FACTORS INFLUENCING THE HINGING BEHAVIOR OF REINFORCED CONCRETE MEMBERS UNDER CYCLIC OVERLOADS

by

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SYNOPSIS

Load-deflection curves of reinforced concrete members subjected to varying load histories are presented. The results provide a means of evaluating the factors influencing the ability of reinforced concrete members to withstand cyclic loads. The primary variable is the loading history. Additional variables considered include the amount and strength of the beam reinforcement, the tie spacing, the shear span, and the level of axial load. The tests indicate that the strength and stiffness are reduced by a gradual deterioration of the shear capacity of the concrete in the hinging region and by deterioration of bond along anchored bars.

INTRODUCTION

Plastic hinging in reinforced concrete frames subjected to seismic loads is generally located in the vicinity of beam-column joints. To adequately design a structure for seismic effects, information regarding the ductility and cyclic load capacity of members is needed. Much information regarding hinging capacity of beams is available from tests of simple beams with hinges at the midspan or adjacent to column stubs at the midspan. However, the hinging behavior of members near supports such as flexural members at exterior column supports has not been explored in detail. The objective of this paper is to review recent research carried out at Rice University (1,2,3) and to use these results to provide a means of determining situations in which hinging capacity of members may be insufficient for seismic resistant structures.

EXPERIMENTAL PROGRAM

To investigate hinging behavior, tests were conducted on cantilever beams which framed into an enlarged end block supporting the beam (Fig. 1). The specimen was intended to simulate an exterior beam-column joint. The end block (or pseudo-column) was subjected to a compressive stress of 1000 psi which was maintained throughout testing. The end block was lightly reinforced with #3 bars. The dimensions of the beam were 6 x 12 in. The prime variable was the loading history. The deflection limit in each direction was either 5 or 10 times the yield deflection. The upward deflection limit was determined by monitoring strains in the longitudinal steel and adding an increment of 5 or 10 Δ_y from the point at which yield was reached. The beams were subjected to slow cycle static loads. In addition to load history, the effect of the following variables on hinging capacity was studied:

(a) Percentage and yield strength of beam reinforcement; #6 or #8 bars, top and bottom ($\rho = \rho' = 0.015$ or 0.026), $f_y = 45$ or 60 ksi.

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(b) Stirrup or tie spacing, #3 closed ties at 2 to 5 in. spacing.
(c) Shear span of cantilever beam, L = 30 or 60 in.
(d) Level of axial load on beam, N varying from 50 to 100% of the balance load N_b on the beam.

The specimens were cycled until failure occurred either by (a) severe reduction in load capacity, (b) buckling of the beam reinforcement, (c) disintegration of concrete in the hinge, or (d) destruction of stress transfer capacity of the anchored bars in the support.

Loads, steel strains, end deflections and deformations of the top and bottom concrete fibers at selected intervals in the hinging region were continuously monitored.

GENERAL OBSERVATIONS

Typical load-deflection curves are shown in Figs. 2-4. Based on the results of 34 tests, the following trends were observed:

(1) The response under load reversal was markedly nonlinear due to a combination of (a) the Bauschinger effect in the steel, (b) shear deformation, (c) closure of residual cracks, (d) nonlinear load-slip behavior of the anchored reinforcement. Addition of axial load tended to reduce the influence of shear deformations and cracking.

(2) The number of cycles to failure was generally improved by reducing stirrup spacing. Compare Fig. 3a and 3b. Fewer cycles to failure were observed in those tests in which the shear span was reduced, or the shear force increased (Fig. 2a and 2b). In some cases with L = 30 in. failure occurred in fewer than 5 cycles of load at five times the yield deformation. The number of cycles to failure was reduced with an increase in the deflection limits as indicated by Fig. 3a and 3c.

(3) Deformation of the anchored beam reinforcement (in the "pseudo-column") contributed significantly to the energy absorbing capacity of the specimens. Bond deterioration along the anchored bar led directly to failure after 2 cycles of a specimen reinforced with steel with f_y = 60 ksi and subjected to large deflection limits (Fig. 4a). With the addition of axial load the mode of failure was changed from bond deterioration to concrete crushing and buckling of the bars (Fig. 4b) during the fourth load cycle.

(4) Specimens reinforced with 60 ksi yield strength steel generally behaved as well as those reinforced with 45 ksi yield steel. The specimen shown in Fig. 3c was loaded through 45 cycles with a gradual reduction in strength and stiffness. A similar specimen reinforced with steel having a yield strength f_y = 45 ksi withstood about the same number of cycles.

(5) Failure of specimens without axial load was generally initiated by large shear deformations along nearly vertical planes which were not crossed by the shear reinforcement. The gradual deterioration of the concrete along the vertical flexural cracks and the opening of diagonal shear cracks led to a reduction of load capacity and stiffness. As a result the load-deflection relationships became "pinched" toward the origin, as can be seen in Figs. 2a, b, and 3a, b, c. Specimens subjected to axial load generally failed by a combination of deterioration of concrete in shear and buckling of the longitudinal beam reinforcement in a region where severe

crushing had occurred; however, the reduction in strength and stiffness was much less severe (Figs. 2c, d and 3d). In the first cycle of all the specimens subjected to axial load, the load peaked at a deflection less than the deflection limit of 5 or 10 Δ_y. The "hump" in the curve is particularly noticeable in the upward loading direction (-P) and is produced by crushing and loss of the concrete outside the confined core.

CONCLUSIONS

The results obtained in this study provide a means of evaluating some of the variables influencing the plastic hinging behavior of reinforced concrete members. These concluding remarks will be divided into those aspects of behavior which are important from the standpoint of analysis and design of seismic resistant structures.

To analyze a structure for seismic loadings, the force-deformation relationships for the members must be established. The results indicate that the load-deflection curve for the first cycle of loading of a reinforced concrete beam subjected to flexure is unique and nonrecurring. In most cases the second cycle and a number of subsequent cycles are similar. With proper representation of stress-strain relationships of the concrete and steel and of the load-slip characteristics (2) of the anchored bars, the response of the second and subsequent cycles can be predicted (1,3). Very little work has been done regarding load-slip relationships for anchored reinforcing bars and this represents an area of needed research.

To design a structure to maintain energy-absorbing capacity through a number of load cycles, the test results indicate that specimens with deflection limits of $5\Delta_y$ performed well through at least 5 cycles of load (generally 10 or more) unless the shear span was short (30 in.) in which case the load capacity and stiffness degraded rapidly. With large axial loads ($N > 0.5N_b$), failure was more sudden and in some tests occurred in less than 5 cycles. With a deflection limit of $10\Delta_y$ the specimens maintained load and energy-absorbing capacity only if the stirrups were spaced closely enough to confine the core ($s \leq 2$ in.), if the bars were adequately anchored and if no axial load was applied. In most cases, the deterioration of the concrete in shear across the hinge region was very severe with deflection limits of $10\Delta_y$. This shear deterioration occurred even with closely spaced stirrups ($s = 2$ in.) and it is unlikely that a closer spacing or larger stirrups would have substantially altered the results. It would appear that only by reducing the shear force or increasing the concrete area would the beams have performed adequately. With regard to anchorage requirements, very little work has been done to evaluate the strength of anchored straight or hooked bars under cyclic loads and varying confinement conditions and is an area of much needed research.

REFERENCES

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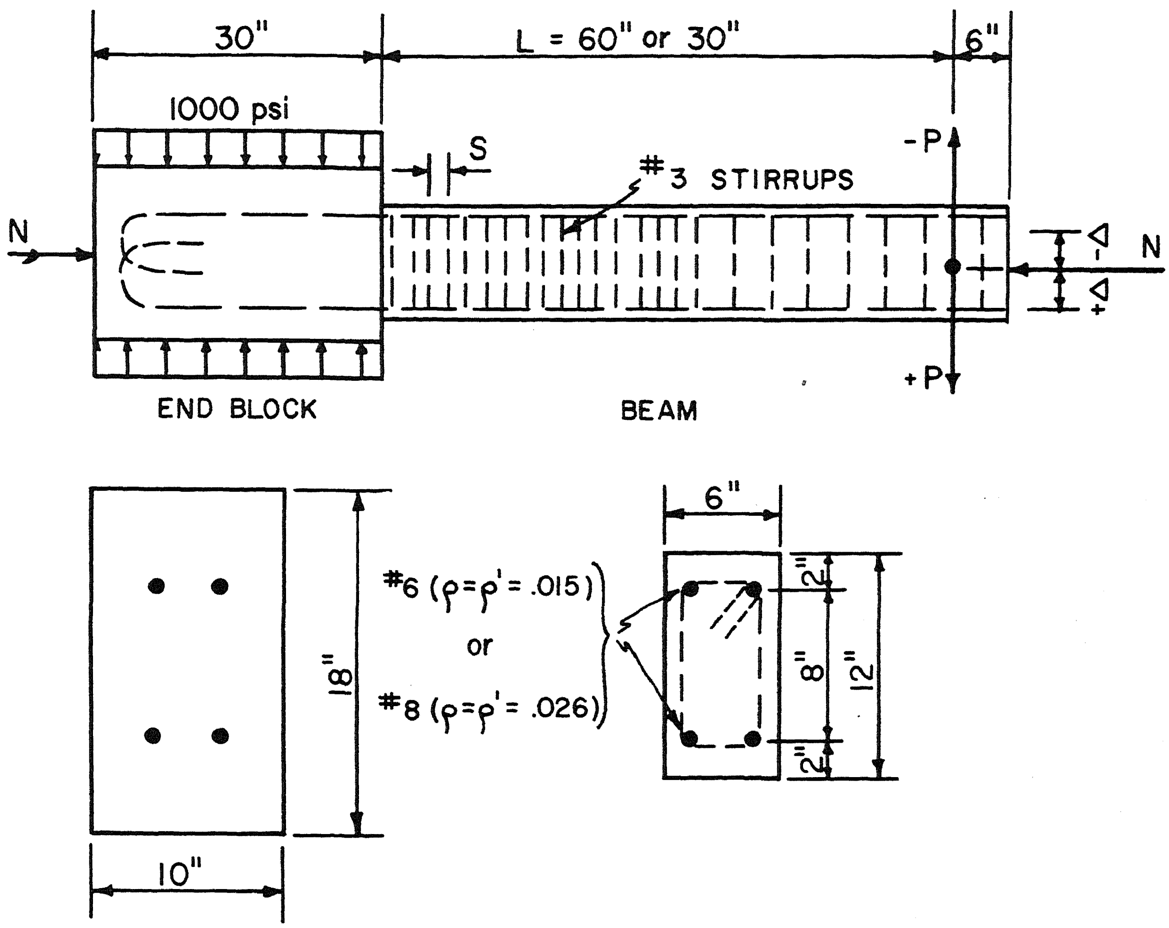

FIG. 1 TEST SPECIMEN

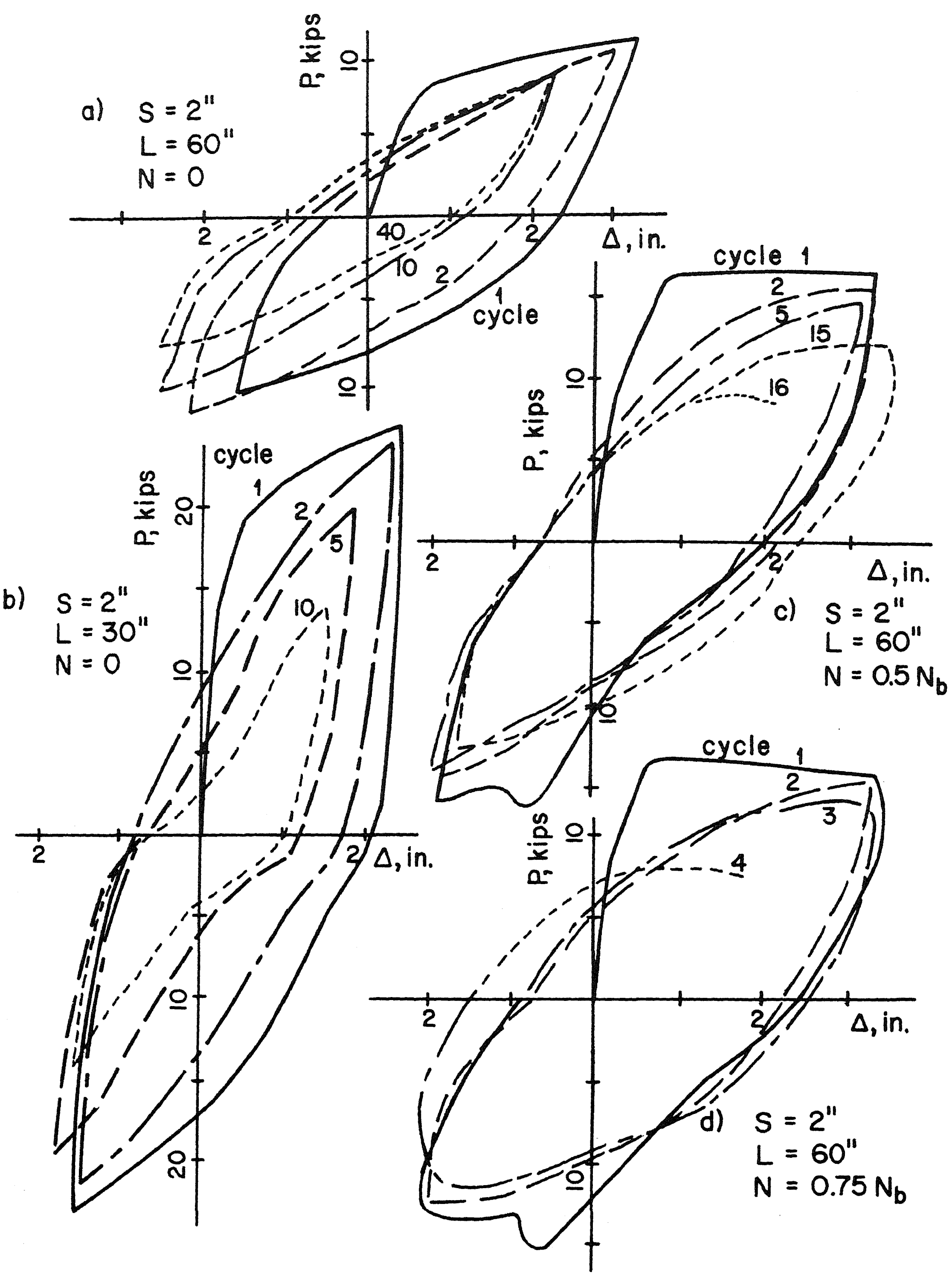

FIG. 2 LOAD-DEFLECTION CURVES, SPECIMENS WITH #6 BARS ($\rho = \rho' = 0.015$, $f_y = 60$ ksi)

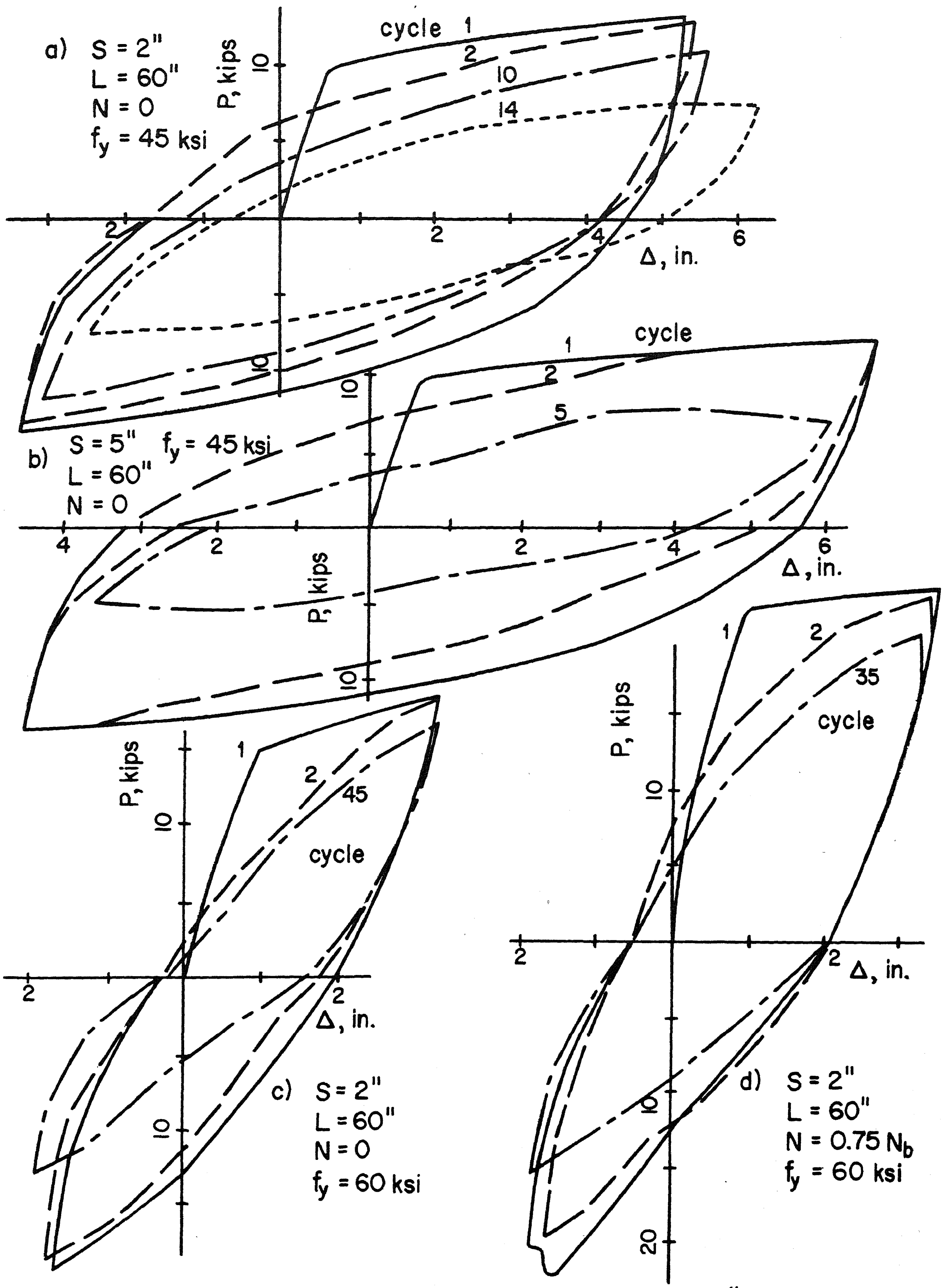

FIG. 3 LOAD - DEFLECTION CURVES, SPECIMENS WITH #8 BARS
($\rho = \rho' = 0.026$)

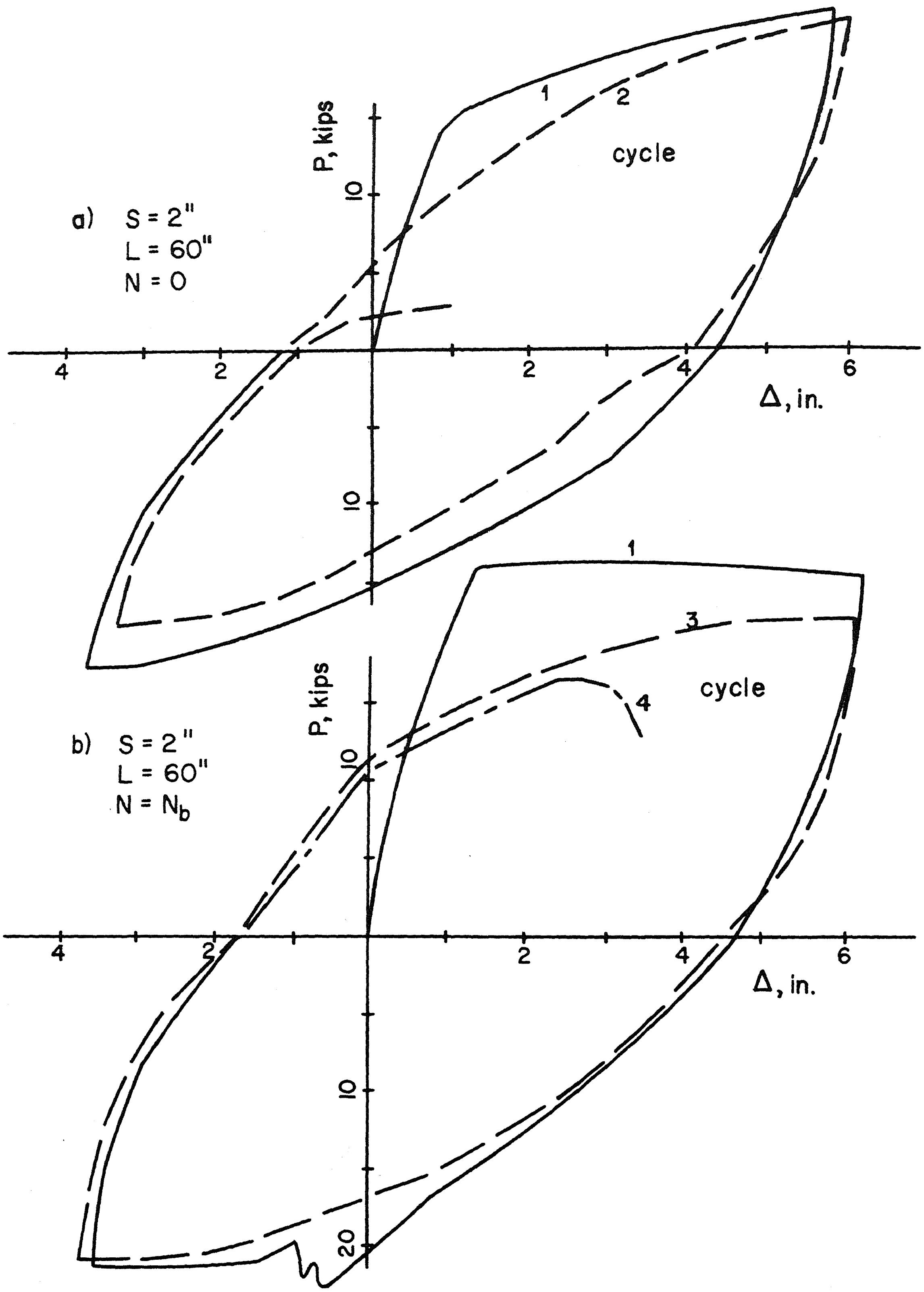

FIG. 4 LOAD-DEFLECTION CURVES, SPECIMENS WITH #8 BARS
($\rho = \rho' = 0.026$, $f_y = 60$ ksi)