EXPERIMENTAL STUDIES ON RETROFITTING OF REINFORCED CONCRETE BUILDING FRAMES

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SUMMARY

In order to investigate the effects of several types of strengthening for existing reinforced concrete buildings, single-bay three-story frame models strengthened by various methods are tested. The authors had reported the test results concerned the strengthening for single-bay single-story frame models at 7th WCEE. From the past results, what were considered comparatively good are selected for this test. The test results are compared with those of single-bay single-story frame models.

The behaviours of all the specimens are analyzed by means of inelastic frame models. And the load-deflection relations obtained from the analysis agree comparatively well with those from the experiments.

INTRODUCTION

In such a country as Japan in the earthquake zone, the reinforced concrete buildings must be provided adequate strength, ductility or both of these factors against severe earthquakes. Refering to a number of the past earthquake disasters[1], it seems that many reinforced concrete buildings in Japan will be damaged at severe earthquakes. Many studies on evaluating aseismic capacity of existing reinforced concrete buildings[2-4] have also pointed out the same fact. Therefore, the reliable, easy and short-term strengthening methods for existing reinforced concrete buildings are necessary.

A number of buildings of reinforced concrete were strengthened by adding infilled shear walls or wing walls. Higashi, one of the authors, and Kokusho studied on these walls and reported the results in 1975[5]. The authors have studied on the effects of these walls since then. The test results on one-story one-bay frames strengthened by adding those walls were published on the 7th World Conference on Earthquake Engineering in 1980[6].

Considering the author's past studies, six types of strengthening, which indicated comparatively good effects, are adopted in this test. Each existing single-bay three-story frames with poor web reinforcement columns is infilled with precast concrete panels, steel brace, steel frame and post casted wall, and the effects of them are investigated through the test under the lateral cyclic load. The test results are compared with those of

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single-bay single-story frames which were tested in 1977 and 1978. Then the behaviors of all the specimens are analyzed by means of inelastic frame models.

TEST SPECIMENS

Eight models of single-bay three-story frames of reinforced mortar (about one-seventh scale), whose columns have relatively poor web reinforcement (reinforcement ratio of hoops; P_w=0.11%), are strengthened by installing following structural members;

(1) infilled reinforced concrete wall cast in place (79-No.2)
(2) precast concrete wall panels in frame (79-No.3, No.4, No.5)
(3) steel brace in frame (79-No.6)
(4) steel frame in frame (79-No.7)

Eight test specimen including a pure frame (79-No.1) and a monolithic wall (79-No.8) are provided as shown in Table 1. The details of the specimens are shown in Figure 1. The pure frame have 14cm by 7cm columns and beams, and the thickness of the wall panel is 5cm (reinforcement ratio of steel; p_s=0.25%). The details of the connection between the steel or precast concrete walls and the frame are shown in Figure 2.

The precast concrete panels or steel members are fixed to beams by using steel pieces of T-section and wedge anchors. For all the specimens which were filled with precast concrete wall panels in frame but for 79-No.5, the joints between panels and strengthened frames are filled up with expansive cement mortar. Concerning the specimens strengthened with steel braces or frames, the steel angle places and the wedge anchors are welded to avoid the individual breaking of wedge anchors.

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

TEST PROCEDURE

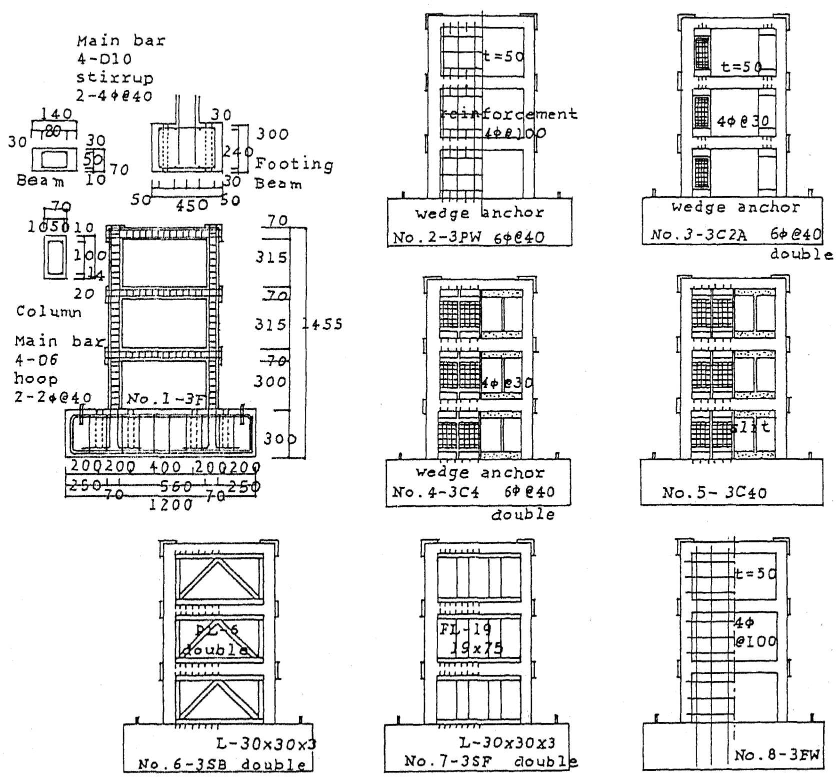
To investigate the cyclic behavior of strengthened frames, each specimen is subjected to reversed cyclic lateral loads. The loading excursion is controlled by selected tip rotation angle(R) of the first story. In this manner the displacement is gradually increased up to the planned displacement. The rotation angles are controlled to R=1/500 for the first cycle, and four complete cycles to R=1/200 and 1/100. In the last loading cycle, the load is increased until a sudden drop in strength which is considered to be the collapse of the specimen or ultimate displacement. The axial load, N is kept 3.0 ton for each column (corresponding to σ_o=30.0 Kg/cm^2 compressive stress).

The complete test set-up for the 79-No.1 are shown in Figure 3. Lateral loads are applied by linked three oil-jacks. The concentrated loads of same intensity are applied at the end of the beam.

At the point close to the center line of beam and column, the horizontal and vertical displacements are measured by means of displacement transducers. The strains in reinforcing steel bars are measured by means of electrical resistance strain gauges.

Table 1. Specimen

	No.1-3F	Pure frame	77-No.1
	No.2-3PW	Post casted shear wall	77-No.2
	No.3-3C2A	Adding 2 precast concrete walls simulated as side walls	78-No.3
79 series	No.4-3C4	Adding 4 precast concrete walls	78-No.5
	No.5-3C40	Adding 4 precast concrete slit walls	78-No.6
	No.6-3SB	Adding steel brace	78-No.7
	No.7-3SF	Adding steel frame	78-No.8
	No.8-3FW	Monolithic wall with frame	78-No.10



unit; mm

Figure 1. Specimen

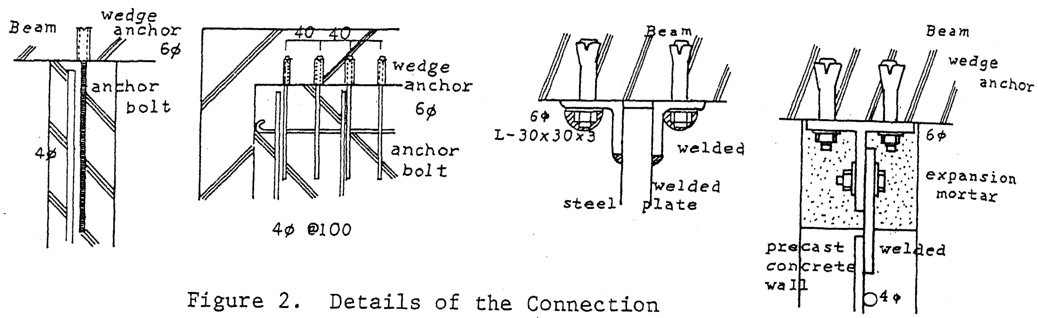


Figure 2. Details of the Connection between Wall Panels and Beams

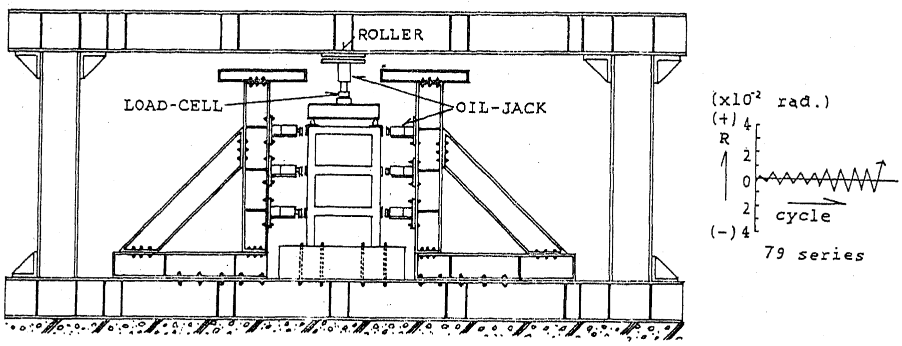


Figure 3. Test set-up for Specimen 79-No.1

Table 2. Physical Properties of Materials

Mortar	$_C\sigma_B$ (Kg/cm^2)
Specimen	144
Filled in mortar	246

Steel Bar	$_s\sigma_y$ (Kg/cm^2)	$_s\sigma_m$ (Kg/cm^2)
2ϕ	2520	3370
4ϕ	4180	4850
D6	3700	5550
D10	3850	5630

Steel	$_s\sigma_y$ (Kg/cm^2)	$_s\sigma_m$ (Kg/cm^2)
L-6	3550	5060
L-19	3700	5010
L-30x30x3	4070	5390

$_C\sigma_B$; concrete compressive strength
$_s\sigma_y$; steel yielding strength
$_s\sigma_m$; steel maximum strength

TEST RESULTS

Comparing this test results with the data of single-story specimens, initial stiffnesses and maximum loads and deflections are shown in Table 3. The load-horizontal displacement curves at the top of the first story are shown in Figure 4 with the final cracking patterns. Three-story specimens with the full width of wall fail in bending because shear span ratio of wall is long, whereas single-story specimens fail in shear. In regard to initial stiffness, strengthened frames have larger value in comparison with pure frame. The specimen strengthened with post casted wall shows highest initial stiffness, and the specimen with the precast concrete wall follows this. The specimens strengthened with the steel braces and frames show low initial stiffness. The specimen with post four cast concrete walls, and that with steel show higher strength in both cases of single and three-story. These strengthened three-story specimens show pretty improvement in ductility.

The specimen with precast concrete side walls does not show the notable effect for increasing the strength, but this method is effective to improve ductility.

ANALYSIS

The behavior of the all specimens were analyzed by using inelastic frame models. Every beam or column of the specimen was used as a line member considering axial deformation and flexure, while every wall was used as a diagonal brace considering only axial deformation. Direct stiffness matrix analysis was used for solving the displacement corresponding to the increment of the horizontal load. This analytical technique consists of the simplified routine method and is widely used to design reinforced concrete buildings. The outlien and the results of the analysis are as follows.

Table 3. Test Results

Single-bay, three-story specimen					Single-bay, single-story specimen				
specimen	Ke t/cm	Pm ton	Pm No.i / Pm No.1	δm mm	specimen	Ke t/cm	Pm ton	Pm No.i / Pm No.1	δm mm
No.1-3F	6	1.95	1.00	9.4	77-No.1	22	10.7	1.00	1.65
No.2-3PW	150	8.64	4.43	7.0	77-No.2	455	40.0	3.74	0.47
No.3-3C2A	13	3.03	1.55	16.8	78-No.3	141	15.7	1.47	2.00
No.4-3C4	45	5.85	3.00	21.2	78-No.5	493	40.0	3.74	0.73
No.5-3C40	55	5.22	2.68	14.9	78-No.6	144	16.0	1.50	2.00
No.6-3SB	45	7.95	4.08	12.0	78-No.7	74	26.1	2.44	3.54
No.7-3SF	35	6.03	3.12	18.8	78-No.8	44	26.2	2.45	4.00
No.8-3FW	155	8.64	4.43	7.0	78-No.10	600	58.0	5.42	0.87

Ke; initial stiffness, Pm, δm; maximum strength and deflection

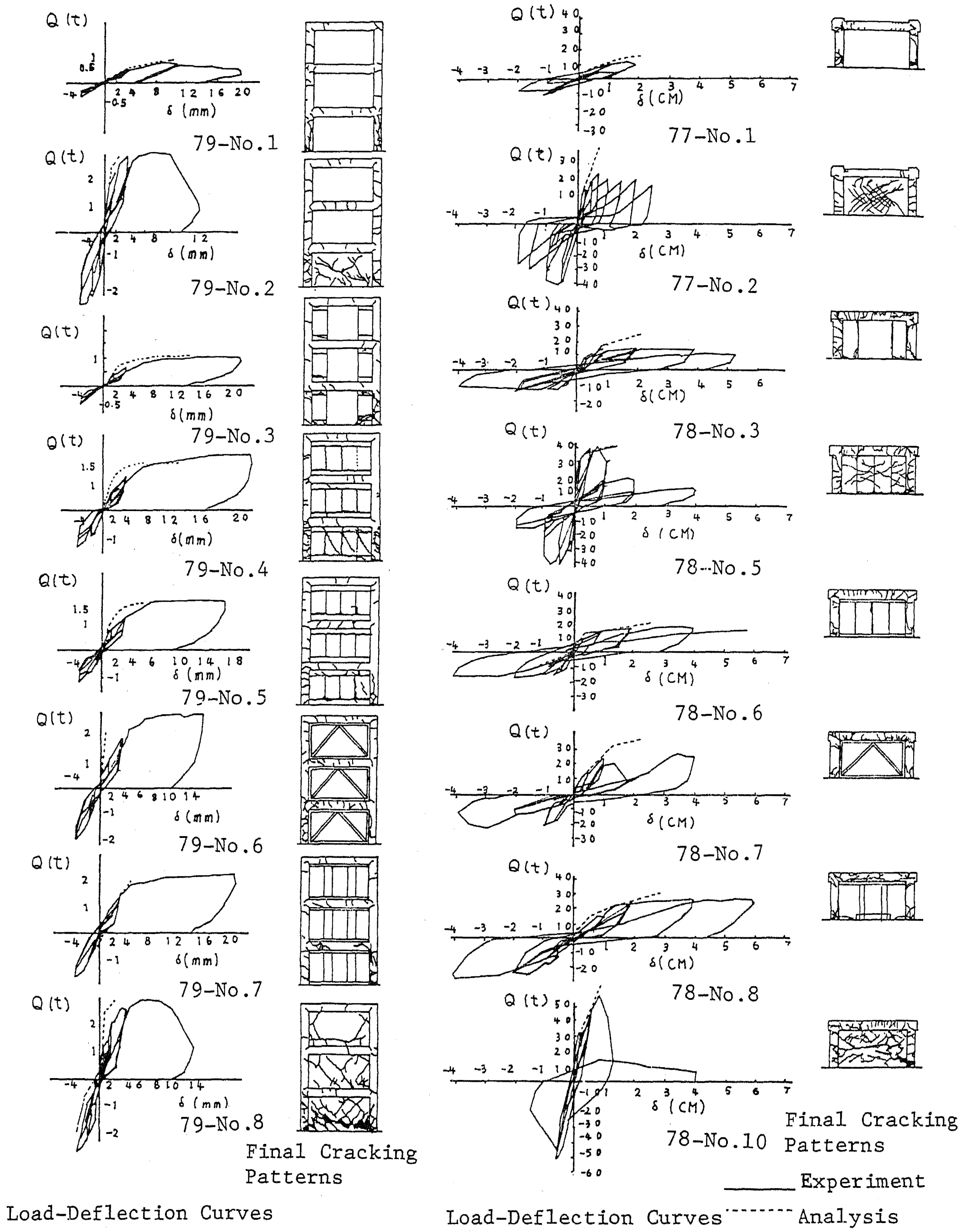


Figure 4. Comparison of the three-story specimens with single-story specimens

The columns and beams are assumed as rigidly jointed elements. The wall panel is idealized as compressive bracing or, compressive bracing plus tensile bracing. Then the both ends of these bracing are simulated as pin joints. Some joints are simulated those with springs, and the others are not. The steel truss is considered as the member of truss with springs, and the steel frame is considered as the members of columns and beams with springs.

Moment and rotation relation at the end of the column of the beam is shown in Figure 5 (a). Yield bending moment in the figure was determined by the interaction curve illustrated in Figure 5 (b). Nonlinear part of the interaction curve was computed by the following equation, proposed by Architectural Institute of Japan.

$$My = 0.8\ a_t \cdot \sigma_y \cdot D + 0.5\ N \cdot D\ (1 - N/b \cdot D \cdot F_c) \quad \ldots\ldots (1)$$

a_t : area of tension reinforcement, σ_y : yield strength of reinforcement, N : axial force, D : overall thickness of member, F_c : compressive strength of concrete.

Stress - strain relation of the brace was assumed as illustrated in Figure 5 (c), referring to the shear deformation characteristics of the corresponding wall. When the shear force of the beam or the column reached the critical shear force computed by the following equation, two yield hinges are formed at the end of the member. The following equation proposed by Arakawa and modified by Hirosawa gives shear strength[3]

$$Q_{su} = \{ \frac{0.092\ k_u \cdot k_p\ (180 + F_c)}{M/Q \cdot d + 0.12} + 2.7\ \sqrt{Pw \cdot w\sigma y} + 0.1\ \sigma_o \}\ b \cdot j \quad \ldots\ldots (2)$$

in which, k_u : modification factor from effective depth, $k_p = 0.82\ p_t 0.23$, $M/Q \cdot d$: shear span ratio, P_w : shear reinforcement ratio, $w\sigma y$: yield stress of shear reinforcement, σ_o : axial force divided by area of the member, P_t : tension reinforcement ratio, b : which of web, j : distance from centroid of compression to centroid of tension in fluxural member.

The relations between the computed shear forces and the first story displacements are shown in Figure 4 by broken lines. In general the computed lines fitted the envelop curve obtained from the experiment.

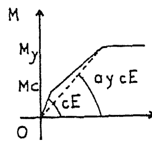
CONCLUSION

Based on the experimental and analytical results on the strengthened reinforced concrete frame tests, several conclusions may be deduced as follows;

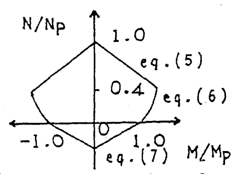
(1) The specimen with reinforced concrete infilled cast-in-place wall showed high strength.

(2) The specimens with steel frame, steel brace and four precast concrete panels showed not only high strength but also improvement in ductility.

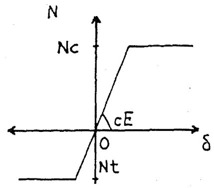
(3) In regard to the single-story specimens, many of them have a tendency to fail in shear, but the three-story specimens have a tendency to fail in bending, because shear span ratio of wall is longer.



(a) Moment and Rotation at the End of Members



(b) Interaction Curve



(c) Axial Load-deflection Relation of the Additional Wall as Bracing

$$M_c = 1.8\sqrt{F_c}\cdot Ze{+}N\cdot D/6 \;\ldots\ldots\; (3)$$

$$M_y = 0.8\ a_t\cdot\sigma_y\cdot D{+}0.5N\cdot D(1-N/bDF) \;\ldots\ldots\; (1)$$

$$\alpha_y = \{0.043+1.64n\cdot P_t+0.043(\frac{a}{D})+0.33\ \eta_o\}(d/D)^2 \;\ldots\ldots\; (4)$$

$$\frac{M_y}{M_p} = \frac{5}{3}(1-\frac{N}{N_p})(1+0.12D\frac{N_p}{M_p}) \;\ldots\ldots\; (5)$$

$$\frac{M_y}{M_p} = 1+0.5D(\frac{N}{M_p})(1-\frac{N}{N_p}) \;\ldots\ldots\; (6)$$

$$\frac{M_y}{M_p} = 1 + (\frac{N}{\sigma_y\cdot {}_sA}) \;\ldots\ldots\; (7) \qquad \left\{ \begin{array}{l} Mp=0.8at\cdot\sigma_y\cdot D \\ Np=F_c\cdot b\cdot D \end{array} \right.$$

$$N_c = F_c\cdot {}_cA \;\ldots\ldots\; (8) \qquad N_c = \sigma_y\cdot {}_sA \;\ldots\ldots\; (9)$$

[Notes] ${}_sE$, ${}_cE$: Young's modulus of steel, concrete, $n={}_sE/{}_cE$, Z_e : section modulus, $\eta_o = N/b\cdot D\cdot F_c$, ${}_sA$: total sectional area of longitudinal reinforcing bar, ${}_cA$: total sectional area of concrete, N_t : tensile strength, N_c : compressive strength.

Figure 5. Assumption of Analysis

(4) The non-linear analysis technique reported in this paper is considered to be efficient for design for strengthening actual building, although this technique consists of the simplified routine method.

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