STRENGTHENING EXISTING RC COLUMNS FOR EARTHQUAKE RESISTANCE

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SUMMARY

Four reinforced concrete columns were constructed with 254 mm square cross-sections, lengths of 3.5 m and with main reinforcement ratios of 0.024. Concrete strength averaged 44.2 mPa, while steel yield stress was 391.7 mPa; ties were spaced on 254 mm centers. The first column was tested, repaired with portland cement mortar and with added hoops, and retested. The other columns first were strengthened externally with either steel packaging bands, 6 mm wire spiral, or 8 mm thick U-clamps. The static, reversed cycle tests showed that the repair and all of the simple strengthening techniques greatly increased the ductility of the RC columns.

INTRODUCTION

Earthquakes have demonstrated that reinforced concrete columns often fail in brittle, shear-type modes when they are not reinforced with adequate transverse steel. These column failures typically occur near the connection of the column to a stiffer girder or foundation system. Current research (Ref. 3) and standards (Ref. 1) have shown and specify the need for closely spaced hoops in rectangular columns in order to confine the core concrete and to provide shear resistance when the concrete cover spalls. Such seismic detailing has improved the earthquake resistance of new structures.

But many existing buildings were constructed before the benefits of close tie spacing and "ductile" concrete concepts were known. Many of these structures may require column strengthening to assure ductile seismic behavior or may require repair and strengthening after a damaging earthquake. With the advent of earthquake predictions, engineers will be called upon to initiate rapid retrofit procedures. They will not be given months to design and construct strengthening systems for a few important structures; after a prediction the public will demand that most structures be seismically improved in a few days or weeks. Therefore, quick, economic and easy-to-construct techniques must be developed for such earthquake strengthening needs.

The purpose of this experimental research was to study three simple and potentially inexpensive techniques for strengthening existing reinforced concrete columns. The scope was limited to testing four 254 mm square columns; a qualitative comparison of the behavior of the columns was the primary objective. Past work by Higashi and Kokusho (Ref. 2) demonstrated some other strengthening techniques.

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SPECIMEN DESIGN AND CONSTRUCTION

The four specimens were designed to represent flexible columns connected to stiff girders. The original columns were not reinforced to provide transverse reinforcement for ductility according to the following equations (from Ref. 1):

$$A_{sh} = \frac{\ell_h \rho_s S_h}{2} \tag{1}$$

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_y}\text{, not less than } 0.12\,\frac{f'_c}{f_y} \tag{2}$$

A_{sh} = area of transverse hoop, one leg

ℓ_h = unsupported length of hoop

ρ_s = ratio of volume of transverse reinforcement

S_h = center-to-center spacing of hoops

A_g = gross area of column

A_{ch} = area of rectangular core of column

f'_c = compressive strength of concrete

f_y = yield stress of hoop steel

The important behavior of the beam-column connection purposely was not considered in this experiment.

Figure 1 illustrates the unstrengthened column specimen; the large center block represents the connection to the stiff girder. The reinforcement ratio (ρ) for each column was 0.024. The yield stress (f_y) for the 22.2 mm diameter bars was 391.7 mPa while the 6 mm bar ties had an f_y of 537.2 mPa. Data for the concrete and strengthening techniques is presented in Table 1.

Specimen 1 was not strengthened prior to testing (Figure 2). After testing, Specimen 1, now termed Specimen 1R, was repaired by first hammering away all fractures concrete (Figure 3). Buckled reinforcement was cut and welded to straight pieces of 7.9 mm thick, 51 mm x 51 mm steel angle. Hoops of 9.5 mm reinforcing bars were bent around the repaired main reinforcement and were spaced 38 mm on centers as shown in Figure 4. The column on only one side of the block was damaged and, therefore, repaired; the remainder of Specimen 1R was strengthened using U-clamps identical to Specimen 4 described later.

Specimen 2 was strengthened by tensioning 50.8 mm wide packaging bands around the column. The band hoops were spaced 102 mm on center and were secured with pressed clips (Figure 5). Tension tests showed that the clipped bands started to slip at a stress of 614 mPa, 5 percent less than the f_y given in Table 1. The 6 mm gap beneath the band hoops was packed with a non-shrink mortar.

Specimen 3 was strengthened by hammering a 6 mm plain steel rod around the column to form a rectangular spiral with a 28 mm pitch

(Figure 6). Splices in the spiral were made by lap welding the bar. The 1 mm spaces beneath the spiral were filled with mortar.

Figure 7 shows the U shaped clamps used to strengthen Specimen 4. These U-clamps were fabricated by welding 7.9 mm x 50.8 mm bar to 79.4 mm thick steel angle (76 mm x 127 mm x 63.5 mm wide). A325-19 mm bolts held two U-clamps together to form a hoop; hoops were spaced 108 mm on centers.

TESTING AND RESULTS

All specimens were instrumented with electrical resistance strain gages bonded to main and transverse reinforcement; deflections (Δ) were measured with dial gages at the face of the center block. As shown in Figure 2, each specimen was mounted horizontally, and a constant axial load of 360 kN was applied. Lateral loads were applied statically to the center block to deflect the column cyclically, down, then up. Three deflection cycles were conducted to each deflection level, $\frac{1}{2}\Delta y$, Δy, $2\Delta y$, and $4\Delta y$, where Δy was the deflection level at which the main reinforcement yielded in tension.

Figures 8 through 11 show the shear force-deflection response for Specimens 1 through 4, respectively. The hysteresis curve for Specimen 1R was identical to that for Specimen 4. Specimen 1 collapsed (lost axial load capacity) at a deflection of 38 mm, which represented a ductility ratio, $\Delta/\Delta y$, of 1.9. Retesting showed that the repaired section of Specimen 1R cracked slightly only at the maximum shear load; the main reinforcement yielded on the other side of the center block which had been previously cracked but not severely damaged. This area of Specimen 1R was strengthened with U-clamps. Plastic hinge formation and cracking of Specimen 1R were identical to Specimen 4, also strengthened with U-clamps.

The hysteresis curves for Specimen 2, 3, and 4 were nearly identical in the inelastic region; the columns demonstrated ductile flexural response without shear deterioration. At the maximum deflection level, a ductility ratio of 4, each column showed slight diagonal X cracking near the center block and some concrete crushing failure beneath the strengthening hoop nearest the center block. For each column this hoop was about 25 mm from the face of the connection. The strains in the band, spiral and U-clamp nearest the connection were less than 14 mPa. Apparently each external strengthening technique confined the concrete so that the shear force was carried primarily by the concrete.

CONCLUSION

The steel packaging bands, rectangular spiral, and U-clamps strengthened the RC columns by increasing their ductility. The unstrengthened column (Specimen 1) collapsed at a deflection ductility ratio less than 2 while the repaired and strengthened columns resisted three deflection cycles to ductility ratios of 4 with little deterioration. Repair using portland cement mortar and 9.5 mm hoops strengthened column 1R so that plastic hinging occurred in another location.

Table 1 illustrates that the bands, rectangular spiral and repair all utilized less confining steel than required by equation (1) and as specified in Reference 1; only the U-clamps provided more A_{sh} than required. Nevertheless, all techniques greatly improved the column earthquake resistance similarly. Equations (1) and (2) were developed to provide a column whose confined core would continue to resist the applied loads after the cover had spalled. By providing external hoops, spalling was prevented in Specimens 1R through 4 so that the total concrete area, A_g, could carry axial and shear loads. This larger effective concrete area permitted the "less-than-required" areas of bands or spirals to be effective. It is noted that retrofit procedures often do not correspond to requirements generally established for new construction.

Construction of the specimens showed that the packaging bands and U-clamps were the easiest to emplace and that the banding appeared quickest and most economic. Placing and bending the 9.5 mm hoops for the repair technique was the most difficult construction procedure. The author concludes that the banding and U-clamps show significant potential for providing a low cost alternative for quickly improving the earthquake resistance of existing reinforced concrete columns.

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Table 1. Specimen Data

Specimen	1	1R	2	3	4
Retrofit Techniques	None	Mortar/hoops U-clamps	Packaging Bands	6 mm Spiral	U-clamps
f'_c (mPa)	43.8	52.0 43.8	44.6	43.8	44.6
f_y, hoops (1) Repair/Strength.		435.2/ (2)	646.2	461.4	293.8(3) 235.2(4)
A_{sh} (mm^2)		71.3/ (2)	58.1	31.7	403.2(3) 504.0(4)
S_h (mm)		38.1/ (2)	102	28	108
A_{sh} required (Eqn. 2, mm^2)		71.2/ (2)	116.1	45.2	251.6
$\frac{A_{sh}}{A_{sh} \text{ required}}$		1.0/ (2)	0.51	0.78	1.61

(1) for 9.5 mm hoops used in repair

(2) same as for Specimen 4, U-clamps

(3) for flat bar

(4) for angle shape

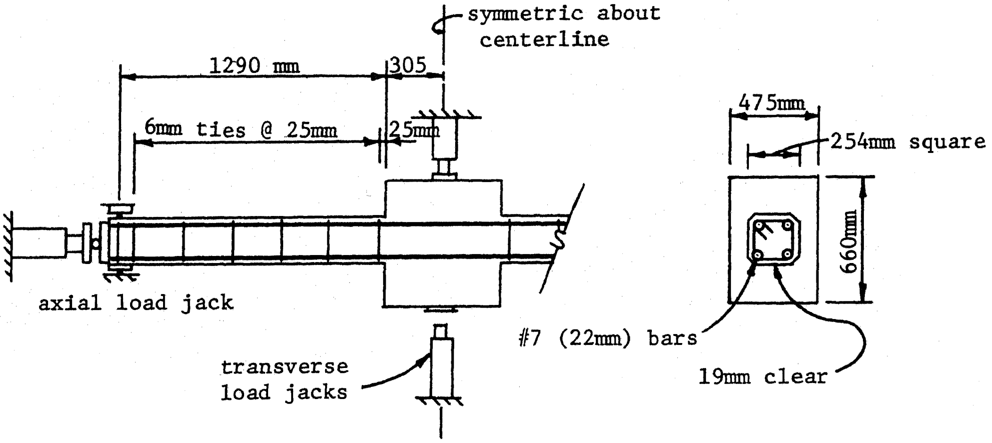

Figure 1. Details of reinforced concrete columns (unstrengthened).

Figure 5. Specimen 2, packaging bands.

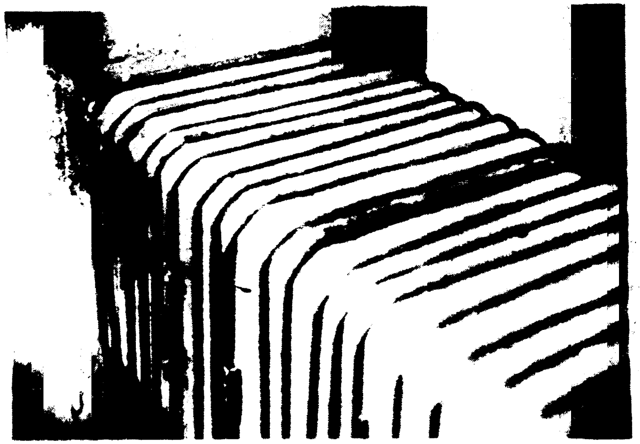

Figure 6. Specimen 3, 6 mm rectangular spiral.

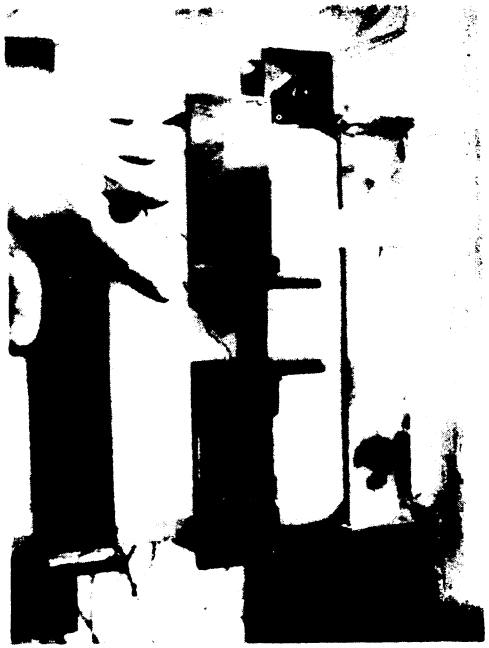

Figure 7. Specimen 4, U-clamps; cracking at maximum deflection.

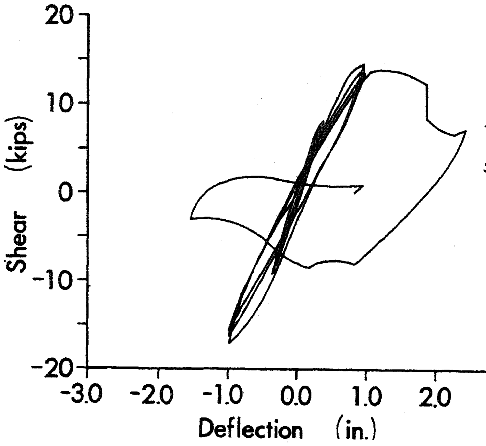

Figure 8. Specimen 1, unstrengthened.

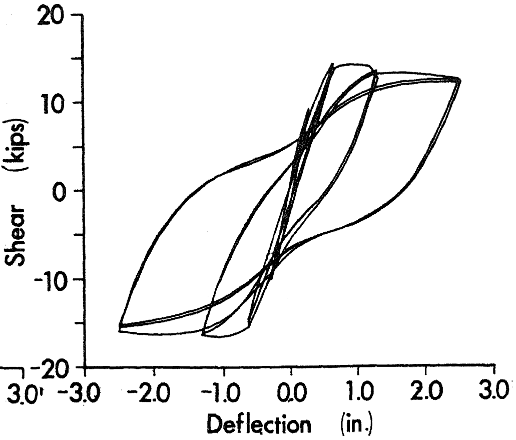

Figure 9. Specimen 2, bands.

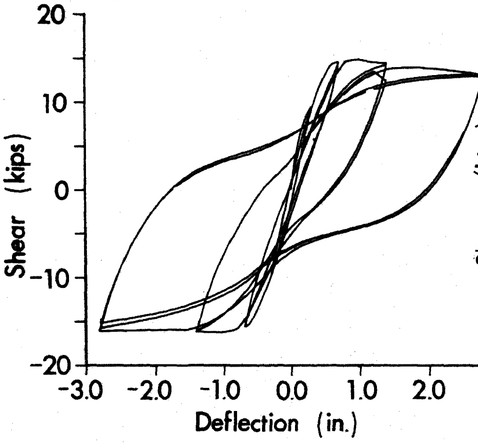

Figure 10. Specimen 3, 6mm spiral.

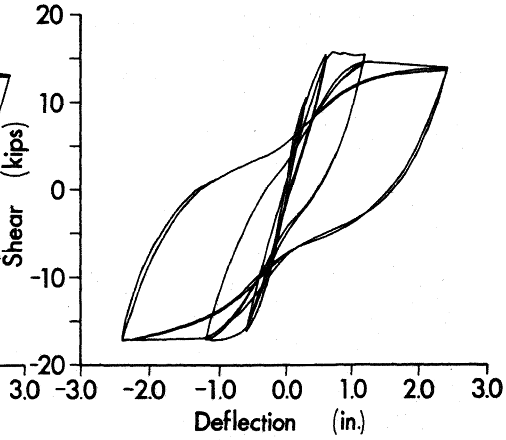

Figure 11. Specimen 4, U-clamps.

Shear: 1 kip = 4.5 kN Deflection: 1 in. = 25.4 mm