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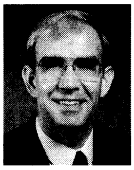
Effect of Reinforcing Bar Deformation Pattern on Flexural Ductility



by Michael L. Tholen and David Darwin

Concern has been expressed that increases in the stiffness of the bond force-slip relationship of reinforcing bars, resulting from increases in the relative rib area (R_r) of the bars, may have a negative impact on the flexural rotation capacity of reinforced concrete beams. To provide a better understanding of the effect of bar deformation pattern on flexural ductility, simply supported reinforced concrete beams were tested to determine load-deflection and moment-rotation behavior using both conventional (R_r = 0.069) and high relative rib area (R_r = 0.119) bars. Reinforcement ratios equal to 43 and 68 percent of the balanced reinforcement ratio were used. The tests show that a relatively large change in the relative rib area of reinforcing bars has no measurable effect on the distribution of flexural cracks or on the displacement and rotational capacity of beams in which plastic hinges develop.

Keywords: beams; bond (concrete to reinforcement); deformed reinforcement; flexural ductility; plastic hinging; reinforced concrete; relative rib area; rotational capacity; structural engineering.

Work has been underway since 1991 to improve the development characteristics of reinforcing bars.[1-7] A principal goal of the study has been to reduce development and splice length through changes in the deformation pattern of reinforcing bars. Early experimental efforts in the study by Darwin and Graham[2] using beam-end specimens to evaluate the bond strength of machined bars with relative rib areas R_r (ratio of projected rib area normal to bar axis to the product of the nominal bar perimeter and the center-to-center rib spacing) ranging between 0.05 and 0.20 showed that, under conditions in which the bars were confined by transverse reinforcement, increases in the relative rib area resulted in significant increases in bond strength. Darwin and Graham also observed that the initial stiffness of the bond force-slip curve increased as the relative rib area increased. Subsequent studies[3-7] have resulted in recommendations for the use of reinforcing bars with $R_r \geq 0.12$ in place of conventional bars which typically have values of R_r between 0.065 and 0.080. This potential change, however, has elicited concern that stiffer bond force-slip behavior may cause a reduction in the ductility of reinforced concrete members. This point was stated succinctly by Cairns[8] in a discussion of the paper by Darwin and Graham[2] in which he expressed the view that "the length of flexural reinforcement which attains yield at a plastic hinge will be reduced by stiffer bond characteristics, resulting in reduced plastic hinge rotation capacity."

To address this concern, this paper describes the tests and analyses of reinforced concrete beams used to investigate the effects of increased relative rib area on the load-deflection and moment-rotation response of reinforced concrete beams. Full details of the study are presented by Tholen and Darwin.[7]

RESEARCH SIGNIFICANCE

The significance of the tests reported in this paper is two-fold. First, the results impact the potential for the use of high relative rib area bars in practice. If the ductility of reinforced concrete members is adversely affected by improved bond behavior, then the industry should be attempting to find the optimum deformation pattern for both bond and flexural performance. Second, there are active discussions in Europe aimed at reducing the relative rib area of reinforcing bars to achieve improved ductility. Those discussions are principally based on analytical, not experimental, studies.[9,10] As will be demonstrated in the paper, concerns on either point do not seem to be justified.

EXPERIMENTAL PROGRAM

This paper describes the results of four beam tests that were designed to determine the level of the potential for reduced ductility as a result of increases in the relative rib area

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*ACI member **Michael L. Tholen** is a structural engineer with Burns & McDonnell International in Kansas City, Missouri. He holds a BS in architectural engineering and an MS and PhD in civil engineering from the University of Kansas.*

***David Darwin**, FACI, is the Deane E. Ackers Professor of Civil Engineering and Director of the Structural Engineering and Materials Laboratory at the University of Kansas. He is a past member of the Board of Direction and the Technical Activities Committee and is past-president of the Kansas Chapter of ACI. Darwin is also past-chairman of the Publications Committee and the Concrete Research Council, and a member and past-chairman of ACI Committee 224, Cracking. He chairs the TAC Technology Transfer Committee and serves on ACI Committees 408, Bond and Development of Reinforcement; 446, Fracture Mechanics; and ACI-ASCE Committees 445, Shear and Torsion, and 447, Finite Element Analysis of Reinforced Concrete Structures. He is a recipient of the Arthur R. Anderson and ACI Structural Research Awards.*

of flexural reinforcement. The main test parameters were relative rib area and reinforcement ratio. The specimens consisted of two matched pairs of beams. One set of specimens contained two No. 8 (25 mm) bars, providing a reinforcement ratio $\rho = 43$ percent of the balanced reinforcement ratio ρ_b (based on actual material properties). The second set of specimens contained three No. 8 (25 mm) bars, with $\rho = 0.68\rho_b$. One beam in each group contained conventional reinforcement, while the other beam contained high relative rib area bars.

Test specimens

The beams were 16 ft (4.9 m) long with nominal widths and depths of 12 and 16 in. (305 and 406 mm), respectively. The specimens were tested as simply supported beams with a concentrated load at the center, as shown in Fig. 1(a). The beams contained two or three continuous bottom-cast test bars [Fig. 1(b)], with nominal bottom and side covers of 2 in. (51 mm). No. 3 (9.5 mm) closed stirrups, spaced 6 in. (152 mm) on center were used throughout the beams to provide shear strength. No. 4 (13 mm) bars were used as top reinforcement. Actual member dimensions are given in Table 1.

Materials

Reinforcing steel—All bars met the requirements of ASTM A 615,[11] except that the high relative rib area bars had no bar markings. The two deformation patterns are shown in Fig. 2. The bars were selected because of their similar yield strengths and the range of relative rib areas. The conventional bars, designated 8N0, have a relative rib area of 0.069 and yield strength of 78.7 ksi (543 MPa). The high R_r bars, designated 8N3, have a relative rib area of 0.119 and a yield strength of 80.5 ksi (555 MPa). Both bars were produced from the same heat of steel. The yield strengths are based on tests of 5 and 6 samples for the 8N0 and 8N3 bar designations, respectively. Bar properties are given in Table 2. The yield strengths of the bars are just above the maximum allowable [78 ksi (538 MPa)] for low-alloy ASTM A

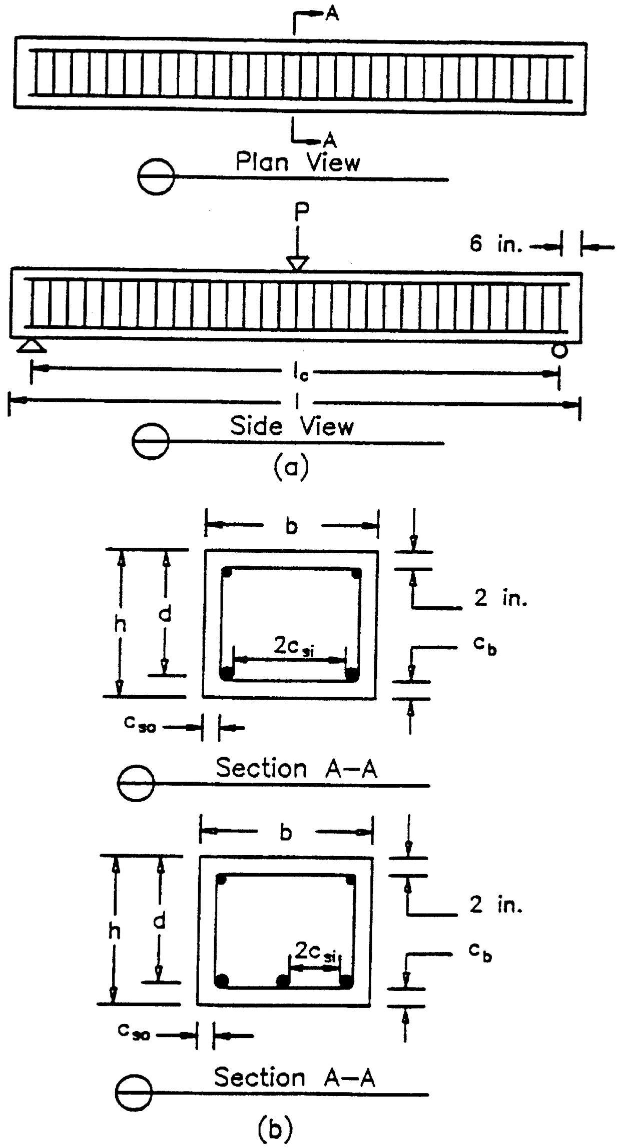

Fig. 1—Moment-rotation test specimens: (a) as tested; (b) cross sections (1 in. = 25.4 mm)

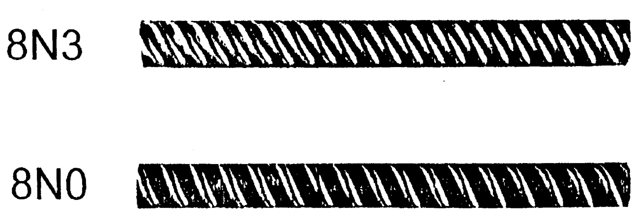

Fig. 2—Reinforcing bar deformation patterns, No. 8 (25 mm) bars

Table 1—Moment-rotation test specimen properties

Bar designation	Number of bars	c_{so}, in.	c_{si}, in.	c_b, in.	d, in.	b, in.	h, in.	l, ft	l_c, ft	a,* in.
8N0	2	2.125	2.813	1.926	13.67	12.00	16.10	16.00	15.00	9.04
8N3	2	2.078	2.875	1.971	13.61	12.06	16.09	16.00	15.00	9.01
8N0	3	2.094	1.156	1.919	13.67	12.04	16.08	16.00	15.00	9.01
8N3	3	2.031	1.094	1.915	13.72	12.06	16.13	16.00	15.00	9.05

*See Fig. 3
1 in. = 25.4 mm; 1 ft = 305 mm

Table 2—Properties of reinforcing bars

Bar designation	Yield strength, ksi	Tensile strength, ksi	Elongation, percent	Nominal diameter, in.	Weight, lb/ft	Percent light or heavy	Rib spacing, in.	Rib height		Relative rib area
								ASTM, in.	Average,* in.	
8N0	78.7	117.6	15.3	1.000	2.594	2.8 L	0.650	0.057	0.054	0.069
8N3	80.5	118.9	15.0	1.000	2.730	2.2 H	0.487	0.072	0.068	0.119

*Average rib height between longitudinal ribs

1 in. = 25.4 mm; 1 ft = 305 mm; 1 ksi = 6.89 MPa; 1 lb/ft = 1.49 kg/m; 1 lb/yd^3 = 0.5933 kg/m^3

706[12] reinforcing bars. However, as shown in Table 2, with tensile strengths of 117.6 and 118.9 ksi (811 and 820 MPa), respectively, and elongations at failure of 15.3 and 15.0 percent, the 8N0 and 8N3 bars satisfy the other tensile requirements of ASTM A 706[12] (tensile strength ≥ 1.25 times yield strength and elongation ≥ 12 percent for No. 8 [No. 25] bars). Therefore, the tests reported in this paper would likely produce similar results if duplicated using A 706 reinforcing bars.

Conventional ASTM A 615[11] Grade 60 bars were used for stirrups and top reinforcement.

Concrete—Air-entrained concrete was supplied by a local ready-mix plant and contained Type I portland cement, Kansas River sand, and $^3/_4$ in. (19 mm) maximum nominal size crushed limestone coarse aggregate. A water-cement ratio of 0.44 was used to produce concrete compressive strengths of 5170 and 4790 psi (36 and 33 MPa) at test ages of 23 and 20 days for the specimens with $\rho = 0.43\rho_b$ and $0.68\rho_b$, respectively.

Test procedure

The specimens were tested as simply supported beams with a concentrated load at the center [Fig. 1(a)], supported 6 in. (305 mm) from the ends by pin and roller supports. Steel plates, attached to the specimen with a thin layer of high-strength gypsum cement, separated the beam from the supports. The load was applied using two 60-ton hollow-core hydraulic jacks. Two load rods were used to transfer the force from the jacks to a steel spreader beam mounted across the specimen. The rods were attached to the spreader beam with semicircular rollers to keep the applied load vertical. The beams were painted with dilute white latex paint to make cracks more visible. Load was applied at about 3 kips (13 kN) per minute up to approximately 90 percent of the yield load. During this period, load was held constant at 5 kip (22 kN) increments as cracks were marked. Above 90 percent of the yield load, the beams were loaded continuously to failure, after which any additional cracks were marked.

Centerline deflection was measured with a spring-loaded linear variable differential transformer (LVDT). Load was measured using the load rods, which were instrumented as load cells. Flexural rotation of the beam on each side of the load point was measured using pairs of LVDTs, as shown in Fig. 3. These LVDTs were attached to $^1/_2$ in. (12.7 mm) diameter rods cast into the specimen. Two LVDTs were mounted on the north face of the beam and measured the rotation on the east side of the load point. Two other LVDTs were mounted on the south side of the beam and measured rotation on the west side of the load point. Cracks were marked on both sides of the beam, opposite the face on which the rotations were measured.

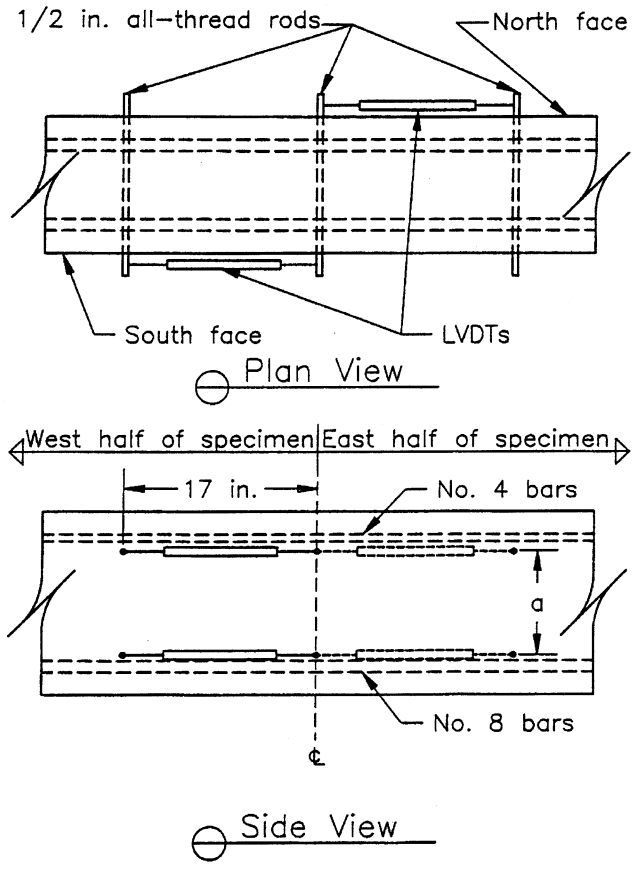

Fig. 3—Schematic of rotation measurement setup (1 in. = 25.4 mm)

RESULTS AND OBSERVATIONS

Load-deflection curves for the specimens are shown in Fig. 4 and 5. Moment rotation curves are shown in Fig. 6 and 7.

The rotation of the beam on each side of the load point was calculated from the LVDT readings as follows:

$$\theta = \tan^{-1}\left(\frac{\Delta_t + \Delta_b}{a}\right) \quad (1)$$

where

θ = rotation, rad

Δ_t = deflection of the top LVDT, positive in compression (Fig. 3)

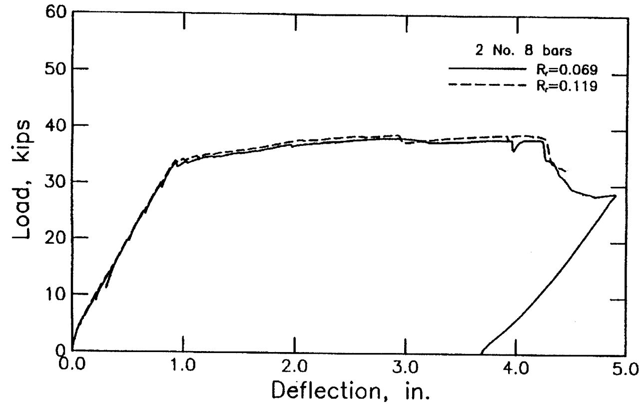

Fig. 4—Load-deflection curves for beams with $\rho = 0.43\ \rho_b$ *(1 kip = 4.45 kN; 1 in. = 25.4 mm)*

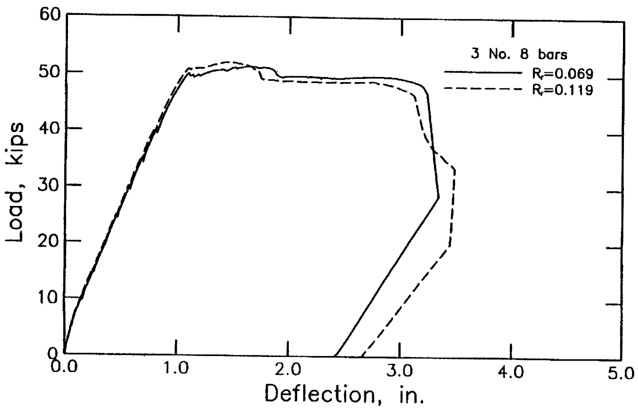

Fig. 5—Load deflection curves for beams with $\rho = 0.68\ \rho_b$ *(1 kip = 4.45 kN; 1 in. = 25.4 mm)*

Δ_b = deflection of the bottom LVDT, positive in tension (Fig. 3)

a = average vertical distance between the top and bottom LVDTs (Table 1 and Fig. 3)

The rotations shown in Fig. 6 and 7 represent the average values for both sides of the load point.

As expected, the specimens with three bars produced higher loads and stiffnesses up to yield than did the specimens with two bars. After reaching the yield load, the stiffnesses of the load-deflection and moment-rotation curves greatly decreased as a plastic hinge developed near the load point. Loading continued until a significant drop in load was produced by crushing the concrete near the load point. Specimens with three bars failed in compression soon after yielding, while specimens with two bars produced significant additional displacement and rotation after yielding.

As shown in Fig. 4-7, the load-deflection and moment-rotation curves for the matched pairs of specimens are nearly identical. These figures indicate that an increase in relative rib area from 0.069 to 0.119 does not have a measurable effect on the displacement or rotational capacity of beams in which plastic hinges form.

Crack patterns for the west half of the specimens are shown in Fig. 8 and 9. Flexural cracks initially formed near the load point. As the load increased, these cracks grew, while more cracks developed farther away from the load point. Figures 8 and 9 show that the crack patterns for the matched pairs of beams are similar and that the spacing and distribution of flexural cracks is not affected by the relative rib area of the flexural reinforcement.

Overall, the test results indicate that aspects of the nonlinear behavior of reinforced concrete, other than differences in the bond force-slip relationship caused by changes in R_r, control moment-rotation behavior. The test results do not support the concerns expressed about the effects of increased relative rib area on the rotational capacity of plastic hinges in beams containing high relative rib area bars.

CONCLUSIONS

Although the scope of this study is limited, the great similarity between the behavior of matched pairs of flexural specimens indicates the general insensitivity of flexural response to large changes in the relative rib area of reinforcing bars. The key conclusions of the study are:

1. For beams with continuous reinforcement, the relative rib area of the reinforcing bars does not affect the distribution of flexural cracks for beams in which plastic hinges develop.

2. For beams with continuous reinforcement, the relative rib area of the reinforcing bars does not affect the displacement or rotational capacity of beams in which plastic hinges develop.

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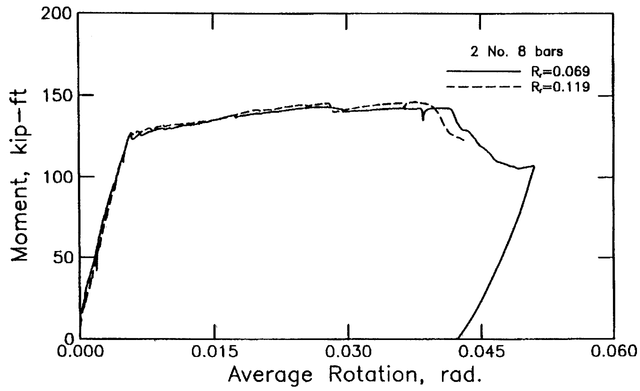

Fig. 6—Moment-rotation curves for beams with $\rho = 0.43\ \rho_b$ *(1 kip = 4.45 kN; 1 in. = 25.4 mm)*

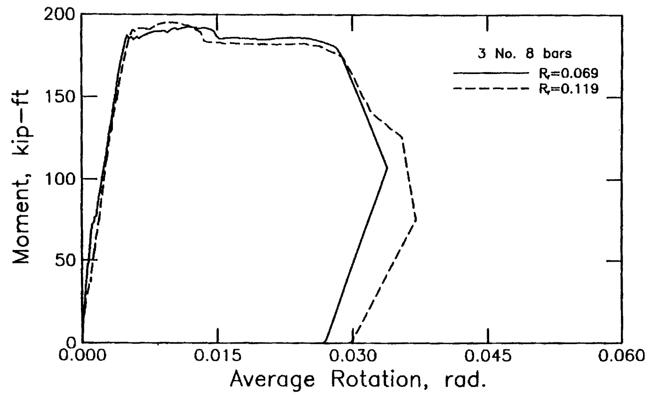

Fig. 7—Moment-rotation curves for beams with $\rho = 0.68\ \rho_b$ *(1 kip = 4.45 kN; 1 in. = 25.4 mm)*

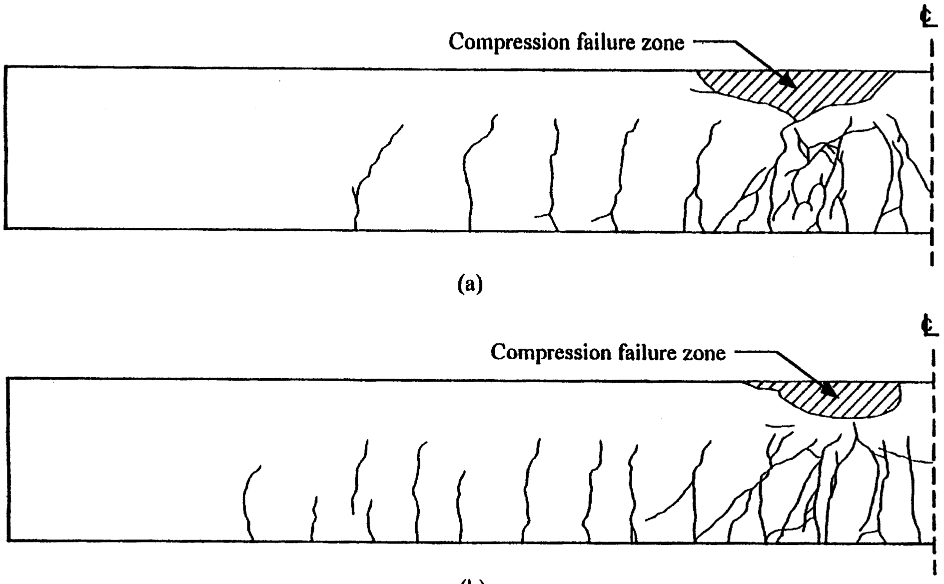

Fig. 8—Crack patterns for west half of beams with $\rho = 0.43\ \rho_b$*: (a)* $R_r = 0.069$*; (b)* $R_r = 0.119$

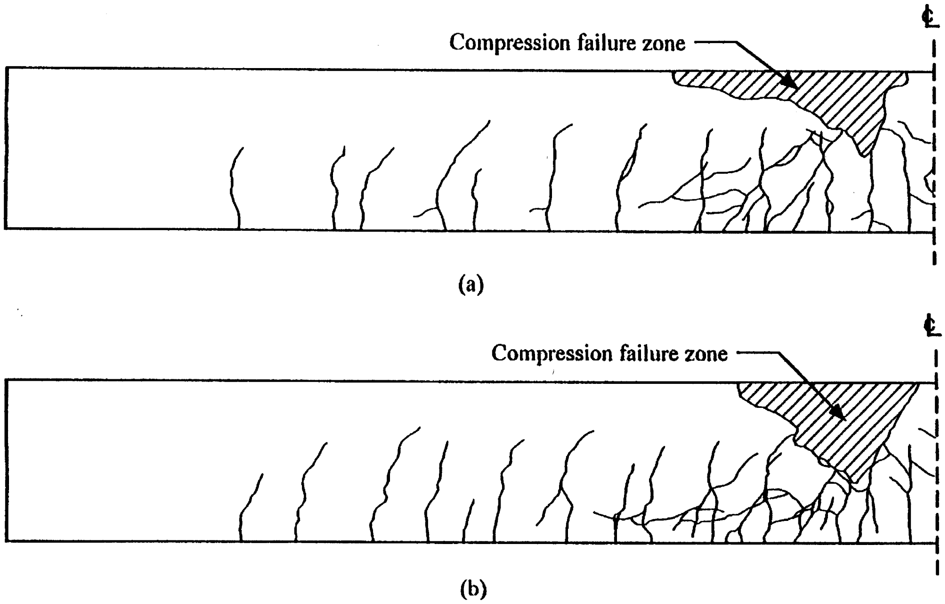

Fig. 9—Crack patterns for west half of beams with $\rho = 0.68\ \rho_b$*: (a)* $R_r = 0.069$*; (b)* $R_r = 0.119$

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