# THE STRENGTHENING METHODS OF THE EXISTING REINFORCED CONCRETE BUILDINGS

by

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## SYNOPSIS

This paper describes experimental investigations into the effects of strengthening methods of the existing reinforced concrete buildings.

Methods for strengthening techniques are 1) to increase the strength of the buildings by full-filling new concrete shear wall inside the existing frame, and 2) to increase the ductility of the buildings by strengthening the surroundings of the existing columns with mortar and welded wire fabrics.

The remarkable effectiveness was obtained for strengthening methods of the existing reinforced concrete buildings from results of these experiments.

## INTRODUCTION

Several reinforced concrete buildings, designed in accordance with the past regulations and specifications, suffered serious damages due to strong earthquake motion occurred recently in Japan. In order to reduce such serious damages of the buildings, strengthening and stiffening are required for the existing reinforced concrete buildings which can not be evaluated to be safe against the strong earthquakes. In this situation, the strengthening and stiffening methods for the existing buildings have been studied.

The policies of strengthening are as follows:

1) to increase the strength of the buildings --- Experiment 1
2) to increase the ductility of the buildings--- Experiment 2

In order to achieve those purposes, the following experimental studies of two strengthening techniques have been carried out.

## EXPERIMENT 1

This is a strengthening method by the new concrete shear wall full-filled in site with pressure at the inside of the existing frame. It has been recognized that frames with infills have more strength and stiffness than the frames without infills. This experiment is designed to give a quantitative understanding of how infilled reinforced concrete wall interacts with an existing reinforced concrete frame and plays a role in improving the strength and stiffness of the frame.

### a) Test Specimens

Test specimens consist of 6 types as shown in Fig.1. The about one-third scale models are one-story and one-bay structures that use the same

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frame. The thickness of walls is 7.5cm.

Specimen W-1 is a rigid frame without infilled wall, 180cm span length, 100cm hight and 20cm x 20cm cross section of the column and beam. Specimen W-2 is a monolithic wall with the surrounding frame. Specimens W-3,4,5 and 6 consist of rigid frames strengthened by a new concrete shear wall full-filled inside the existing frame. The concrete of infilled wall was applied directly within the mould with pressure of $0.3kg/cm^2$ by pumping. These four specimens are different in joining methods by mechanical shear connectors between the new infilled concrete wall and the surrounding frame.

The joining methods by mechanical shear connectors are as follows:

1) small concrete shear keys, whose height, length and width are 2cm, 4cm and 7.5cm respectively, connecting to the rigid frame with binding agent. ---Specimen W-3.

2) steel pieces as shown in Fig.2, anchored only under the upper beam and roughening the other three inner sides of the concrete frame. ---Specimen W-4.

3) steel pieces anchored at all the inner sides without roughening ---Specimen W-5.

4) steel pieces anchored at all the inner sides and roughening all the inner sides except the upper beam. ---Specimen W-6.

The arrangement of reinforcement for the frame is shown in Fig.3, and the tensile reinforcement ratio, $p_t$, is 1.27% and 1.91% for column and beam respectively. The shear reinforcement ratio, $p_w$, is 0.11% for column and beam, too. The wall reinforcement ratio is 0.75% for all specimens as shown in Fig.4.

The mechanical properties of materials are shown in Table 1. The strength of the infilled concrete were obtained from cylinder specimens that were dug out in the wall constructed under the same condition and pressure of the infilled test specimens.

### b) Loading and Measuring Method

The testing facility is shown in Fig.5. Each specimen was fixed to the base of loading frame with high strength steel bolts.

The axial load, 12ton, was applied at the top of each column through a load cell by an oil jack with a maximum capacity of 50ton. The lateral load was applied reversally at the either side on the centerline level of the beam through a load cell by two reversal oil jacks with a maximum capacity of 50ton. The specimen was pushed at one side and pulled at the other side at the same time on each loading cycle.

The horizontal relative displacements were measured between the center level of the beam and the basement by electric gages.

### c) Experimental Results

The calculated values (1) and the experimental results (2) of initial stiffness and various strengths are shown in Table 2.

Fig.6 shows the relations between load and deflection of all specimens. Fig.7 shows the envelope of the load-deflection curves.

The final failure mode of Specimen W-2, the monolithic wall and frame, was shear failure with numerous shear cracks sprending over the whole wall. In Specimen W-3, using the shear connector of small concrete keys, the concrete keys under the beam were got out of place resulting in punching shear failure at the top of the column and rapid deterioration of load capacity. In Specimens W-4,5 and 6, using the shear connector of steel pieces anchored under the beam, the steel pieces were sheared out between the beam and the top of the wall at the large deflection.

Load capacities of the frames having infilled wall are 3.5-5.0 times that of the frame only, and 0.55-0.72 times that of the monolithic wall. Maximum loads of all infilled walls are 0.78-1.10 times calculated bending strength in Table 2, on the assumption that the frame having infilled wall is a cantilever beam of I-section.

The slip displacement between wall and beam was a little before the maximum load.

As mentioned above, this method of the infilled shear wall inside the existing frame can be remarkably effective for strengthening of the existing reinforced concrete buildings that are short of strength and stiffness.

## EXPERIMENT 2

This method is strengthening the surroundings of the existing reinforced concrete column by mortar and welded wire fabrics. This experiment is designed to give a quantitative understanding of how mortar reinforced with welded wire fabrics increases the shear strength and ductility of the existing column.

### a) Test Specimens

Test specimens consist of 4 types as shown in Fig.8 and 9. The scale of the model is about one-half, 180cm height, 45cm x 45cm of column cross section and shear span ratio, M/QD, is 2.0.

Specimen C-1 is designed to be an existing column with stubs for loading at both ends, and this is the standard type model of other strengthened column specimens. Specimens C-2 and 3 are columns strengthened by mortar of 4.5cm thickness and welded wire fabrics. These specimens are different in location of joining welded wire fabrics to the loading direction as shown in Fig.8. Specimen C-4 is a strengthened column in the same way by mortar of 9.0cm thickness and welded wire fabrics. Each strengthened column has gaps of 3.0cm width at both ends of the column.

The mortar was cast into the mould with pressure as the same way of the infilled wall in Experiment 1. Welded wire fabrics consist of steel bars of 6mm diameter and 50mm x 50mm-mesh.

The arrangement of reinforcement of the column and the strengthened section are shown in Fig.9, tensile reinforcement ratio, $p_t$, is 0.7% and shear reinforcement ratio, $p_w$, is 0.11% of column cross section. The quantity of welded wire fabric for strengthening is determined that the capacity of shear resistance is equal with that of bending resistance of column.

The mechanical properties of materials are shown in Table 3.

### b) Loading and Measuring Method

Testing facility is shown in Fig.10. Each specimen was set horizontally and steel stabs was fixed to the concrete stabs of specimen at both ends to support the specimen on the testing frame.

The axial load, 81ton, ( $\sigma_o$=40kg/cm$^2$), was applied through a load cell by an oil jack with capacity of 100ton. The alternately reversal load was applied antisymetrically at the center of concrete stub on both ends of specimen by four oil jacks.

The relative displacement between the top and bottom of column was measured.

c) Experimental Results

The calculated values (1) and the experimental results (2) of initial stiffness and various strengths are shown in Table 4.

Load-deflection curves of each specimen and the envelope curves of all specimens are shown in Fig.11 and 12.

Specimen C-1, un-strengthened column, deteriorated its capacity at relatively early stage before the tensile reinforcing bar yielded, when bond split failure of concrete along the tensile reinforcing bar was occurred at lateral load of 64ton and at the rotation angle of member of 1/200 rad. Thereafter, as deflection increased, the load capacity was lost rapidly to 53% of the maximum load at the rotation angle of 1/50 rad.

All strengthened specimens showed deterioration of load capacity when tensile reinforcement yielded before the maximum load. Good agreement of the maximum load was obtained between the experimental results and the calculated values by Dr. UMEMURA's e-Function Method.

In Specimens C-2,3 and 4 that were strengthened all sides of the column, the load capacity were not lost before the rotation angle of 1/50 rad. In Specimens C-2 and 3, the increased section of mortar and welded wire fabrics ruptured at the ends of column not to be able to hold the axial load, though Specimen C-4, the mortar thickness of 9.0cm, could hold its ultimate load capacity without the rupture at the rotation angle of 1/25 rad.

As shown in Fig.12, each strengthened column was recognized to increase deflection capacity by this kind of strengthening method.

As mentioned above, these strengthening method can be recognized to be very effective for the column that is short of shear strength and ductility.

## CONCLUSIONS

These experimental results indicate the general adequancy for two kinds of strengthening techniques,

1) to increase the lateral resistance of buildings by the infilled wall inside the existing frame, and
2) to increase shear strength and ductility by strengthening the surroundings of the existing column with mortar and welded wire fabrics in order to prevent the column from brittle shear failure.

With reference to these investigations, strengthening methods were refined to be more effective for application to the existing reinforced concrete buildings that can be evaluated to be short of earthquake resistance, and have been carried out for several buildings in Japan.

## ACKNOWLEDGEMENTS

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The authors wish to acknowledge gratefully for these contributions.

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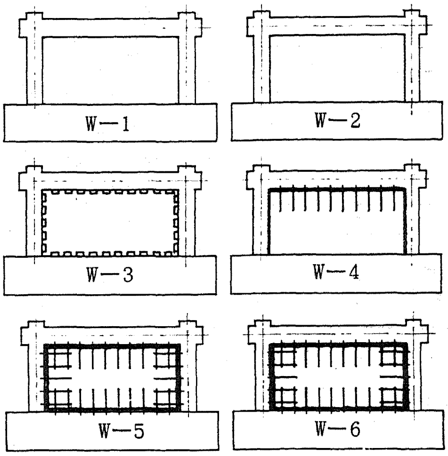

Fig.1 Specimen of frame and infilled wall

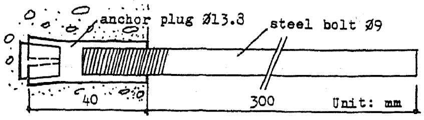

Fig.2 Steel piece anchor

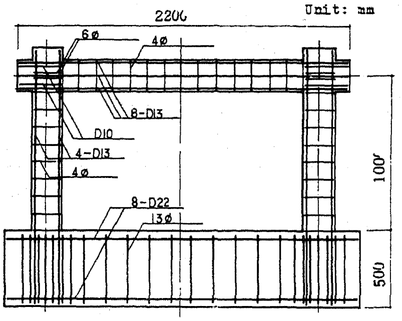

Fig.3 Reinforcing arrangement of standard frame type

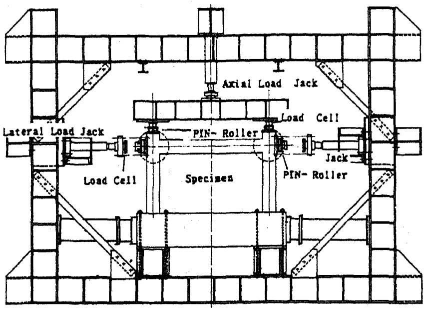

Fig.5 Testing facility

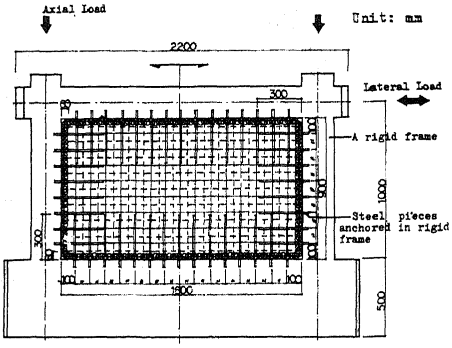

Fig.4 Reinforcing arrangement of infilled wall

Table 1
Mechanical properties of materials

| | concrete kg./cm² | | | steel TON./cm² | | | |
|---|---|---|---|---|---|---|---|
| | column and beam | wall (1) | wall (2) | | yield strength | ultimate strength | modulus of elasticity |
| **W-1** | 183 | — | — | tensile reinforcement D13 | 3.55 | 5.38 | 1970 |
| **W-2** | 183 | 183 | — | | | | |
| **W-3** | 183 | 306 | 394 | shear reinforcement φ4 | 2.30 | 3.14 | 1950 |
| **W-4** | 183 | 272 | — | welded wire fabric φ6 | — | 7.83 | — |
| **W-5** | 185 | 306 | 394 | | | | |
| **W-6** | 185 | 306 | 394 | steel piece anchor φ9 | 2.79 | 3.70 | 1680 |

Table 2 Calculated values(1) and experimental results(2)

| | intial stiffness (1) TON/cm² | | bending cracking load (2) TON | | shear cracking load (3) TON | | ultimate shear strength (4) TON | bending strength (5) TON | ultimate strength TON |
|---|---|---|---|---|---|---|---|---|---|
| | (1) | (2) | (1) | (2) | (1) | (2) | (1) | (1) | (2) |
| **W-1** | 442 | 58 | — | — | — | — | — | 9.5 | 11.2 |
| **W-2** | 960 | 720 | 15.2<br>(29.5) | 14.5 | 31.9<br>(26.1) | 31.5 | 68.5<br>(66.5) | 102.5 | 77.5 |
| **W-3** | 1070 | 1200 | 15.2<br>(29.5) | 13.5 | 43.1<br>(37.3) | 40.0 | 90.8<br>(88.8) | 53.1 | 41.5 |
| **W-4** | 1860 | 1000 | 15.2<br>(29.5) | 22.5 | 37.3<br>(31.4) | 40.0 | 77.9<br>(55.5) | 53.1 | 55.0 |
| **W-5** | 1070 | 720 | 16.2<br>(30.6) | 17.5 | 43.1<br>(37.3) | 26.0 | 90.8<br>(88.8) | 53.1 | 48.0 |
| **W-6** | 1070 | 1200 | 16.2<br>(30.6) | 26.0 | 43.1<br>(37.3) | 35.5 | 90.8<br>(88.8) | 53.1 | 55.0 |

Note:

(1) $K_i = 1/(\frac{\ell^3}{3EI_e}+\frac{K}{GA})$

(2) $P_{bc}=({}_c\sigma_t+\sigma_n)\cdot Z_0/\ell$

(3) $Q_{sc} = K_s\cdot K_B\sqrt{(1.8F_c)^2+1.8\ F_c\cdot\sigma_0}\times t\cdot\ell$

(4) $Q_u=\left\{\frac{0.0679p_t^{0.23}(F_c+180)}{\sqrt{M/QD+0.12}}+2.7\sqrt{p_w\cdot{}_w\sigma_y}+0.1\sigma_0\right\}\times b_e\cdot j$

(5),(6) e-Function Method

$K=1.2$ $G=\frac{3}{7}E$ A=thickness x span length $K_s=1-0.52p_w$ $K_B=(1.0+1.05t/B)/(1.0+1.77t/B)$

$F_c$=concrete strength

$\sigma_0$: axial stress, loading area is of frame and infilled wall
( )loading area is frame only

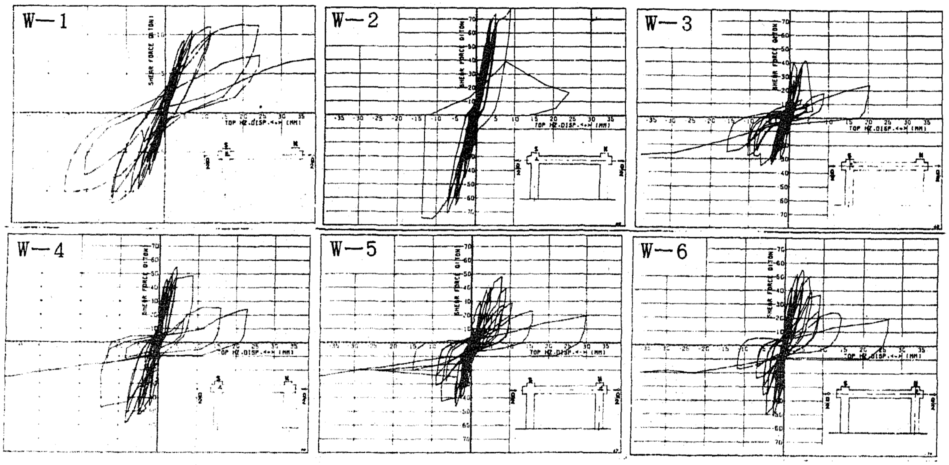

Fig.6 Relationships between load and deflection

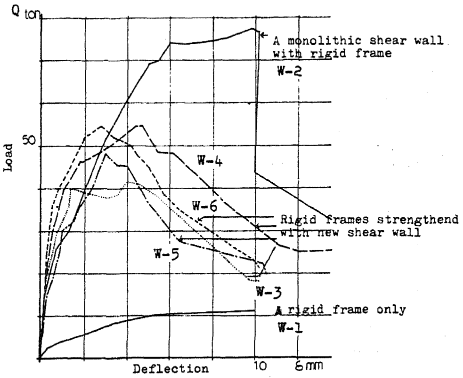

Fig.7 Envelope curves of load-deflection

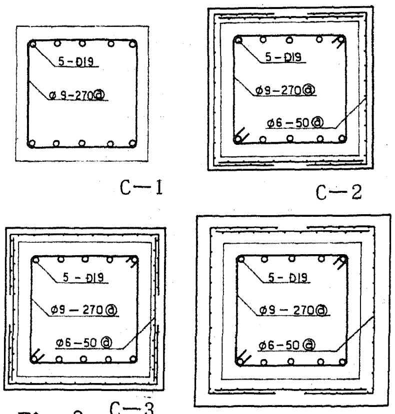

Fig.8 Test specimen (Cross section of column)

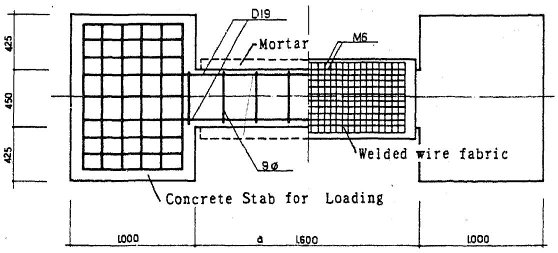

Fig.9 Reinforcing arrangement

Unit: mm

Table 3 Mechanical properties of materials

| | concrete and mortar kg/cm² | | |
|---|---|---|---|
| | concrete | mortar (1) | mortar (2) |
| **C-1** | 206 | — | — |
| **C-2** | 172 | 250 | 294 |
| **C-3** | 172 | 250 | 294 |
| **C-4** | 213 | 241 | 225 |
| **C-5** | 213 | 241 | 225 |

| steel TON/cm² | yield strength | ultimate strength | modulus of elasticity |
|---|---|---|---|
| tensile reinforcement D19 | 3.57 | 5.38 | 2214 |
| shear reinforcement ø9 | 3.01 | 4.17 | 3200 |
| welded wire fablic ø6 | 3.25 | 6.01 | 2028 |

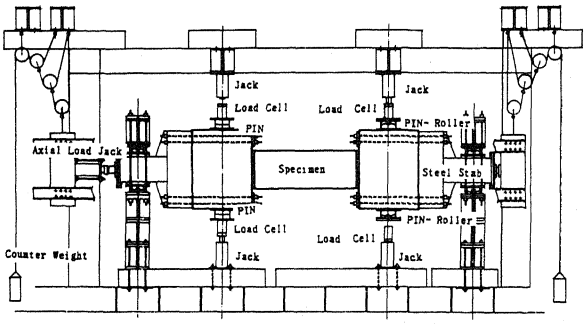

Fig.10 Testing facility

Table 4 Calculated values(1) and experimental results(2)

| | intial stiffness (1) TON/cm² | | bending cracking load (2) TON | | shear cracking load (3) TON | | yield strength of E-fanction (4) TON | ultimate strength of E-fanction (5) TON | ultimate shear strength TON (6) | ultimate strength TON |
|---|---|---|---|---|---|---|---|---|---|---|
| | (1) | (2) | (1) | (2) | (1) | (2) | | | (1) | (2) |
| **C-1** | 164.5 | 69.9 | 12.25 | 15.0 | 25.8 | 25.8 | 33.82 | 36.16 | 30.9 | 30.5 |
| **C-2** | 278.0 | 109.7 | 17.90 | 15.0 | 33.1 | 30.8 | 33.35 | 35.36 | 46.7 | 41.6 |
| **C-3** | 278.0 | 100.0 | 11.90 | 15.0 | 33.1 | 31.5 | 33.35 | 35.36 | 46.7 | 42.6 |
| **C-4** | 497.4 | 93.8 | 12.35 | 15.0 | 42.0 | 32.5 | 34.05 | 36.11 | 57.5 | 39.1 |
| **C-5** | 344.8 | 79.5 | 12.35 | 15.0 | 32.6 | — | 34.05 | 36.11 | 43.8 | 37.1 |

Note:

(1) $K1=1/\left\{2\int_{0}^{a_1}\left(\frac{x^2}{cEI_e}+\frac{K}{GA}\right)dx + 2\int_{a_1}^{a_2}\left(\frac{x^2}{cEI_e}+\frac{K}{GA}\right)dx\right\}$

(2) $Q_{bc}=\left\{1.8\,F_c\cdot Z_e+\frac{ND}{6}\right\}/\left(\frac{l}{2}\right)$

(3) $Q_{sc}=0.055+0.15N/BD\cdot {}_c\sigma_b$

(4).(5) e-Function Method

(6) $Q_{su}$

$=\left\{\frac{0.053p_t^{0.23}(180+F_c)}{M/QD+0.12}+2.7\,p_w\cdot {}_s\sigma_{wy}+0.1\sigma_0\right\}$

$\times\, b\cdot j$

Fc: concrete strength

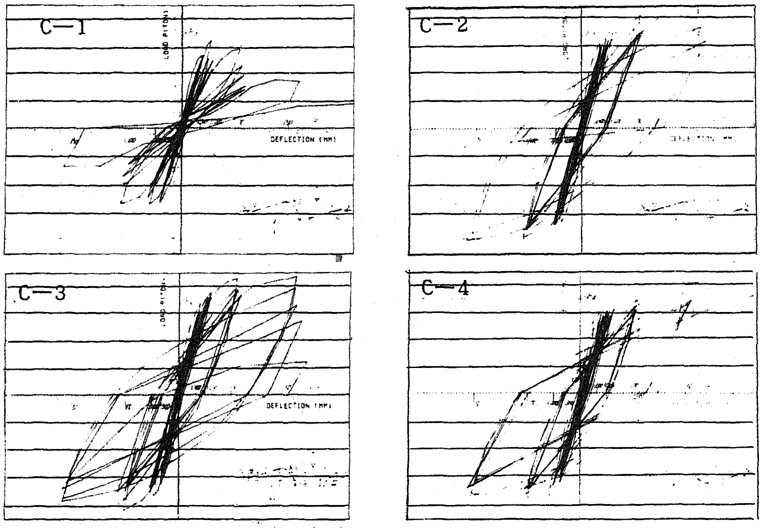

Fig.11 Relationships between load and deflection

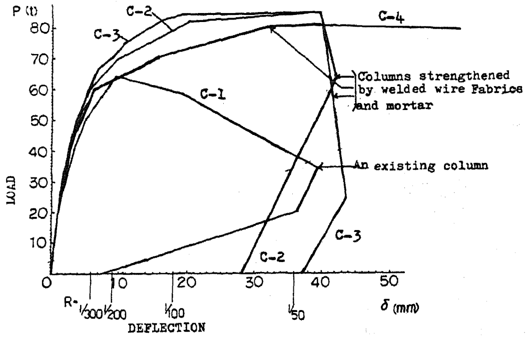

Fig.12 Envelope curves of load-deflection