# BEHAVIOR OF SHORT R/C COLUMNS SUBJECTED TO CYCLIC BILATERAL DEFORMATIONS

K. A. Woodward[I]

SUMMARY

The types of behavior exhibited by short reinforced concrete columns was experimentally investigated. The columns were subjected to cyclic bidirectional translations which introduced high shear stresses and double curvature onto the column. The main variables were the amounts of longitudinal and transverse reinforcement in the column. The purpose was to vary the ratio of shear to flexural capacity of the section. A range of behaviors were observed including a diagonal-tension failure and flexural hinging.

## INTRODUCTION

The principal objective of the study is to explore the range of behavior exhibited by short reinforced concrete columns subjected to high shear stresses and cyclic bidirectional deformations. The cyclic bidirectional deformations though applied slowly to the test specimens represent the general characteristics of a seismic loading. The application of bidirectional deformations reflects the three-dimensional nature of seismic excitations. Several investigations [1,2,3] have indicated the need to consider the effects of bidirectional loadings on the response of a structure. The study of short columns was considered necessary because of the relative lack of research on such members and the reported failures of short columns during earthquakes, including the 1968 Tokachi-Oki earthquake in Japan [4] and the 1967 Caracas earthquake in Venezuela [5].

The current investigation on short columns was preceded by two others [6,7] which examined the effect of deformation path and axial load on the load-deflection characteristics of a short column. These earlier investigations established much of the procedure and apparatus used in the current study.

## EXPERIMENTAL INVESTIGATION

Eleven reinforced concrete short columns were tested in the current study. The test specimens represented a column bounded by large framing members which restrained rotation. The columns were subjected to slowly applied cyclic translations of the upper end relative to the bottom end to simulate the action of a building column subjected to seismic excitation.

---

[I] The University of Texas at Austin, Austin, Texas, USA.

The main variable in the test series was the ratio of shear capacity to flexural capacity in order to obtain a wide range of member behavior. Two other variables were also included in the investigation-- (1) axial load and (2) loading history. The capacity ratio (shear to flexure) was varied by altering the amounts of longitudinal and transverse reinforcement relative to each other.

The overall geometry of the specimen was not varied. The column was a 2/3-scale model of a prototype column 137 cm long with a 46 cm × 46 cm cross section and a 3.8 cm clear cover. The resulting test column was 91 cm long with a 30 cm × 30 cm cross section and a 2.5 cm clear cover. The longitudinal reinforcement was either eight 19 mm bars (86 series) or eight 13 mm bars (84 series) uniformly arranged around the section. The transverse reinforcement consisted of 6mm perimeter ties. The tie spacing in the test series varied from 2.9 cm to 30 cm. The range of tie spacings and the specimen configurations are illustrated in Fig. 1.

The column was rotationally restrained at each end and the lower end was held stationary. The upper end of the column was translated laterally relative to the lower end producing reversed curvature conditions in the column.

The same deformation path and loading history were used to test the majority of the eleven columns. The columns were deflected along their diagonals producing bilateral column deflections and forces. Three cycles of reversed deflection were applied along each diagonal at each deflection limit. A schematic of the loading history is shown in Fig. 2. A constant 535 kN compressive axial load was applied to the columns during testing. The axial load was approximately 50 percent of the axial load at balanced strain conditions.

Of the eleven columns, three were subjected to different loading conditions. Two of the three columns were monotonically deflected along a diagonal to a high deflection level, one having no applied axial load. The third specimen had ties at the minimum spacing and was cyclically loaded, but had no applied axial load.

The test schedule is listed in Table 1 which includes the key characteristics of each test.

Figure 3 shows a typical test setup. The specimen was placed between two crossheads and bolted to each. The lower crosshead was anchored to a reaction floor while the upper crosshead was attached to the three loading actuators and six rotation restraining rams. The lateral loading actuators had a maximum load capacity of 670 kN and a stroke of 30 cm. The axial actuator had a maximum capacity of 1335 kN and a stroke of 15 cm.

Load cells, linear potentiometers, and strain gages were used to monitor the performance of the specimen. The linear potentiometers measured the deflections imposed on the specimen both vertically and laterally. The strain gages were bonded to various locations on both the transverse and longitudinal reinforcement.

The acquisition of data and loading of the specimen were controlled through a computer based load control system. The data were reduced using computer based software which included digital plotting capabilities.

## OBSERVATIONS

General Load-Deflection Relations--The broad range of behavior observed in the investigation are typified by the load-deflection curves from three tests. Figure 4 shows the results of a specimen with 19 mm longitudinal bars and a tie spacing of 2.9 cm. The loss of lateral load and stiffness is fairly rapid after achieving the maximum lateral load. The degradation is restrained in comparison to the degradation of the load-deflection curve in Fig. 5 which is from a specimen with 19 mm longitudinal bars and a 30 cm tie spacing. The specimen in Fig. 5 exhibited a diagonal-tension failure and a nearly complete loss of load capacity. In contrast to the previous two load-deflection curves, the load-deflection curve for the specimen with 13 mm longitudinal bars and a 2.9 cm tie spacing (Fig. 6) exhibited much more stable hysteretic load-deflection curves. The degradation of load capacity at high deflections was the result of longitudinal bar buckling.

Transverse Reinforcement--A short column with ties at a 30 cm spacing exhibited a very brittle diagonal-tension failure. The maximum lateral load attained by the specimen was taken to be a result of the concrete shear capacity only. The maximum load achieved by the column was substantially (about 3 times) higher than the concrete shear capacity predicted by the 1977 ACI Building Code [8] Chapter 11 shear capacity equations.

The short column with ties at a 30 cm spacing had practically the same observed shear capacity as a short column with ties at 6.5 cm spacings. The observed shear capacity was 80 to 90 percent of the computed flexural capacity. Columns with smaller tie spacings (4.5 and 2.9 cm) achieved computed flexural capacity.

The columns with smaller tie spacings exhibited a more stable hysteretic load-deflection relation. However, because of sliding shear at the ends of the column it may be extremely difficult to achieve a true flexural hysteretic behavior (Fig. 7).

Longitudinal Reinforcement--Two different amounts of longitudinal reinforcement were used in the experimental investigation. The longitudinal reinforcement was either eight 19 mm bars or eight 13 mm bars. The most significant difference was the occurrence of bond degradation in the specimens with the 19 mm bars. The specimens with the 19 mm bars exhibited loss of concrete around the longitudinal bars primarily in the mid-height region of the column (Fig. 8).

The specimens with 19 mm bars exhibited more inclined cracking than the specimens with 13 mm bars. The increased amount of inclined cracking was the result of a higher flexural capacity. The increased flexural capacity led to a higher imposed shear force on the specimens with 19 mm bars as compared to the specimens with 13 mm bars.

The specimens with 13 mm longitudinal bars exhibited a more stable hysteretic behavior than the specimens with 19 mm bars. The load-deflection curves for the 13 mm bar specimens exhibited less pinching and did not exhibit the severe loss of stiffness that was common in the specimens with 19 mm bars. The improved behavior was attributable to less bond degradation along the 13 mm bars and a lower imposed shear force on the column as the result of a smaller flexural capacity in the 13 mm bar specimens.

## CONCLUSIONS

Based on the tests, the following general conclusions may be made:

(1) The shear capacity of a short column is most dependent on the capacity of the concrete to resist shear before inclined cracking. After cracking, the shear resistance of the column is strongly related to the effectiveness of aggregate interlock along the inclined cracks.

(2) The concrete contribution to the shear capacity of a short column is conservatively estimated by the shear provisions of the 1977 ACI Building Code. The shear provisions of the Code give a conservative estimate of the ultimate shear capacity of the short columns. However, the calculated values from the shear provisions do not follow the trend of the short column test data.

(3) In the short columns tested, there was a lower limit on the amount of transverse reinforcement which was required before an increase in the maximum lateral load of the column was observed. Varying the amount of transverse reinforcement while still below the limit did not cause a proportionate increase in the shear capacity of the short columns.

(4) Bond degradation has a significant detrimental effect on the short column load-deflection hysteresis loops. Bond degradation along the longitudinal bars is strongly affected by the boundary and loading conditions imposed on the test specimen. Double curvature and the lack of positive restraint to cover spalling on the sides of the column seemed to have a detrimental effect on bond conditions.

(5) In cyclically loaded columns which exhibit shear distress or serious bond degradation, increasing the deflection of the column does not necessarily cause the load-deflection curves to reach the monotonic load-deflection curve.

(6) In short columns which exhibit degrading hysteretic behavior, the degradation begins with cycling at the deflection at which the maximum lateral load is achieved and continues with both cycling and increased deflections.

(7) No single parameter uniquely determines the behavior a short column will exhibit, but there does seem to be a hierarchy of parameters which do define the member behavior.

## ACKNOWLEDGMENT

The support of the National Science Foundation through Grants ENV75-00192 and ENV77-20816 for this work is gratefully acknowledged. The opinions, findings, and conclusions are those of the author and do not necessarily reflect the views of the National Science Foundation.

## REFERENCES

1. Pecknold, D., and Sozen, M., "Calculated Inelastic Structural Response to Uniaxial and Biaxial Earthquake Motions," Fifth World Conference on Earthquake Engineering, Preprint Paper 223, Session 5B: Response of Structures to Ground Shaking, Rome, 1973.

2. Pecknold, D., and Suharwardy, M., "Effects of Two-Dimensional Earthquake Motion on Response of R/C Columns," Earthquake-Resistant Reinforced Concrete Building Construction, Proceedings of a Workshop Held at The University of California, Berkeley, California, July 1977, Volume II, Technical Papers, University of California, University Extension, Berkeley, California, June 1978, pp. 950-959.

3. Selna, L., and Lawder, J., "Biaxial Inelastic Frame Seismic Behavior," Reinforced Concrete Structures in Seismic Zones, American Concrete Institute, SP-53, Detroit, 1973, pp. 439-462.

4. Aoyama, H., et al., "A Study on the Cause of Damage to the Hachinohe Technical College due to 1968 Tokachi-Oki Earthquake, (part 1)," Proceedings of the U.S.-Japan Seminar on Earthquake Engineering with Emphasis on the Safety of School Buildings, Sendai, Japan, September 1970, The Japan Earthquake Engineering Promotion Society, Tokyo, Japan, 1971, pp. 199-212.

5. Sozen, M., et al., "Engineering Report on the Caracas Earthquake of 29 July 1967," Prepared for the Committee on Earthquake Engineering Research, Division of Engineering, National Research Council, National Academy of Engineering, Washington, D.C., 1968.

6. Maruyama, K., and Jirsa, J., "Shear Behavior of Reinforced Concrete Members under Bidirectional Reversed Lateral Loading," CESRL Report No. 79-1, The University of Texas at Austin, August 1979.

7. Ramirez, H., "Effect of Axial Loads on the Behavior of Reinforced Concrete Short Columns under Cyclic Lateral Deformations," unpublished Ph.D. dissertation, The University of Texas at Austin, Texas, 1979.

8. ACI Committee 318, Building Code Requirements for Reinforced Concrete (ACI 318-77), American Concrete Institute, Detroit, 1977.

TABLE 1 TEST SCHEDULE

| Specimen Identifier | Longitudinal Reinforcement | Tie Spacing* (cm.) | Axial Load (kN) | Loading History** |
|---|---|---|---|---|
| 0-86-14-DM | 8- 19mm Bars | 6.5 | 0 | Monotonic |
| C-86-14-DM | 8- 19mm Bars | 6.5 | 535 | Monotonic |
| 0-86-32-D | 8- 19mm Bars | 2.9 | 0 | Cyclic |
| C-86-32-D | 8- 19mm Bars | 2.9 | 535 | Cyclic |
| C-86-21-D | 8- 19mm Bars | 4.5 | 535 | Cyclic |
| C-86-14-D | 8- 19mm Bars | 6.5 | 535 | Cyclic |
| C-86-09-D | 8- 19mm Bars | 10.2 | 535 | Cyclic |
| C-86-03-D | 8- 19mm Bars | 30.5 | 535 | Cyclic |
| C-84-32-D | 8- 13mm Bars | 2.9 | 535 | Cyclic |
| C-84-21-D | 8- 13mm Bars | 4.5 | 535 | Cyclic |
| C-84-14-D | 8- 13mm Bars | 6.5 | 535 | Cyclic |

* Ties were 6mm bars.

** Deformation path was bilateral diagonal.

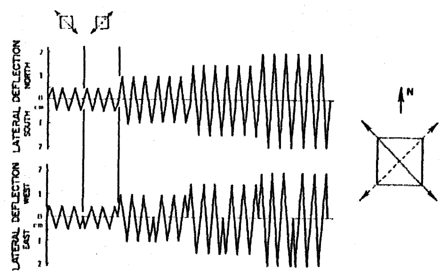


Fig. 2 Loading history

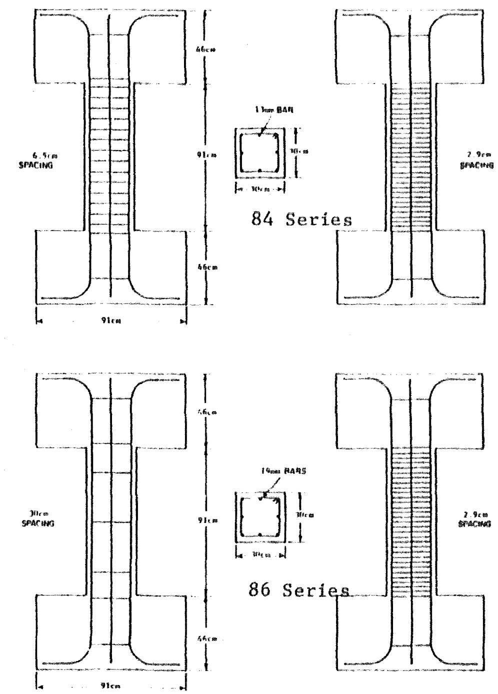


Fig. 1 Test specimens

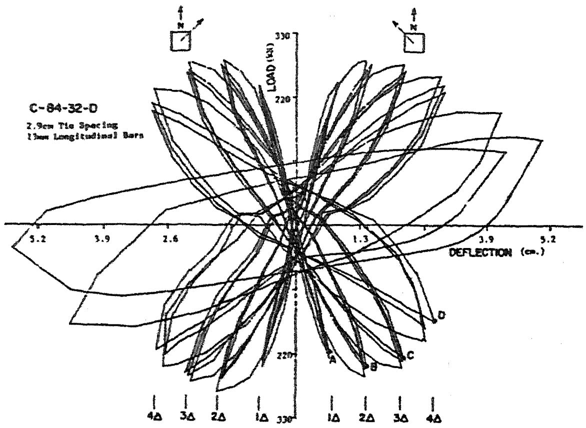


Fig. 6 Load-deflection curves, C-84-32-D

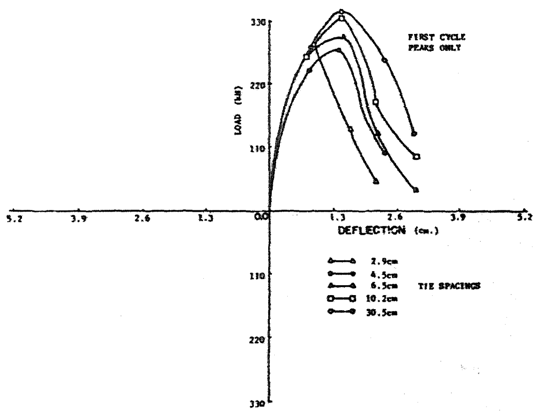


Fig. 7 Effect of varying tie spacing

Fig. 8 Midheight damage in specimens with 19 mm bars