EXPERIMENTAL STUDY ON STRENGTHENING REINFORCED CONCRETE STRUCTURE BY ADDING SHEAR WALL

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SUMMARY

In order to contribute the improvement of aseismic properties of existing reinforced concrete low-rise buildings, inelastic behaviors of 13 one-bay, one-story reinforced concrete frames (with poor web reinforcement columns) strengthened by adding various shear walls were investigated experimentally using about one third scale specimens.

The increasing ratios of maximum strength after strengthening to pure frame were about $1.31 \sim 4.30$. Stiffness and strength of all specimens were analyzed by means of inelastic frame models. The shear force to horizontal displacement curve obtained by analysis comparatively agrees with that by test in every specimen.

INTRODUCTION

In the earthquake countries, the reinforced concrete buildings must be provided adequate strength, ductility or both of them against large response earthquake force. Recently, some of the reinforced concrete buildings, especially those built more than 10 years ago (when Reinforced Concrete Structural Standard of Architecture Institute of Japan was revised) were evaluated as not always secure for shearing force of column at severe earthquake in Japan. Therefore, the reliable, easy and short-term strengthening methods for existing reinforced concrete buildings are necessary and the development of these methods is desired in our society.

Following two basic policies would be considered for strengthening the existing reinforced concrete buildings;

1) increasing the ultimate strength of the reinforced concrete building.
2) increasing the ductility of the reinforced concrete building for absorbing earthquake energy by plastic deformation.

In this paper, considering these policies 10 types of strengthening methods were adopted. The existing one-bay, one story frames with poor web reinforcement columns were infilled with precast concrete panels, steel bracing, steel frame and so on, and the effects of them were investigated by tests under static, lateral cyclic loading reversals. Then the behaviours of all specimens were analysed using inelastic frame models.

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TEST SPECIMENS

About one third scale models of one-bay, one-story reinforced concrete frames, which had poor web reinforcement column, were strengthened by the following methods; 1) infilled reinforced concrete wall cast in place 2) precast concrete wall panels in frame 3) precast concrete wall panels with door openings in frame 4) steel bracing in frame 5) steel frame in frame 6) steel truss in frame.

10 types of strengthening techniques which used different elements such as those were selected. Then 13 specimens including two pure frames and the monolithic wall were provided as shown in Table 1, and they were tested at laboratory of Tokyo Metropolitan University.

Their dimensions and details are shown in Fig. 1 and Table 1. The columns and beam had comparatively poor reinforcement ratios for flexure and shearing. Particularly, the web reinforcements of columns were poor. Therefore, columns may fail in shear and beams may fail in bending under severe horizontal load.

The details of the wall panel with connector to frame is shown in Fig. 2. For all specimens but No. 9-C40, the connection of adding precast concrete panel to existing members were filled up with expanding mortar.

The physical characteristics of materials used in this test are shown in Table 2.

TEST PROCEDURE

To investigate the cyclic behavior of strengthened frames, each specimen was subjected to similar sequences of static reversed cycle deflections.

Fig. 3 shows the complete test set-up with Specimen No. 3-C3 under loads. The specimens were loaded laterally in both side using two 50-ton hydraulic jack under constant axial loads N, which was 12 ton (σ_0 = 30 kg/cm^2) for each column.

The horizontal deflection at the top of the specimen were measured using displacement transducers. Besides, the strains of the steels and the widthes of the cracks were measured using electric resistance strain gauges and crack scale respectively.

TEST RESULTS

Initial stiffness, yielding load, maximum and ultimate load with deflection at each test are summarized in Table 3. Fig. 5 shows the load-displacement responses for the complete loading hysteresis loop of the 14 specimens. Moreover, the crack patterns of all specimens at the ultimate state are shown in Fig. 5. Several findings are illustrated;

1) Maximum Strength ..... The lateral load carrying capacities of all strengthened specimens came between that of the pure frame and that of the monolithic wall. Particularly, the strength of specimens with precast concrete panels without opening or with reinforced concrete infilled cast-in-place wall were heightened and were from three to four times as much as the

pure frame specimen. Therefore, these walls will be useful for increasing the strength of structure. Conversely, some of the specimens did not show the notable effect for increasing the strength, for example No. 6-C2A, No. 7-C2B, No. 9-C40.

2) Strengthening Effect ..... Considering ductility together with strength, C_E factor defined as the following formula is used in this paper as an index of antiseismic performance and shown in Table 3,

$$C_E = \frac{Qu}{2N} \times \sqrt{2\mu-1}$$

In which Qu and 2N means ultimate horizontal load and total axial load simulated gravity force respectively, and μ means ductility factor obtained from the test.

The specimens strengthened by steel bracing and steel frame had large C_E factor as much as the precast concrete walls. Namely, No. 3-C3, No. 10-SB, No. 9-C40 and No. 11-SF had large C_E. On the other side, No. 12-ST, No. 7-C2B had the lower C_E factor.

ANALYSIS

1) Analytical model; Each model of those specimens is shown in Fig. 5. In this paper, the columns and beam are considered as the rigid-jointed frame model element. The wall panel, the additional wall or precast concrete panel is idealized as compressive bracing or, compressive bracing plus tensile bracing. Then, the both ends of these bracing are simulated as pin joints with or without springs, and the cross sectional area Ac is assumed as follow;

$$Ac = \alpha_B \cdot t^2$$

where

$$\alpha_B = \left\{ 2a_g \cdot {}_s\sigma_y + {}_w\sigma_y \left({}_va_w + \frac{{}_ha_w}{\lambda^2} \right) \right\} \frac{\sqrt{1+\lambda^2} \cdot \ell}{2A_w \cdot t \cdot F_c}$$

but, when α_B exceeds 5.62, it is assumed as $\alpha_B = 5.62$.

t; thickness of wall. a_g and ${}_s\sigma_y$; gross cross sectional area and yielding stress of longitudinal reinforcement in each column.

${}_w\sigma_y$; yielding stress of reinforcement in wall. ${}_va_w$ and ${}_ha_w$; gross cross sectional area of the vertical and horizontal reinforcements in wall anchored to beam and column respectively.

$$\lambda = \frac{\ell}{h}$$

h; height of wall panel. ℓ; length of wall panel or connected panels.

A_w; horizontal cross sectional area of wall. F_c; compressive strength of concrete.

Besides, concerning to the precast concrete panel, ${}_va_w$ and ${}_w\sigma_y$ are substituted by cross sectional area and tensile strength of anchor bolt. And, because the holizontal reinforcements of precast concrete panel were not anchored, it is assumed as ${}_ha_w = 0.0$.

2) Load-deflection relation of members; The moment-rotation relation at the end of members in the frame and also the stress-strain relation of the additional walls assumed as tri-linear, which is shown in Fig. 4. However, the load-deflection relation of spring, and the axial stress-strain relation of the steel bracing are assumed as bi-linear, shown in Fig. 4.

3) Method of analysis; Stiffness matrices are derived for above-mentioned frame model and analysis are performed by load incremental method. Concerning the yielding of the member, after both sides of a frame member are yielded, the bending stiffness of this member is assumed to be 1/100 initial stiffness.

4) Results of analysis; The calculated load-deflection relations were compared with the envelope (skelton curves) of the experimental load-deflection relations in Fig. 5. Both curves comparatively agree in every specimen.

CONCLUSION

The test results indicated that many methods in this study could be efficient enough to increase the aseismic capacity of a strengthened reinforced concrete frame. Therefore, in case of the frame with ductile beam and weak columns for severe earthquake response force, it is possible to strengthen the frame by easy methods which is adding the precast concrete panels or steel bracing and so forth.

Besides, the inelastic frame analytical method reported in this paper would be useful for determining the skelton curve of load-deflection relation on the strengthened structure.

REFERENCES

1) Lawrence F. K., "Reinforced Concrete Infilled Shear Walls for Aseismic Strengthening", Doctoral Dissertation, University of Michigan, 1976.

2) Lawrence F. K., "Reinforced Concrete Infilled Shear Walls for Aseismic Strengthening, Volume II", Doctoral Dissertation, University of Michigan, 1976.

3) Yoichi Higashi, Masamichi Ohkubo and Yasushi Shimizu, "Experimental Study on the Aseismic Strengthening Methods for Reinforced Concrete Frames", 1979 Japan Concrete Institute 1st Conference.

Table 1. Specimen (p.c.; precast concrete)

No. 1-F1	Pure frame
No. 2-PW	Post casted shear wall in frame
No. 3-C3	Adding 3 p.c. walls
No. 4-C3C	Adding 3 p.c. walls with cotter
No. 5-F2	Pure frame
No. 6-C2A	Adding 2 p.c. walls with center opening
No. 7-C2B	Adding 2 p.c. walls with side openings
No. 8-C4	Adding 4 p.c. walls
No. 9-C40	Adding 4 p.c. walls without infilled mortal
No. 10-SB	Adding steel bracing
No. 11-SF	Adding steel frame
No. 12-ST	Adding steel truss
No. 13-FW	Monolithic wall with frame

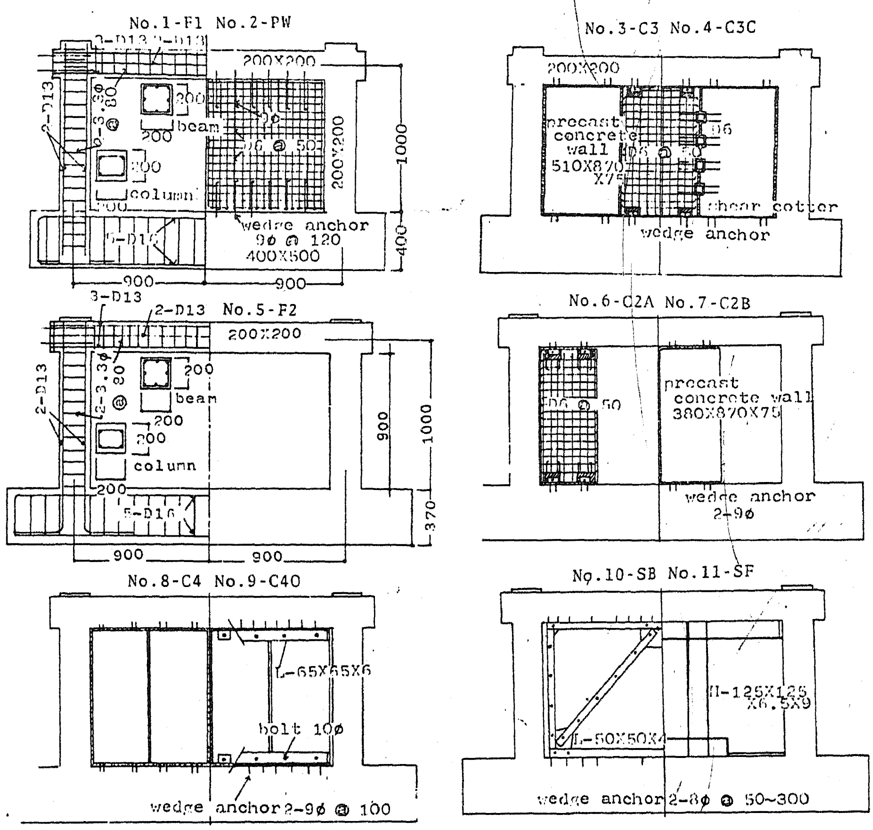


Figure 1. Specimen

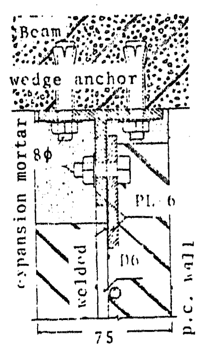


Figure 2. Contact Point between Precast Concrete Wall and Frame

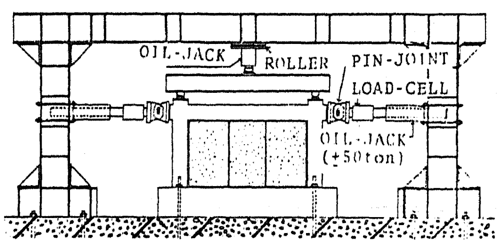


Figure 3. Test Set-up for Specimen No.3-C3

Table 2. Physical Properties of Materials

Concrete (kg/cm^2)

Sprcimen		Fc
frame	No.1-No.4	176
	No.5-No.13	210
No.2-PW (post casted wall)		219
precast concrete wall	No.3,4	242
	No.6 - No.9	228
mortar		474

Fc; concrete cylinder compressive strength at test
$_s\sigma_y$, $_s\sigma_m$; steel yielding and maximum strength

Reinforcing Steel Bar (Kg/cm^2)

Name		$_s\sigma_y$	$_s\sigma_m$
No.1	D13(SD35)	3980	5920
	D6 (SD35)	3430	5710
No.4	3.3ϕ	2320	3320
No.5	D13(SD35)	3960	5880
	D6 (SD35)	3760	5330
No.13	3.3ϕ	5430	6610

Steel Shapes and Plates

Name	$_s\sigma_y$	$_s\sigma_m$
L-50x50x4 (SS41)	4210	6510
H-125x125x 6.5x9 (SS41)	3190	4450
PL-9 (SS41)	3210	4390
PL-1 (SS41)	2890	3520

Table 3. Test Results

Q, δ
(t, cm)

	No.1	No.2	No.3	No.4	No.5	No.6	No.7	No.8	No.9	No.10	No.11	No.12	No.13
Ke	22	455	136	192	26	141	80	493	144	74	44	38	600
Qy	9.6	40.0	20.0	44.0	9.3	12.8	9.5	39.8	9.8	17.6	16.4	14.8	50.0
δy	1.15	0.47	0.45	0.86	0.96	0.50	0.50	0.50	0.20	0.48	0.76	0.77	0.51
Qm	10.7	40.0	33.0	46.0	11.1	15.7	14.5	40.0	16.0	26.1	26.2	18.6	58.0
δm	1.05	0.47	1.89	1.03	1.95	2.00	2.00	0.73	2.00	3.54	4.00	1.97	0.87
Qu	10.7	36.0	33.0	46.0	11.1	14.1	13.2	40.0	15.4	26.1	25.4	18.6	58.0
δu	1.65	0.73	1.89	1.03	1.95	3.35	2.56	0.73	4.00	3.54	5.50	1.97	0.87
C_E	0.61	2.18	3.74	2.26	0.81	2.07	1.67	2.31	4.01	4.03	3.88	1.57	3.75

Ke; Initial Stiffness. (t/cm)
Qy, δy; Yielding Load and Deflection.
Qm, δm; Maximum Load and Deflection.
Qu, δu; Ultimate Load and Deflection.
$C_E = (Qu/2N) \times \sqrt{2\mu - 1}$

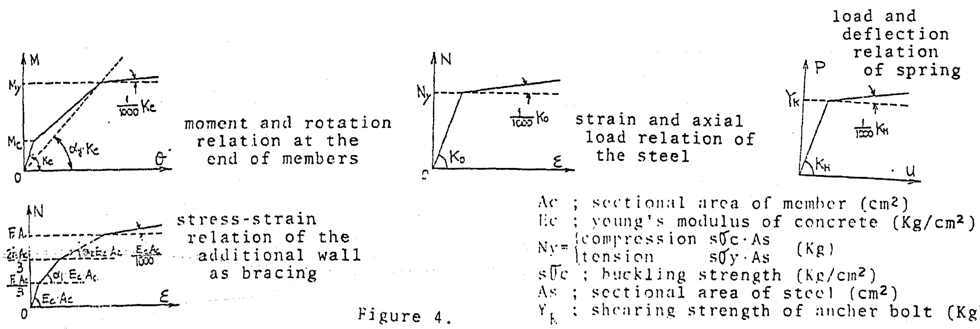
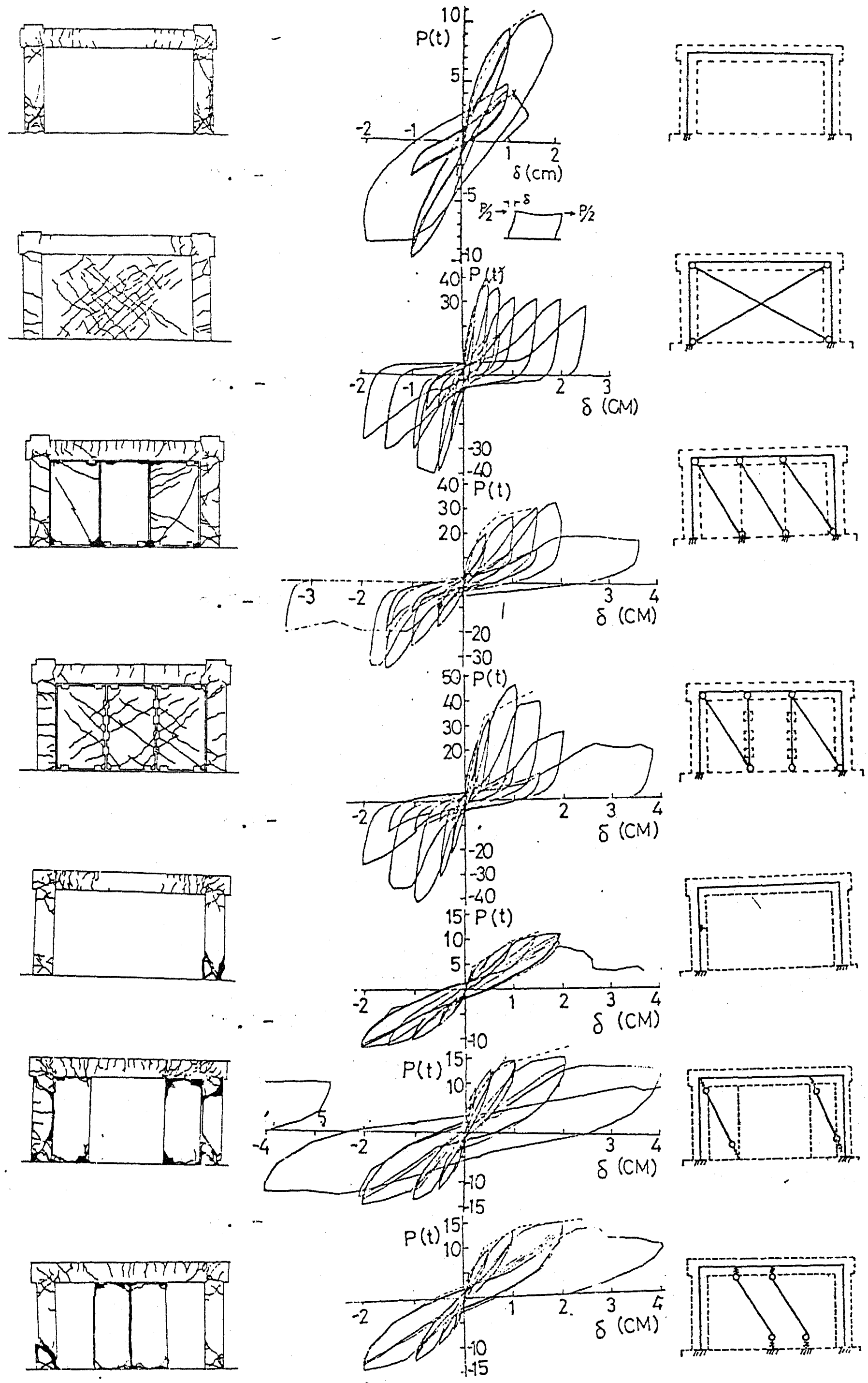


Figure 4.

Ac ; sectional area of member (cm^2)
Ec ; young's modulus of concrete (Kg/cm^2)
Ny = {compression $_s\sigma_c \cdot As$ / tension $_s\sigma_y \cdot As$ (Kg)
$_s\sigma_c$; buckling strength (Kg/cm^2)
As ; sectional area of steel (cm^2)
Y_k ; shearing strength of ancher bolt (Kg)


P(t)
δ (cm)
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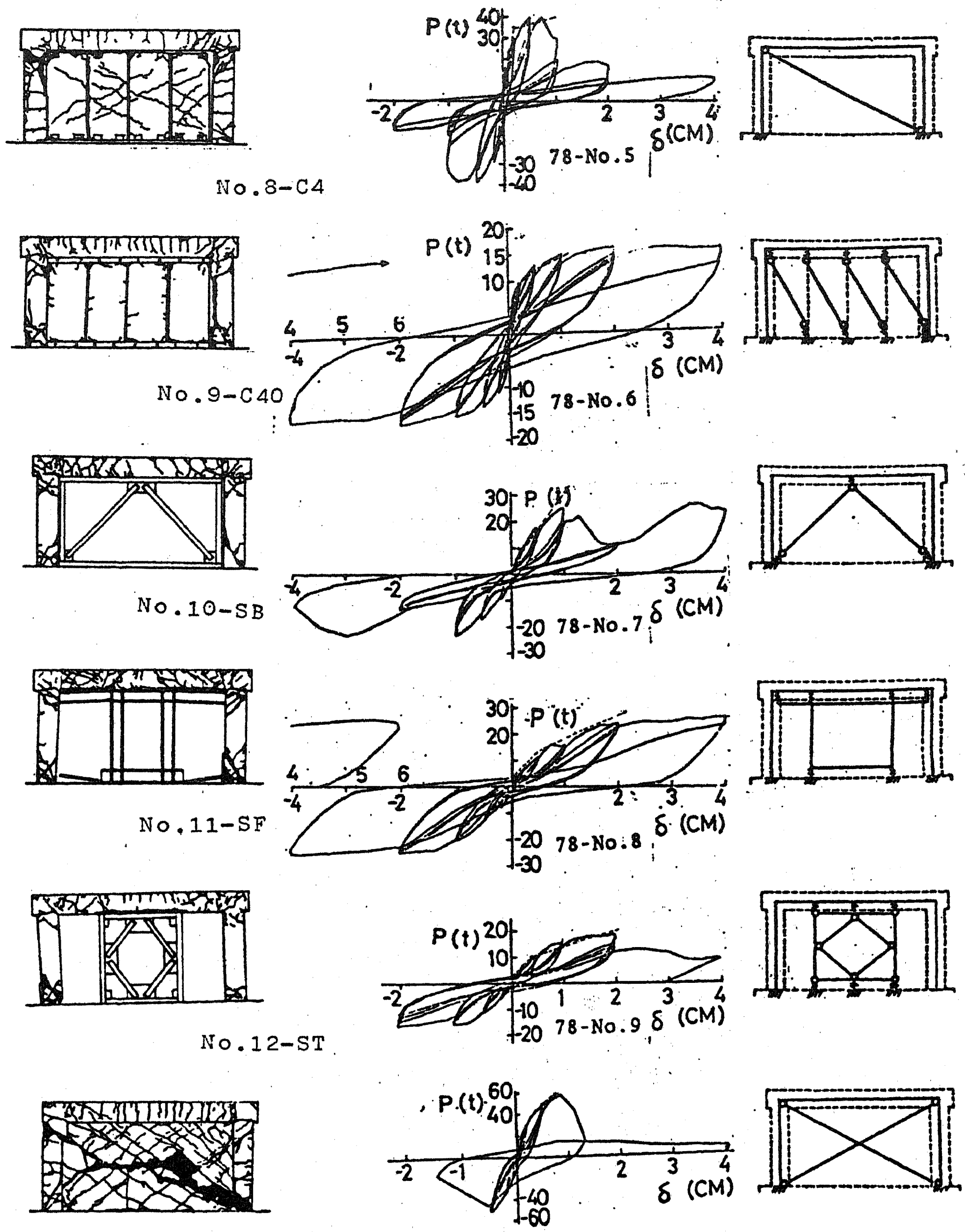

Final Cracking

Load-Deflection Curve
——— experiment
------ analysis (positive side envelope)

Analytical Model

Figure 5-b.