elasto-plastic response analysis

	ELCE NS	TAF EW	HAC NS	HAC EW	AVERAGE
Base shear coefficient	0.238	0.283	0.303	0.334	0.289
Maximum story drift (cm)	1.43	1.89	2.01	3.89	2.31
Maximum ductility factor	1.12	1.28	1.65	2.27	1.58
M/M_u	0.484	0.569	0.603	0.789	0.611

ELCE NS ; NS component of EL CENTRO 1940
TAF EW ; EW component of TAFT 1952
HAC NS ; NS component of HACHINOHE 1968
HAC EW ; EW component of HACHINOHE 1968
M ; Maximum bending moment responsed
Mu ; Ultimate bending moment

Table 1 The dimension and compressive strength of the concrete of members

Member	Dimension	Compressive strength of the concrete
Column	650 × 750	300 kg/cm²
Girder (1)	450 × 700	″
Girder (2)	450 × 650	″
Shear wall	t= 150	″
Roof	t= 120 ~ 185	″
Slab	t= 120	300
Footing beam	800 × 1500	210
Pile	φ1,000 ~ 1,500	180
Curtain wall	t= 180 ~ 210	210

Table 5 Results of the experiment

No	1st cycle F.C. V1 (t·m)	F.C. V2	F.S.C. V1 (kg/cm²)	F.S.C. V3	2nd cycle S.C. V1 (kg/cm²)	S.C. V4	S.C. V5	F.S. V1 (t·m)	F.S. V6	S.S. V1 (kg/cm²)	S.S. V7	Failure mode
3	34.4	1.45	12.5	0.80	9.2	1.12	0.92	134.3	1.21	11.3	0.85	flexure
1	31.9	0.91	14.8	0.72	14.7	1.40	0.99	159.1	1.15	25.9	1.05	″
4	34.4	0.97	18.3	0.88	15.9	1.50	1.07	183.6	1.32	29.6	1.19	″
2	51.7	1.17	—	—	25.7	1.89	1.30	172.5	1.12	41.5	1.12	″
5	44.1	0.98	23.3	0.92	23.6	1.73	1.18	196.1	1.26	47.2	1.25	″
6	51.7	1.20	23.3	0.89	25.7	1.80	1.26	180.7	1.14	43.4	1.17	flexure

F.C.;flexure crack F.S.C.;flexure-shear crack S.C.;shear crack
F.S.;flexural ultimate strength S.S.;shear ultimate strength
V1;strength obtained by experiment
V2,V3,V4,V5,V6 and V7;the ratio of V1 to the strength calculated by eq. in A.I.J., eq. proposed by Dr. Sozen, eq. proposed by Dr. Arakawa, modified eq. proposed by Dr. Arakawa, eq. in A.I.J. and modified eq. proposed by Dr. Arakawa respectively.

Table 3 Items and compressive strength of concrete of specimens

No.		b × D (cm×cm)	ho (cm)	Fc (kg/cm²)	L.R. pt(%)	L.R.	σo (kg/cm²)	W.R. pw(%)	W.R.	Remarks
1	5	60×70	200	261	0.97	4-D51	42.7	0.74	2-D16@ 90	without joint
2	2	″	″	294	″	″	61.4	1.69	2-D25@100	without joint
3	9	″	″	264	″	″	17.7	0.24	2-D10@100	with joint
4	5	″	″	254	″	″	42.7	0.74	2-D16@ 90	″
5	2	″	″	279	″	″	61.4	1.69	2-D25@100	″
6	2	60×70	200	327	0.97	4-D51	61.4	1.71	2-D22@100 tie 2-D13	with joint and tie hoop

L.R.; Longitudinal reinforcement
W.R.; Web reinforcement

Table 4 Characteristics of reinforcing bars

	σy (kg/cm²)	σmax (kg/cm²)	E (ton/cm²)	elongation (%)
D10	3746	5892	1800	26.9
D13	3775	5604	1810	27.9
D16	3762	5799	1860	27.1
D22	3721	5810	1830	25.1
D25	4247	6157	1890	23.4
D51	3709	5463	1820	16.7

Table 6 The bond stresses of D51

No.		Load Axial Stress (kg/cm²)	Load Shear Force (ton)	Tensile bar (kg/cm²)	Compressive bar (kg/cm²)
3	9	17.7	4th 33.0	44.0	17.0
1	5	42.7	4th 73.0	33.0	30.0
4	5	42.7	3rd 81.0	66.0	12.0
2	2	61.4	4th 115.0	35.0	35.0
5	2	61.4	3rd 146.0	39.0	44.0
6	2	61.4	3rd 128.2	53.0	31.0

Table 7 Deformability of specimens

STORY	with joint	without joint
9	4.6 (No.3)	
5	5.3 (No.4)	5.0 (No.1)
2	3.8 (No.5) 6.6 (No.6)	6.4 (No.2)

($\times 10^{-2}$ rad.)

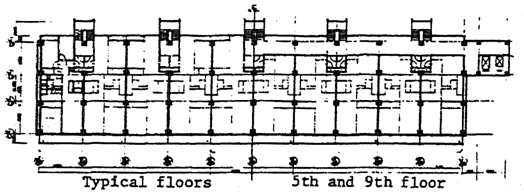

Fig. 1 The plan

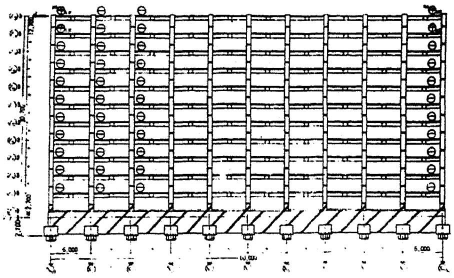

Fig. 2 The section in longitudinal direction

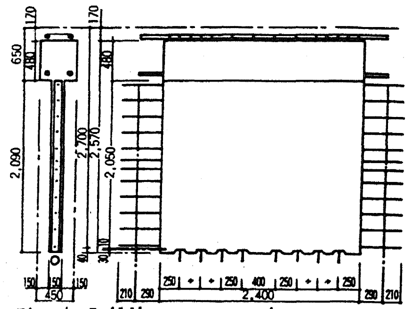

Fig. 4 Building component in transverse direction

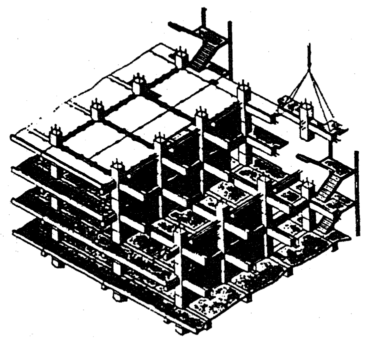

Fig. 5 The constructional process of building

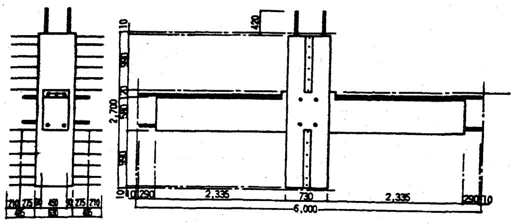

Fig. 3 Builaing componenc in longitudinal direction

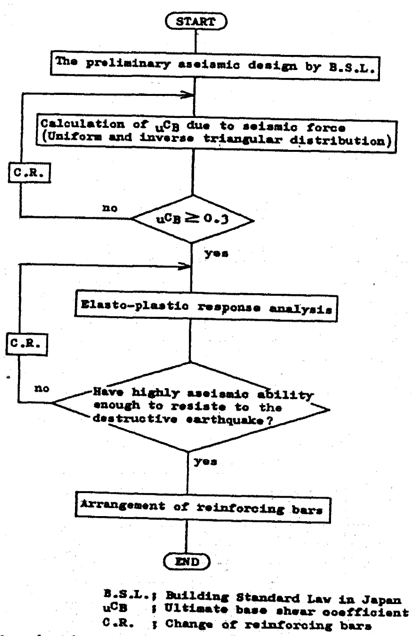

Fig. 6 Flow-chart for the aseismic design in longitudinal direction

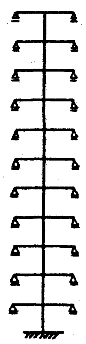

Fig. 7 Unit model

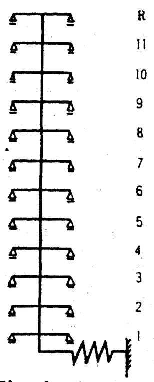

Fig. 9 The model for dynamic analysis

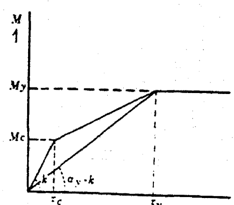

Fig.10 Degrading Tri-linear Type

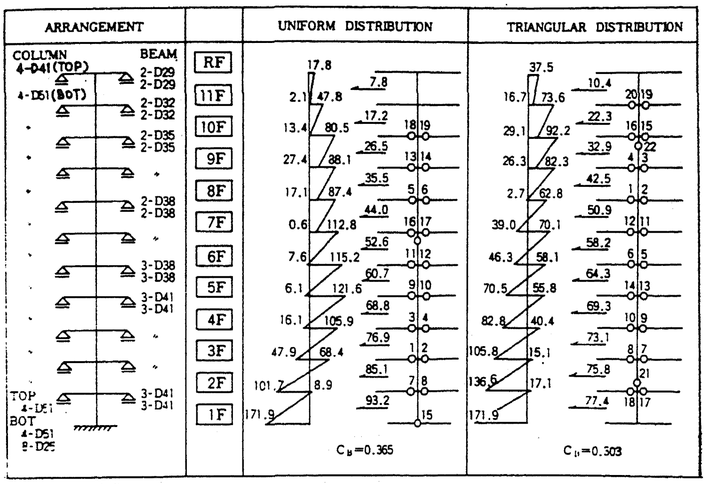

Fig. 8 The state at the ultimate strength

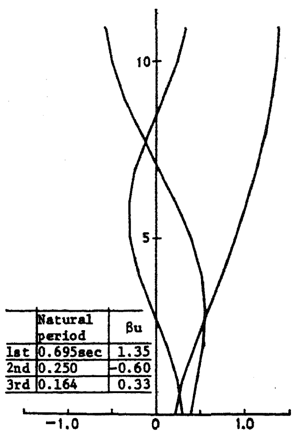

Fig.11 Natural periods and participation vectors

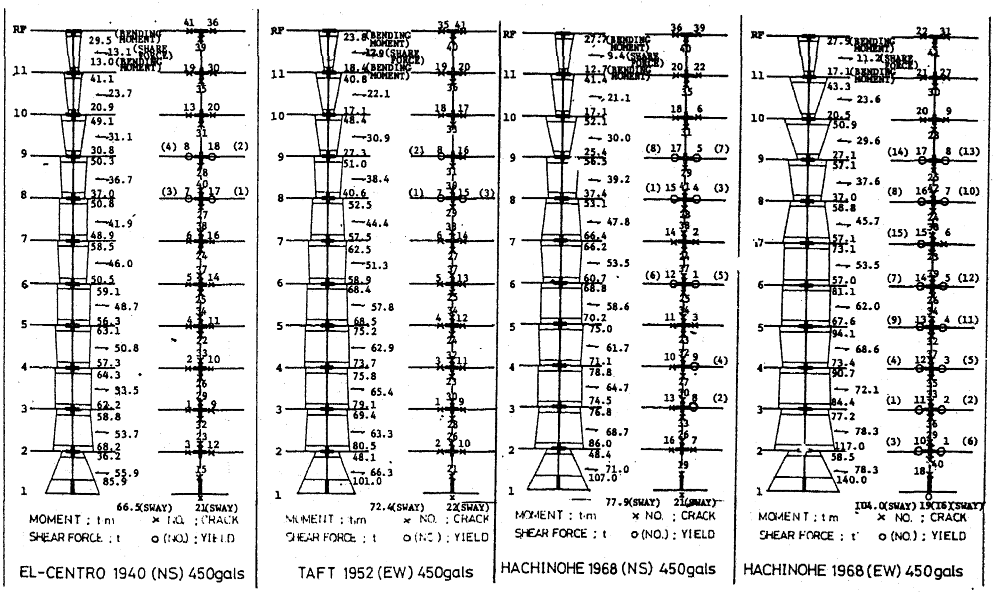

Fig.12 The results by elasto-plastic response analysis

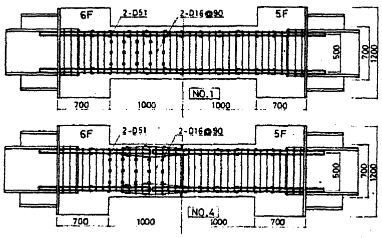

Fig.13 Examples of specimens

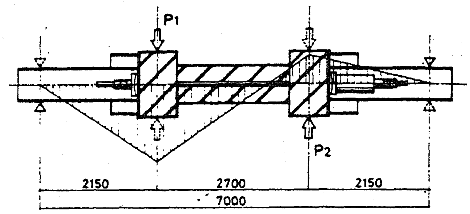

Fig.14 Loading method

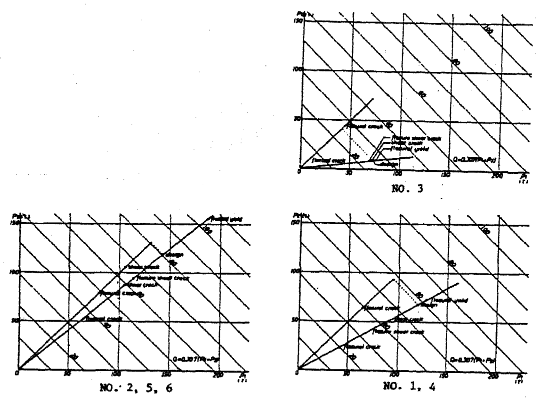

Fig.15 The ratio of P_1 and P_2

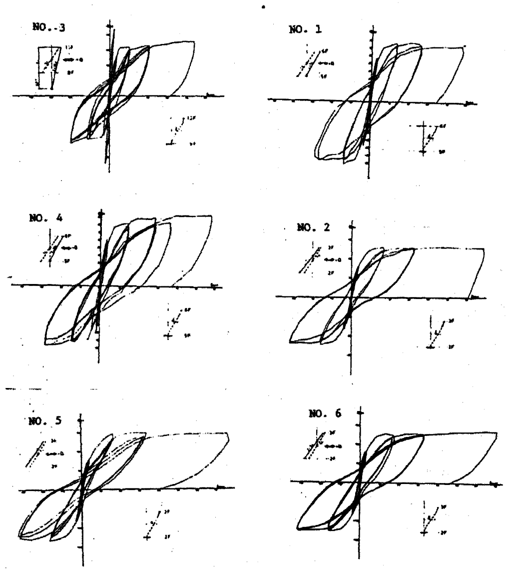

Fig.17 Load-deflection relationships

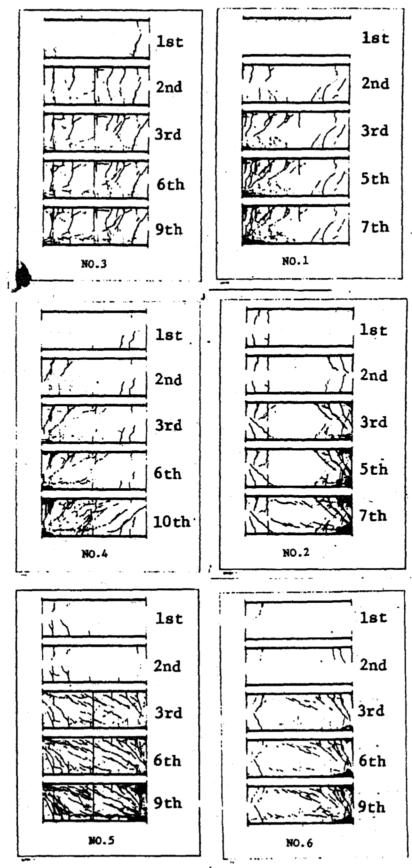

Fig.16 Cracking patterns