THE INELASTIC RESPONSE OF REPAIRED REINFORCED CONCRETE BEAM-COLUMN CONNECTIONS

by

Polat Gulkan[I]

SYNOPSIS

Results obtained from two 3/4-size beam-column test specimens, subjected to alternated loads applied at the beam ends before and after damaged columns were repaired, are described. The repair technique in which the original shell was chipped off and replaced, with additional reinforcement, by a thicker shell improved the strength of the subassemblage considerably. It was observed that the joint core where the repair process remains ineffective may then become the critical location. There appears to be no reduction of the ductility and energy absorption capacity of the specimens after repair.

INTRODUCTION

Research conducted during the past decade on the cyclic behavior of concrete structural elements has yielded information which may be used in mathematical models for dynamic analysis (e.g. 1,2,3). While such information is required for the prediction of behavior under load reversals, there exists inadequate information on the cyclic response of structural elements after they have been repaired. Recent experience has indicated that after a moderate earthquake hits a reinforced concrete structure it is difficult to decide if repair of damaged elements would be feasible (4,5). The injection of epoxy resin into cracks for the repair of flexural elements seems to be effective (5); however it is not known how good this would be for axial members in which the cracks would close. Another method of repairing such elements consists of casting an additional reinforced shell around the original section (4). In this paper, a description is first given for the specimens selected and the method of testing them. Next, description of the repair process and a comparison of the behavior and strength of original and repaired elements follows. Recommendations for improving the repair process conclude the report.

SELECTION, DETAILS OF SPECIMENS AND EXPERIMENTAL PROCEDURE

In the experimental program conducted at METU a total of five specimens were tested before and after repair. The beam-column subassemblage representing an interior joint of a "tall" building frame is shown in Fig. 1. In contrast to the majority of tests conducted on similar specimens, load reversals were applied through forces acting at the ends of the beams in opposite directions while columns extending above and below the joint to inflection points were pinned at these locations.

The details of test specimens 3N and 5N are shown in Fig. 2a. The only difference of 5N was the lack of the beam stub representing an out-of-plane member spanning into the core. The main longitudinal reinforcement for the beams consisted of six plain 14 mm bars at the top and the bottom. Stirrups were 6 mm plain bars placed at 10 cm on center. As can be seen in Fig. 2a, the main reinforcement was continuous through the joint area along the entire

I Department of Civil Engineering, Middle East Technical University, Ankara, Turkey.

length of the two beams. Likewise, the main longitudinal reinforcement for the columns consisted of four continuous 14 mm plain bars, one at each corner, and the ties were 6mm bars placed 10 cm apart within Zone 1 and 5 cm within Zone 2 and the joint core. The average yield strength for the 14 mm diameter bars was 3,000 kg/cm^2 with an ultimate strength of 4,500 kg/cm^2. For the 6 mm diameter bars, these values were 3,500 and 5,000 kg/cm^2, respectively. The average concrete strength for specimen 3N was 165 kg/cm^2, and its repaired counterpart 3M had a concrete strength of 325 kg/cm^2. The corresponding values for 5N and 5M were 280 and 255 kg/cm^2, respectively.

All specimens were tested in a vertical position as shown in Fig. 1. After a specimen was placed in the test frame and the columns "pinned" at both ends, the axial load was applied by means of a jack placed at the top of the column and attached to a high strength bar running through a centrally placed duct. Then, by means of hand operated jacks, opposite forces were applied at the ends of the beams. Inasmuch as the jacks could not pull, their positions were alternated each half cycle. The displacement histories which were followed at the left ends for specimens 3N, 3M, 5N and 5M are shown in Fig. 3.

REPAIR OF SPECIMENS

After a particular displacement history was forced through the beam ends, the specimens were removed for repair. This consisted of chiseling off the concrete shell of the columns until transverse reinforcement became visible and then placing the specimen in a horizontal position where additional longitudinal and transverse reinforcement was provided along the column length. This additional reinforcement which consisted of four 14 mm bars in the corners and 6 mm ties at 10 cm was carefully centered around the original section and welded by short studs at several points to the existing bars. Next, concrete was poured to form a box-like shell of new uncracked concrete around the original core. Additional transverse reinforcement could of course not be made continuous through the joint area (see Fig. 2b). After the repair process was completed, the specimen was tested in similar fashion as the original element had been tested.

EXPERIMENTAL RESULTS

A summary of experimental results is given in Table 1. The load-deflection diagram for the left beam and the moment-rotation curve for the upper column for specimen 3N are shown in Figs. 4a and 4b, respectively. Drawn to the same scale, similar curves are given for the same specimen (now designated as 3M) after repair in Figs. 4c and 4d. The failure moment computed for the column with nominal material properties is 240 t-cm for Section A-A and 520 t-cm for Section C-C (Fig. 2) assuming that old and new concrete form a homogenous cross section. Experimental results indicate that these values could be developed and that there was no serious loss of strength after a few cycles past yield. In general, cracks in the column of the repaired specimen were initiated from previous cracks in the original member. Because of the great difference between the ultimate moment capacities of the columns and the beams (Sections A-A and C-C vs Section B-B in Fig. 2), cracking was confined almost entirely to within the column region immediately above and below the joint core. The presence of the 20 cm long beam stubs on either side of the core prevented the formation of diagonal cracks within the joint. Special precautions taken in the core region prevented the buckling of column longitudinal reinforcement outward. This is the most important reason for the specimen to have maintained its strength.

In Figs. 5a to 5d, the same set of results are indicated for specimens 5N and 5M which did not have the lateral beam stub. In both of these specimens there was extensive diagonal cracking within the joint area, especially in the case of 5M. This can be explained in terms of the higher shear forces that must be transmitted through the joint core due to the increased strength of the repaired columns. Since additional ties could not be provided for the joint, there was no increase in its strength, and this forced the failure into the joint. For this particular specimen, the weak link was the joint as illustrated by the 2-3 mm wide cracks that appeared in the diagonal direction in spite of the axial load. Also the full capacity of the columns could not be developed (compare Figs. 5c and 5d with Figs. 4c and 4d).

The ductility of specimens 4N and 4M showed no adverse reduction after repair. In general reduction in stiffness was more rapid in the repaired specimens because of the lesser amount of uncracked concrete.

CONCLUSIONS

The repair technique described in this paper appears to have merit in improving the strength of reinforced concrete columns damaged previously due to alternated loading. The strength of the new composite column may be determined through ordinary flexural mechanics computations where the effect of composite action is ignored. Additional reinforcement should be continuous and reinforcement through the joint area should be welded to the existing bars to prevent outward buckling.

ACKNOWLEDGMENTS

The research project reported in this paper was sponsored by the Turkish Scientific and Technical Research Council (TBTAK) under grant MAG-372. Chief technician B. Avci supervised the laboratory work, and graduate student D. Aksiyote reduced the voluminous amount of data obtained.

REFERENCES

1. Hanson, N.W., and W.H. Conner, "Seismic Resistance of Reinforced Concrete Beam-Column Joints," Journal of the Structural Division, ASCE, October, 1967, pp. 533-560.

2. Karlsson B.I., H. Aoyama, and M.A. Sozen, "Spirally Reinforced Concrete Columns Subjected to Loading Reversals," Proceedings of the Fifth World Conference on Earthquake Engineering, Rome, 1973.

3. Takeda, T., M.A. Sozen, and N.N. Nielsen, "Reinforced Concrete Response to Simulated Earthquakes," Journal of the Structural Division, ASCE, December, 1970, pp. 2557-2573.

4. Anadol, K., U. Arioglu, and E. Arioglu, "Repair and Strengthening of the Sakarya Governmental Building Damaged Heavily during the 1967 Akyazi Earthquake," Proceedings of the Earthquake Engineering Symposium, Middle East Technical University, Ankara, 1972 (in Turkish).

5. Popov, E.P., and V.V. Bertero, "Repaired R/C Members under Cyclic Loading," International Journal on Earthquake Engineering and Structural Dynamics, Vol. 4, pp. 129-144 (1975).

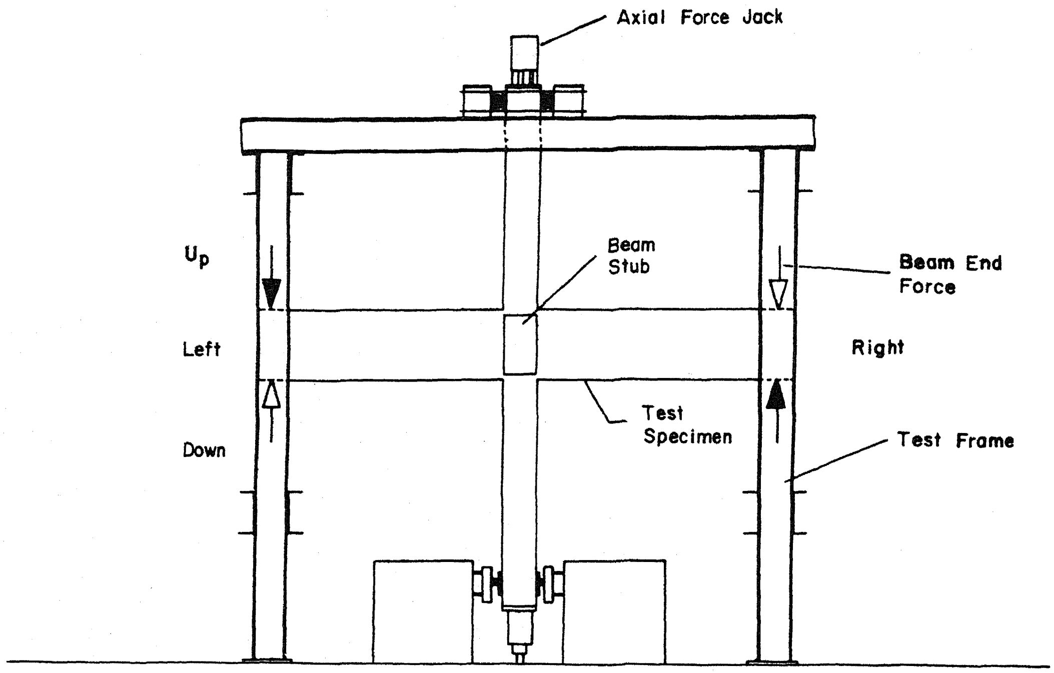

FIG. 1 - SCHEMATIC VIEW OF TEST SETUP

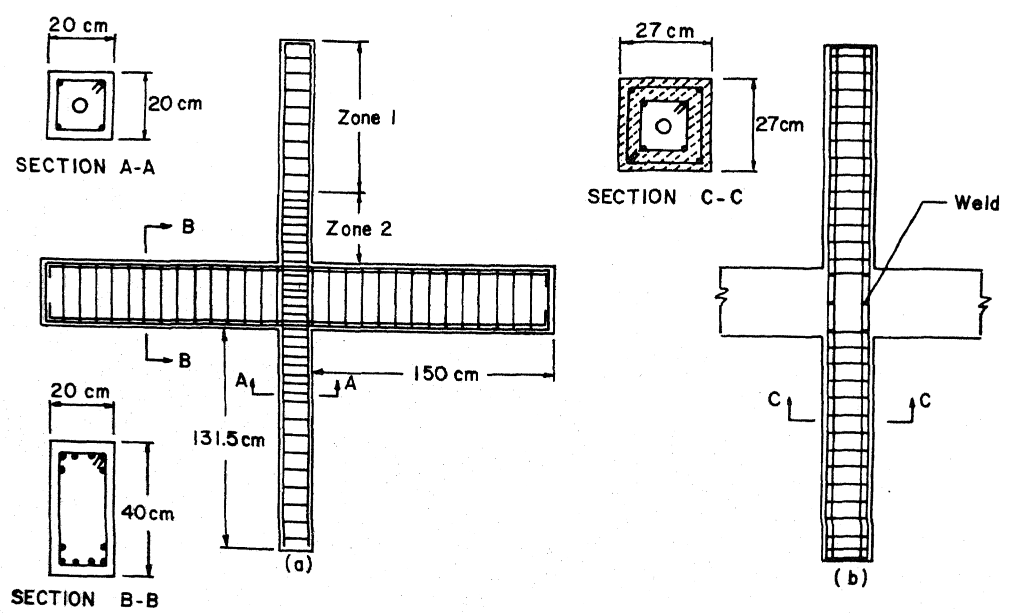

FIG. 2 - TEST SPECIMEN DETAILS

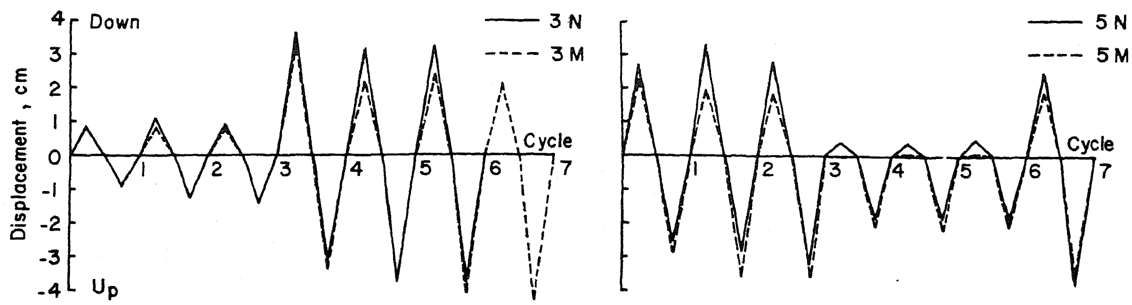

FIG. 3 - DISPLACEMENT HISTORIES

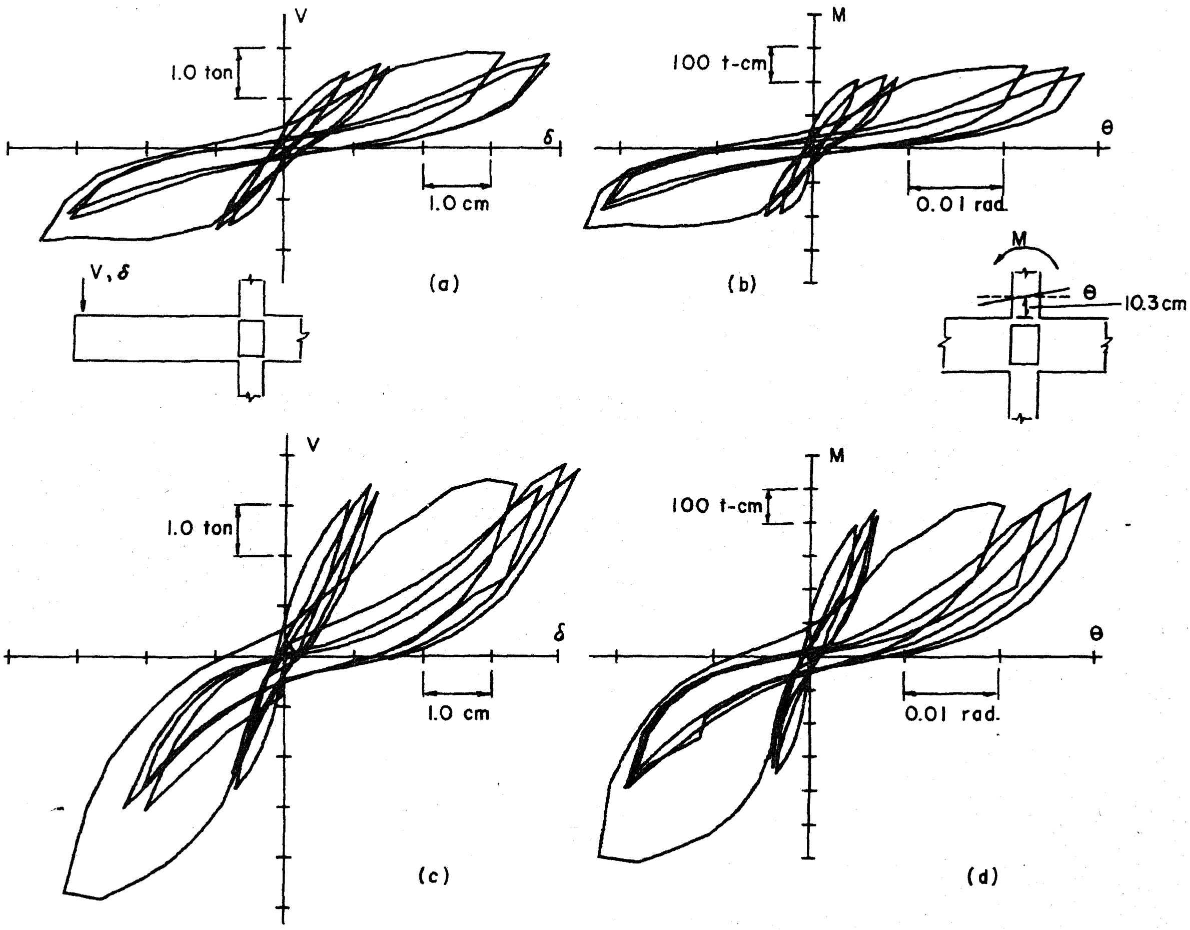

FIG. 4 - EXPERIMENTAL RESULTS FOR SPECIMENS 3N AND 3M

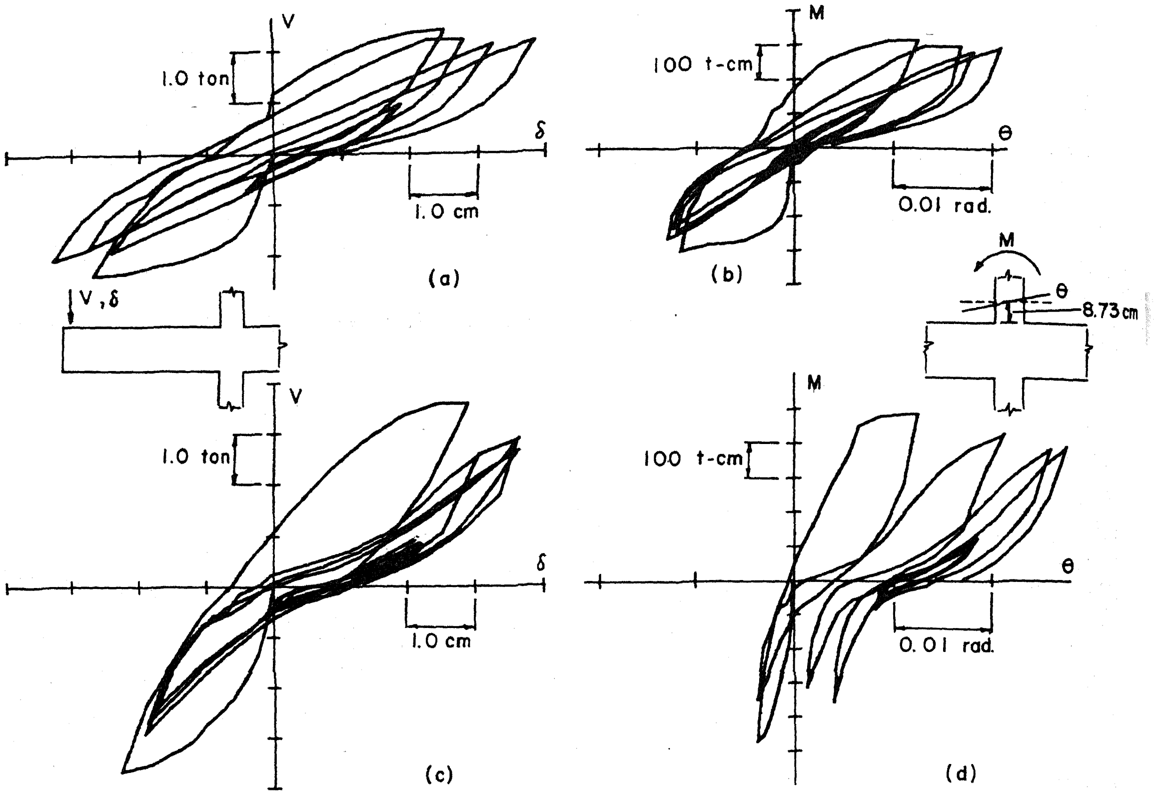

FIG. 5 - EXPERIMENTAL RESULTS FOR SPECIMENS 5N AND 5M

TABLE I
SUMMARY OF EXPERIMENTAL MAXIMA

Specimen	Condition	V^{*}_{max}, ton	δ^{*}_{max}, cm	M^{**}_{max} ton-cm	θ^{**}_{max}, rad
3 N	As Cast	+ 1.80	+ 3.59	+ 238.3	$+2.38 \times 10^{-2}$
		- 1.87	- 3.77	- 233.1	-2.83×10^{-2}
3 M	Repaired	+ 4.72	+ 3.25	+ 607.9	$+2.20 \times 10^{-2}$
		- 3.83	- 4.27	- 493.3	-2.94×10^{-2}
5 N	As Cast	+ 2.33	+ 3.32	+ 304.6	$+1.30 \times 10^{-2}$
		- 2.40	- 3.85	- 301.4	-2.08×10^{-2}
5 M	Repaired	+ 3.68	+ 2.31	+ 474.0	$+3.93 \times 10^{-3}$
		- 3.55	- 3.62	- 486.2	-2.77×10^{-2}

* Relative to the "left" beam

** Relative to the upper column

DISCUSSION

M.K. Aggarwal (India)

Let the discussor know the suggestions from author about repair of a three storeyed concrete frame structure which has shown cracks in column because of bad quality of concrete (porous) in columns during concreting. Though the columns are safe for non seismic loads but may fail for earthquake forces. The method expressed by author in the paper is excellent till the concrete in the inner shell is strong enough but what will happen if the concrete in inner shell has also undergone crushing.

B.R. Seth (India)

Such an arrangement may not be effective for a column in the actual building as that is subjected to bending moment as well. To improve, it is suggested that cross grooves should be cut on the surface, which shall transfer shear stresses to make the exposed reinforcement effective to resist moment.

Author's Closure

Not received.