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Strength Decay of RC Columns under Shear Reversals

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Introduction

This paper is concerned with energy dissipation through flexural deformation of reinforced concrete elements with or without axial loads. Specifically, the writers examine the question whether current procedures used for the design of web reinforcement, as given by the latest building code of the American Concrete Institute (1), ACI 318-71, are applicable to earthquake resistant design, if the design basis assumes a number of excursions into the inelastic range of response.

A series of 12 test specimens, some loaded axially, were subjected to large transverse displacement reversals. The results are described and interpreted herein.

Previous Investigations

Several experimental investigations of the behavior of reinforced concrete members subjected to large shear reversals have been conducted in the United States and Japan. Most of the Japanese investigations were initiated after the Tokachi-Oki Earthquake of 1968. During this earthquake several reinforced concrete columns failed in shear and consequently, the resulting experimental investigations were concerned primarily with axially loaded members. Results of investigations by Hisada, et al. (2), Ikeda (3), and Kanoh, et al. (5) on reinforced concrete columns subjected to large shear reversals emphasized the following: (1) A minimum shear span to depth ratio of 2.0 is required to ensure a flexural failure; (2) the axial load must be kept below one-third of the ultimate axial load to ensure some ductility; (3) transverse reinforcement consisting of closed rectilinear ties are preferred over circular hoops; and (4) a minimum

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transverse reinforcement ratio of 0.6% is required to suppress a shear failure.

Experimental investigations in the United States on reinforced concrete members under large shear reversals have been limited to members with no axial load. Results reported by Jirsa and Brown (4) indicated that the ability of their test specimens to maintain load capacity and a substantial energy dissipation capacity was improved by reducing the spacing between stirrups in a flexural hinging zone. However, closely spaced stirrups in their tests did not prevent shear failure along nearly vertical planes not crossed by stirrups. Results reported by Popov, et al. (6) also emphasized the severity of shear deformations along nearly vertical cracks.

Experimental Program

The principal variables of the testing program were: (1) Axial load, which varied from zero to one-half the balance load (the load calculated for tensile yielding strain in the steel and a limiting compressive strain of 0.003 in the extreme concrete fiber); (2) transverse reinforcement ratio which varied from 0.33%-1.47%; and (3) total deflection for each cycle.

Specimens represented that part of two columns between the points of contraflexure above and below a story level. Total specimen length was 102 in. (2,590 mm), with 34.5 in. (876 mm) between the load point and the face of the central joint (Fig. 1). Dimensions of the columns were 6 in. × 12 in. (150 mm × 300 mm). The gross reinforcement ratio was 2.4% (four No. 6 bars). The central joint was 21 in. (530 mm) long with cross-sectional dimensions of 18 in. × 16 in. (460 mm × 410 mm). Pea gravel concrete with a target compressive strength of 5,000 psi (34 MN/m^2) and grade 60 longitudinal reinforcement were used throughout this investigation. The stirrups in each specimen were either grade 40 No. 2 plain bars or grade 40 No. 3 deformed bars. Grade 60 No. 4 deformed bars were used as additional shear reinforcement in the central joint.

During the test, a servoram attached to the specimen applied an axial load through a pair of external cables, and a pair of hydraulic jacks were used to hold the central joint stationary. The clamping load applied to the central joint and the axial load were held constant throughout the test.

The two columns were simultaneously deflected in opposite directions through several cycles of load reversals with frequent stops to record applied shear, deflection, rotation at the joint, and strains in the reinforcing steel. The applied deflections usually followed one of the deflection schedules shown in Fig. 2 (yield deflection Δ_y defined as the deflection corresponding to yielding of the longitudinal reinforcement), but the amount of damage suffered by the specimen sometimes forced a departure from the intended deflection schedule. One load cycle took approx 20 min to complete.

Each specimen is designated by two numbers separated by a decimal. The first number denotes the amount of axial load, in kips, and the second number represents the transverse reinforcement ratio times 10^4 (Table 1).

Mean concrete properties, amount of axial load, transverse reinforcement ratio, and intended deflection schedule for each specimen are summarized in Table 1. The scatter of measured compressive and splitting strength from the

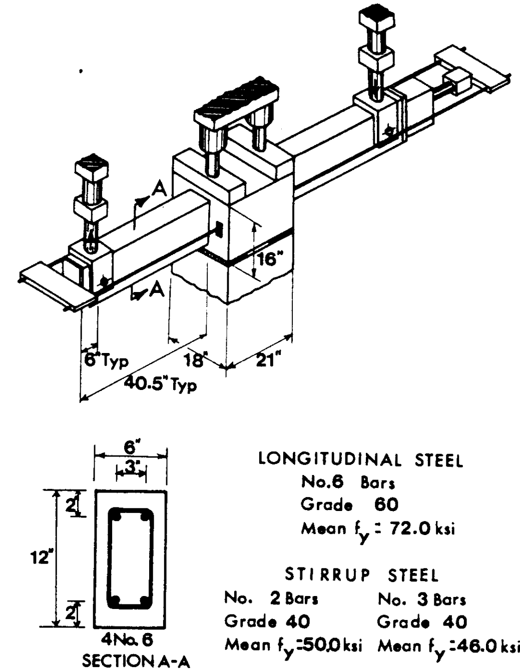


FIG. 1.—Test Setup, Dimensions, and Properties of Specimens (1 in. = 25.4 mm; 1 ksi = 6.89 MN/m^2)

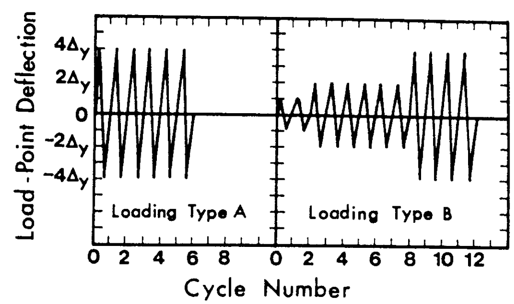


FIG. 2.—Deflection Schedules

concrete test cylinders and yield stress from the reinforcing steel coupons is given in Ref. 7.

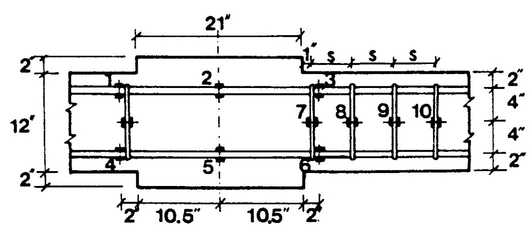
Measurements and Instrumentation

Load cells [sensitivity = 0.1 kip (440 N)] and differential transformers [sensitivity = 0.01 in. (0.25 mm)] were used to measure the applied shear and resulting load-point deflection at each end of the specimen. After each deflection increment was applied, the following additional measurements were recorded: (1) The relative displacement between the face of the joint and a

TABLE 1.—Summary of Experimental Program

Mark (1)	Deflection schedule[a] (2)	Axial load, in kips (3)	Concrete: Compression strength,[b] in pounds per square inch (4)	Concrete: Splitting strength,[c] in pounds per square inch (5)	Transverse Reinforcement: Size number (6)	Transverse Reinforcement: Spacing, in inches (7)	Transverse Reinforcement: Reinforcement ratio (8)
40.033A	A	42.5	5,030	391	2	5.0	0.0033
40.033	B	40	4,870	378	2	5.0	0.0033
25.033	B	25	4,880	400	2	5.0	0.0033
00.033	B	0	4,640	359	2	5.0	0.0033
40.048	B	40	3,780	320	2	3.5	0.0048
00.048	B	0	3,750	356	2	3.5	0.0048
40.067	B	40	4,840	398	2	2.5	0.0067
00.067	B	0	4,610	379	2	2.5	0.0067
40.092	A	40	5,150	438	3	4.0	0.0092
00.105	A	0	4,850	419	3	3.5	0.0105
40.147	A	40	4,860	423	3	2.5	0.0147
00.147	A	0	4,900	414	3	2.5	0.0147

[a] See Fig. 2.
[b] Mean of six compression tests on 6-in. × 12-in. (300-mm) cylinders.
[c] Mean of six splitting tests on 6-in × 6-in. (150-mm) cylinders.
Note: 1 kip = 4.45 kN; 1 in. = 25.4 mm; 1 psi = 6.89 kN/m². See Fig. 1 for mean yield stress of reinforcement.



Note: Pairs of Gages Were Used at All Gage Points to Eliminate the Bending Component of Measured Strain

FIG. 3.—Strain Gage Locations (1 in. = 25.4 mm)

reference point 10 in. (250 mm) from the face of the joint; and (2) the strains in the longitudinal and transverse reinforcement at several locations (Fig. 3). The displacements between the face of the joint and the reference point were

measured with differential transformers. Reinforcement strains were measured with pairs of high-elongation etched foil strain gages.

Strength Characteristics

Specimens were designed to have an initial shear capacity greater than or equal to their initial flexural capacity. The ultimate shear due to flexure for the test specimens is

$$V_{uf} = \frac{M_u}{a} \quad (1)$$

in which M_u = ultimate moment; and a = shear span. Eq. 1 ignores the P

TABLE 2.—Shear Capacities of Specimens

Mark (1)	Ultimate shear based on M_u, V_{uf}, in kips (2)	Shear capacity of concrete, V_c, in kips (3)	Shear capacity of stirrups, V_s, in kips (4)	Nominal shear capacity of specimen, V_u, in kips (5)	Measured maximum shear, V_m, in kips (6)	V_u/V_{uf} (7)	V_u/V_m (8)
40.033A	20.6	11.2	10.0	21.2	21.5	1.03	0.99
40.033	20.4	10.7	10.0	20.7	21.9	1.01	0.95
25.033	18.9	9.8	10.0	19.8	19.7	1.05	1.01
00.033	16.1	8.2	10.0	18.2	18.2	1.13	1.00
40.048	19.9	9.4	14.2	23.6	21.4	1.19	1.10
00.048	15.7	7.3	14.2	21.5	19.3	1.37	1.11
40.067	20.7	10.7	19.9	30.6	20.2	1.48	1.51
00.067	16.2	8.1	19.9	28.0	20.4	1.73	1.37
40.092	20.5	11.0	25.3	36.3	23.4	1.77	1.55
00.105	16.3	8.4	28.9	37.3	23.6	2.29	1.58
40.147	20.4	10.7	40.5	51.2	23.8	2.51	2.15
00.147	16.3	8.4	40.5	48.9	22.9	3.00	2.14

Note: 1 kip = 4.45 kN Cols. 2 and 5 indicate the shears corresponding to calculations based on ACI 318-71 with ϕ = 1.0.

$- \Delta$ effect, a conservative assumption for determining the amount of web reinforcement.

The following specific assumptions of ACI 318-71 were used to calculate the flexural capacity of the specimens: (1) Linear strain distribution through the section; (2) limiting concrete compressive strain of 0.003; (3) strength of concrete in beam is 85% of the mean cylinder strength; (4) no strain hardening in the reinforcing steel (an unconservative assumption when designing shear reinforcement); and (5) no tensile strength for the concrete.

To proportion the web reinforcement, it was assumed that

$$V_u = V_c + V_s \quad (2)$$

in which V_u = shear capacity of a member; V_c = shear assigned to the concrete; and V_s = shear assigned to web reinforcement. The following expression (ACI 318-71) was used to calculate the contribution of the concrete to the shear strength of a member:

$$V_c = 2.0\left(1 + 0.0005\frac{N_u}{A_g}\right) b\,d\sqrt{f'_c} \quad \text{. (3)}$$

in which b = width of the cross section; d = depth to the tensile reinforcement; N_u = axial load on the member corresponding to V_u; A_g = gross cross-sectional area; and f'_c = concrete compressive strength. (Note that $\sqrt{\text{psi}}$ = 0.265 $\sqrt{\text{kg/cm}^2}$ in Eq. 3.) The contribution of transverse reinforcement to shear strength was taken as

$$V_s = A_v f_{ys}\frac{d}{s} \quad \text{. . . . . (4)}$$

in which A_v = cross-sectional area of the stirrup legs; f_{ys} = the yield stress of the transverse reinforcement; and s = spacing between stirrups.

Nominal dimensions and mean values for concrete compressive strength were used to calculate the values of V_c, V_s, and V_u given in Cols. 3, 4, and 5 of Table 2. Col. 6 of Table 2 lists the maximum measured shears, V_m, and Cols. 7 and 8 compare the calculated shear capacity of the specimens, V_u, with the ultimate shear based on calculated flexural strength, V_{uf}, and the maximum measured shear, V_m.

Test Results

General.—Representative behavior of the test columns, with the transverse reinforcement designed on the basis of the concept that part of the shear is assigned to the concrete, is shown by the shear versus deflection curves in Figs. 4(*a*) and 4(*b*). Although the test specimens would develop the expected yield moment in the first quarter cycle and maintain that load for some inelastic deflection, the repetition of these deflections resulted in a decay in the stiffness and strength of the member. The phenomenon of decay in shear strength as a result of loading into the inelastic range and cycling the inelastic deflections is related to a change in the shear carrying mechanism of the member.

Shear Carrying Mechanism.—The first cracks to appear in the specimens, as the applied deflection was continuously increased from zero, were short vertical flexure cracks in the cantilevered portions of the specimens. Continued increases in the deflection (and load) led to the formation of inclined cracks which extended out from the vertical cracks to form flexure-shear cracks [shown ideally in Fig. 5(*a*)]. After the formation of flexure-shear cracks, the shear carrying mechanism was assumed to consist of contributions from the compressed concrete above the crack, stirrups crossed by the inclined crack, aggregate interlock or friction forces along the crack, and dowel forces from the tension reinforcement [Fig. 5(*b*)]. The idealized (in linear segments) curves shown in Figs. 6 and 7 indicate how the assumed shear carrying mechanism shown in Fig. 5(*b*) changed as the test progressed.

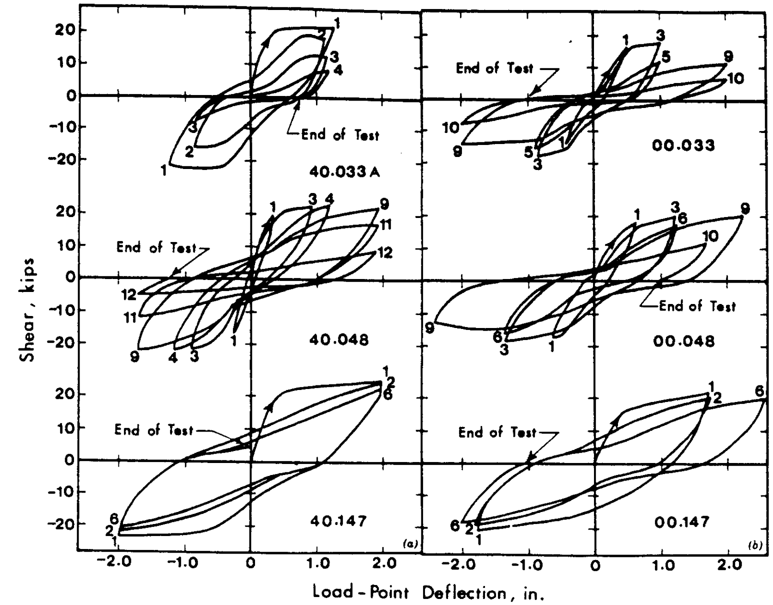


FIG. 4.—Measured Shear Versus Deflection Relationships: (*a*) for Specimens with 40-kip (180-kN) Axial Load; (*b*) for Specimens with No Axial Load (1 kip = 4.45 kN)

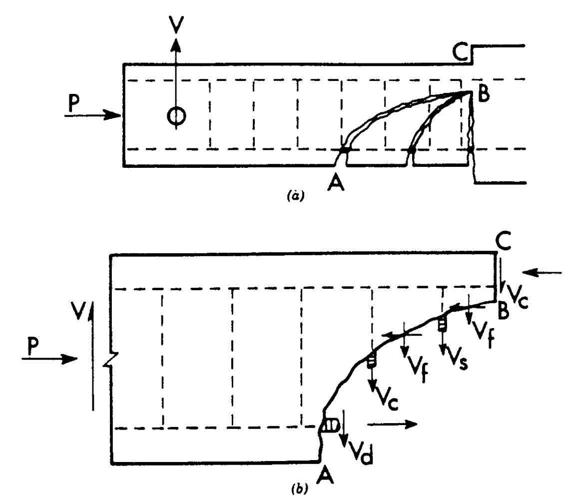


FIG. 5.—(*a*) Idealized Crack Pattern for Test Specimens during First Loading; (*b*) Assumed Shear Carrying Mechanism of Test Specimens

The axes for Fig. 6 represent the applied shear and the deflection, normalized on the basis of the yield values, for a test specimen for the first quarter cycle. Curve 1 represents, in idealized linear segments, the routine load versus deflection curve for monotonic loading of a specimen to four times the yield deflection.

Curve 2, also in idealized linear segments, represents the portion of the shear carried by the transverse reinforcement. It is based on measurements of strain in the stirrups (transverse reinforcement) crossing the critical inclined crack

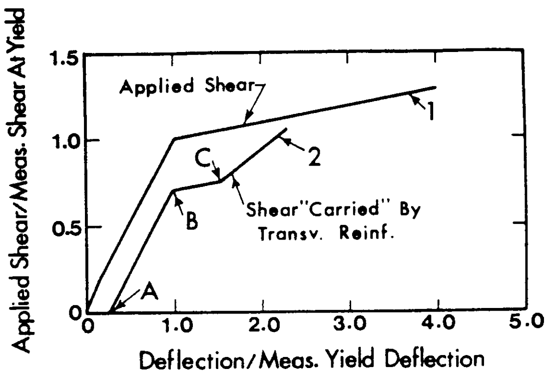


FIG. 6.—Shear Carried by Transverse Reinforcement Compared with Total Applied Shear

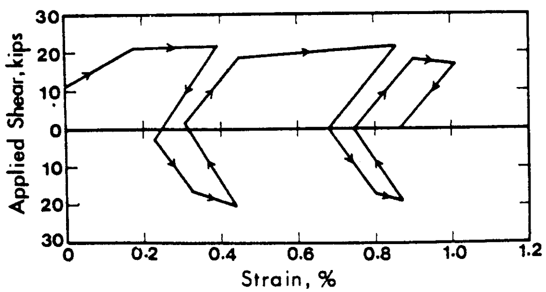


FIG. 7.—Applied Shear Versus Measured Strain in Stirrup (Transverse Reinforcement)

near the joint. The variations in the proportion of the total shear carried by the stirrups give a clue to the decay in the shear strength of the test specimen.

Virtually no shear is carried by the transverse reinforcement up to point A on curve 2 which must refer to the development of the inclined crack. After this point, the stirrups pick up shear at a steady rate up to point B where the longitudinal reinforcement yields and the rate of shear increase with deflection becomes less. Beyond point B there is some increase in shear due primarily to the strain hardening in the longitudinal reinforcement. Therefore, the stirrups

continue picking up shear. At point C the compressed concrete in the extreme fiber in compression starts exhibiting longitudinal cracks corresponding to compressive distress. As the concrete loses its capacity to carry compression, it must also lose its capacity to carry shear. Therefore, the "shear carried by the concrete" starts shifting to the transverse reinforcement which picks it up at a rate and to a magnitude which is a function of the amount and stiffness of the transverse reinforcement in relation to the stiffness of other elements which can carry the shear, including the doweling capacity of the longitudinal reinforcement. During this process, the transverse reinforcement (designed to carry less than the total shear) is likely to yield. Yielding of the reinforcement sets the stage for deterioration of the shear carrying capacity of the concrete shown in Fig. 7.

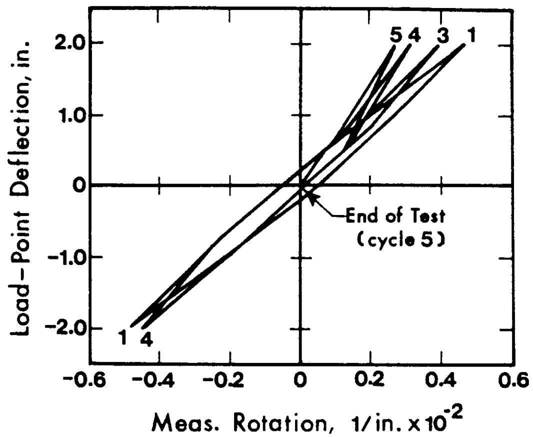
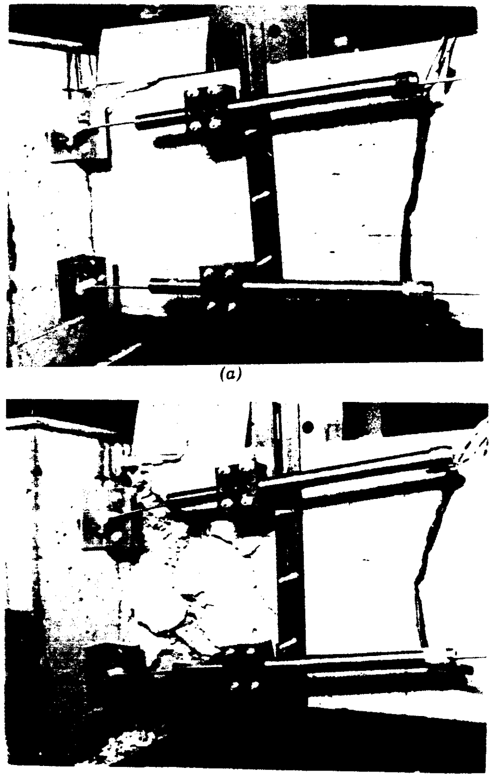


FIG. 8.—Relationship Between Measured Rotation and Load-Point Deflection, Specimen 00.105 (1 in. = 25.4 mm)

Fig. 7 shows ideally the applied shear versus the stirrup strain in a particular specimen. Again, the stirrup strain is not perceptible until inclined cracking occurs. After inclined cracking, the stirrup strain increases with applied shear, this increase becoming much faster after yielding of the stirrup. Thus, when the load is taken off, there is a permanent lengthening of the stirrup. When the load is repeated in the other direction, the stirrup yields again, resulting in a further increase in permanent strain. As this process is repeated, the concrete section, which must ultimately provide the compressive thrust, becomes distorted. As a result, the shear strength decays.

The change in resistance mechanism of the specimens could also be sensed from measured rotation-deflection relationships (Fig. 8). The horizontal axis represents the rotation measured over a 10-in. (250-mm) gage length starting at the face of the central joint. It is seen that during cycle 3 a trend is initiated such that less rotation is required for a given deflection, indicating an increase in the contribution of shear deformations (not reflected in the rotation measurements) to the total deflection. Fig. 9 confirms this interpretation.

Strength Decay.—The rate of decay was a function of axial load, percentage of transverse reinforcement, and total deflection. It has been observed in the course of these tests that the decay in shear strength is less in elements with higher axial loads, everything else being equal. This can be attributed to the fact that, in specimens with large axial loads, part of the compressed concrete is already confined by the stirrups and therefore does not start losing strength



(a)

(b)

FIG. 9.—Hinging Zone for Specimen 00.105: (*a*) During First Cycle; (*b*) During Fourth Cycle

early in the process of loading and the presence of the constant axial load makes it possible to mobilize friction forces.

A comparison among either the three sets of curves in Fig. 4(*a*) for specimens with a 40-kip (180-kN) axial load or the three sets of curves in Fig. 4(*b*) for specimens without an axial load indicates that the rate of strength and stiffness decay decreased as the amount of transverse reinforcement was increased.

The ratio of maximum deflection per cycle to the yield deflection also had a significant effect on the reduction in strength and stiffness with cycling, as

shown by the second set of curves in Fig. 4(*a*). At a deflection ductility of two, there was no indication of a loss in strength. However, at the deflection ductility of four, there was a significant reduction in strength during each complete cycle. A summary of the experimental investigation is given in Table 3.

Recommendations for Design

The experimental work and its interpretation reported herein indicate that current concepts for the design of web reinforcement are adequate for monotonic loading up to the spalling of concrete under flexural compression. However, for repeated loading beyond that level, the amount of transverse reinforcement

TABLE 3.—Summary of Experimental Results

Mark (1)	Deflection schedule[a] (2)	Cycles before Failure: At two times yield deflection (3)	Cycles before Failure: At four times yield deflection (4)	Mode of failure (5)
40.033A	A	0	1	Shear
40.033	B	6	1	Shear
25.033	B	6	0	Shear
00.033	B	6	1	Shear
40.048	B	6	2	Shear
00.048	B	6	1	Shear
40.067	B	6	2	Shear
00.067	B	6	3	Shear
40.092	A	0	4	Shear
00.105	A	0	3	Shear
40.147[b]	A	0	6	—
00.147[b]	A	0	6	—

[a]See Fig. 2.

[b]These specimens did not exhibit any significant strength decay, so testing was stopped after six cycles.

required by ACI 318-71 was not adequate to insure a flexural failure in the test specimens.

Therefore, it is recommended that if reinforced concrete elements are designed to resist earthquake effects by energy dissipation in the inelastic range, the transverse reinforcement must be designed to carry the entire shear.

It should also be emphasized that the maximum shear on a reinforced concrete element is likely to be more than that corresponding to the development of the yield stress in the flexural reinforcement because of strain hardening.

Although the spacing of the reinforcement was not specifically investigated in this study, the behavior of the specimens suggested strongly that the spacing of the stirrups should not exceed one-fourth of the effective depth.

However, the use of closely spaced stirrups that are designed to carry all of the shear acting on the member does not necessarily prevent shear failures in reinforced concrete members when they are subjected to large load reversals. This point must be emphasized because all of the loads applied to a reinforced concrete member are ultimately carried by the concrete and if the concrete does not stay intact, the strength of the reinforcement cannot be developed. Therefore, it is impractical to design stirrups to carry all of the shear if the concrete is not effectively confined. The results of this investigation and a comparison of the results from investigations conducted in Japan (2,3,5) with those from investigations conducted in the United States (4,6) indicate that the problem is more severe in members with no axial load.

Summary and Conclusions

Twelve reinforced concrete column specimens were subjected to a series of load reversals to deflections beyond yield to study the shear strength decay of reinforced concrete columns under such loading conditions. On the basis of the results presented herein, the following conclusions can be made:

1. There was a discernible change in the shear mechanism of a reinforced concrete member subjected to deflections corresponding to the onset of spalling cracks in the compressed concrete. The primary reason for this change was a transfer of shear from the compressed concrete to the stirrups.
2. There is a progressive decrease in strength and stiffness of a reinforced concrete member with cycling into the inelastic range unless enough transverse reinforcement is provided to confine the core and to carry the total shear.
3. The presence of an axial compressive load slowed the decay in strength and stiffness with cycling.

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Appendix II.—Notation

The following symbols are used in this paper:

A_g = gross area of cross section;
A_v = cross-sectional areas of stirrup legs;
a = shear span;
b = width of cross section;
d = distance from extreme compressive fiber to centroid of tension reinforcement;
f'_c = compressive strength of concrete;
f_{ys} = yield stress of transverse reinforcement;
M_u = ultimate moment capacity of section;
N_u = axial load normal to cross section occurring simultaneously with V_u;
P = axial load;
s = stirrup spacing;
V_c = shear assigned to concrete;
V_m = maximum measured shear;
V_s = shear assigned to transverse reinforcement;
V_u = shear strength of member;
V_{uf} = shear required to develop ultimate moment capacity;
Δ_y = yield deflection; and
ϕ = capacity reduction factor.