SHEAR FAILURE OF REINFORCED CONCRETE COLUMNS SUBJECTED TO CYCLIC LOADING

by

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SYNOPSIS

Results of tests on twelve column specimens subjected to loading reversals into the inelastic range are reported. A description and explanation is given for the progressive shear failure observed in many of the test specimens. The principal variables were: (a) the axial load which varied from zero to fifty percent of balance load, (b) the percentage of transverse reinforcement which varied from 0.33 to 1.47, and (c) the required displacement ductility which varied from one to four.

EXPERIMENTAL INVESTIGATION

This investigation, which was initiated by Professor Sozen and the writers, was conducted in the Civil Engineering Department of the University of Illinois under National Science Grant GI 30760. The experimental setup is shown in Fig. 1. The specimens were held at the joint with the ends moved simultaneously in opposite directions to create a moment reversal through the joint. Displacements were controlled during the test and followed one of the procedures shown in Fig. 2. The shear span ratio was 3.5 and closed ties made of plain No. 2 bars or deformed No. 3 bars were used throughout. The weight of the ram attached to the specimen is 0.4 kips and was added to (or subtracted from) the measured shear. Table 1 gives the values of the test variables and a brief description of the test results.

The test results indicate that the generally accepted formula for calculating the ultimate shear strength of reinforced concrete members

$$V_u = V_c + V_s \qquad (1)$$

is not always valid for inelastic loading reversals. Table 2 shows that for most of the specimens the maximum shear measured during the tests was less than the ultimate shear capacity computed by Eq. 1. However, only two specimens clearly survived the testing program.

The presence of a progressive shear failure is quite apparent from the shear vs. load-point displacement curves recorded during the tests. Figure 3 for specimen 40.048 is typical. During the cycles with a displacement ductility value of about two there is no reduction in shear capacity and only a minor reduction in stiffness. However, when the ductility per cycle changes to four there is a rapid reduction in shear strength and stiffness. This type of failure can be explained by combining geometric compatibility with strength calculations.

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FAILURE HYPOTHESIS

The process of a progressive shear failure starts when the load-point displacement is large enough to produce a strain in the extreme concrete compression fiber that exceeds some critical value (≈ 0.0035). Beyond this point the shear carrying capacity of the shell concrete starts to decrease. Thus, the stirrups and the concrete within the core must carry more shear. If this shear can be absorbed without the stirrups yielding, the specimen can undergo several loading reversals without suffering a noticeable drop in shear capacity. The second critical point occurs when the load-point displacement is large enough to produce compressive strains in the shell concrete that are larger than the limiting strain for unconfined concrete (≈ 0.006). These large strains induced splitting and spalling of the shell concrete and therefore any shear that was being carried by the shell concrete must be absorbed by the stirrups and the core concrete. Also, after the shell has spalled off the shear resisting mechanism of the specimen has changed from the gross section to the core only and, as shown in Table 2, most of the specimens were unable to absorb this shear without the stirrups yielding.

The sequence of events described above is reflected in Fig. 4 which shows the strain recorded in a stirrup 4.5 in. from the joint vs. the load-point displacement for specimen 00.105. Points A, B, C, and D correspond to shear cracking, yield, the start of decrease in concrete shear capacity, and spalling of the concrete shell respectively.

SUMMARY

The test results indicate that the following items should be considered in the design of reinforced concrete members to resist seismic loading. (a) The expected number of inelastic cycles and ductility per cycle must be considered instead of using strength requirements alone. (b) It is conservative to base the shear strength on the shear capacity of the core only. (c) The spacing of the stirrups within a hinging zone must be small enough to ensure confinement of the core. (d) The contribution of the concrete to the shear capacity of the core must be reduced for all members subjected to shear reversals and should be ignored for a member with no axial load.

REFERENCES

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TABLE 1 SUMMARY OF THE EXPERIMENTAL PROGRAM

Mark	Loading Type	Axial Load (kips)	CONCRETE Comp. Strength f'_c (psi)	Splitting Strength f'_{sp} (psi)	WEB REINFORCEMENT Bar Size No.	Spacing s (in.)	Reinf. Ratio	Cycles to Failure	Mode of Failure
40.033a	B	42.5	5030	391	2	5.0	.0033	1	Shear
00.033	A	0	4640	359	2	5.0	.0033	4	Shear
40.048	A	40	3780	320	2	3.5	.0048	9	Shear
00.048	A	0	3750	356	2	3.5	.0048	3	Shear
40.033	A	40	4870	378	2	5.0	.0033	9	Shear
25.033	A	25	4880	400	2	5.0	.0033	3	Shear
40.067	A	40	4840	398	2	2.5	.0067	10	Shear*
00.067	A	0	4610	379	2	2.5	.0067	10	Shear
40.147	B	40	4860	423	3	2.5	.0147	-	-
00.147	B	0	4900	414	3	2.5	.0147	-	-
40.092	B	40	5150	438	3	4.0	.0092	4	Shear*
00.105	B	0	4850	419	3	3.5	.0105	3	Shear

*Specimens not tested to destruction

TABLE 2 COMPARISON OF MEASURED AND CALCULATED SHEAR CAPACITIES

Mark	Meas. Ult. Shear V_m (kips)	GROSS SECTION Shear Cap. of Concrete V_c (kips)	Shear Cap. of Stirrups V_s (kips)	$\frac{\text{Calc.}}{\text{Meas.}}$ $\frac{V_u}{V_m}$	CORE ONLY Shear Cap. of Concrete V_c (kips)	Shear Cap. of Stirrups V_s (kips)	$\frac{\text{Calc.}}{\text{Meas.}}$ $\frac{V_u}{V_m}$
40.033a	22.5	11.1	10.0	0.94	6.4	8.0	0.64
00.033	18.2	8.3	10.0	1.01	3.3	8.0	0.62
40.048	22.5	9.6	14.2	1.06	5.4	11.4	0.75
00.048	19.5	7.6	14.2	1.12	2.9	11.4	0.73
40.033	22.8	11.0	10.0	0.92	6.2	8.0	0.62
25.033	20.5	10.0	10.0	0.98	5.1	8.0	0.64
40.067	22.4	10.8	19.9	1.37	6.1	16.0	0.99
00.067	20.5	8.4	19.9	1.38	3.3	16.0	0.95
40.147	26.4	10.7	40.5	1.94	6.1	32.4	1.46
00.147	22.9	8.5	40.5	2.14	3.4	32.4	1.55
40.092	25.3	10.7	25.3	1.42	6.0	20.2	1.04
00 .105	22.9	8.4	28.9	1.63	3.3	23.2	1.16

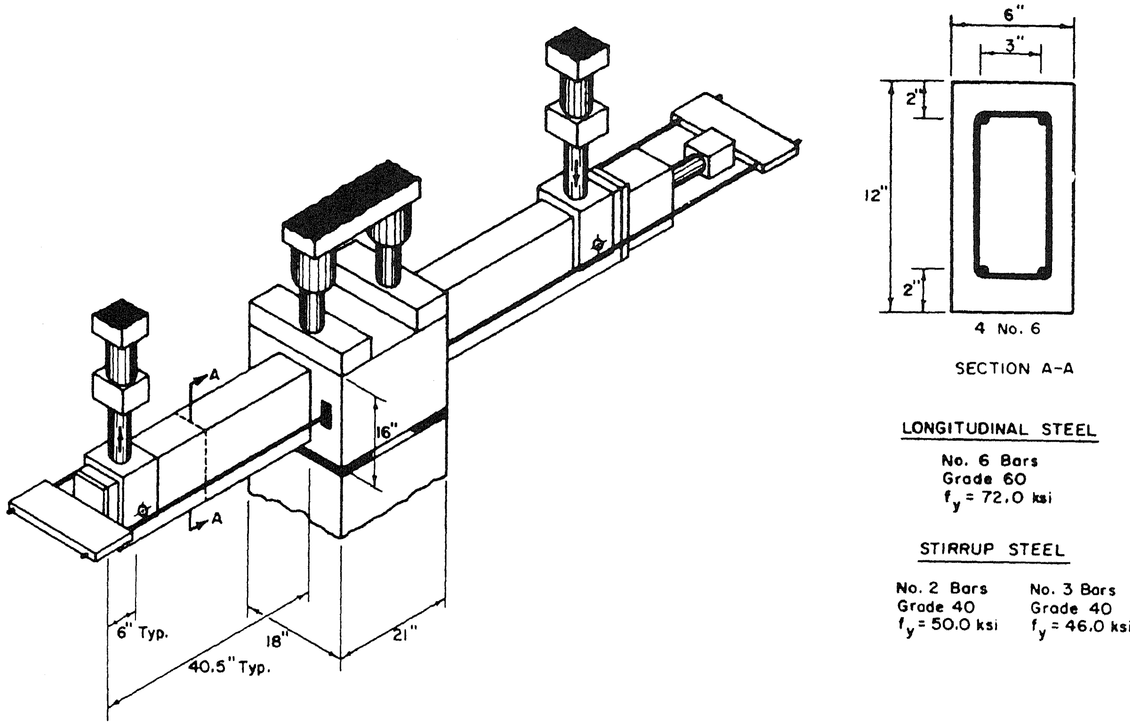
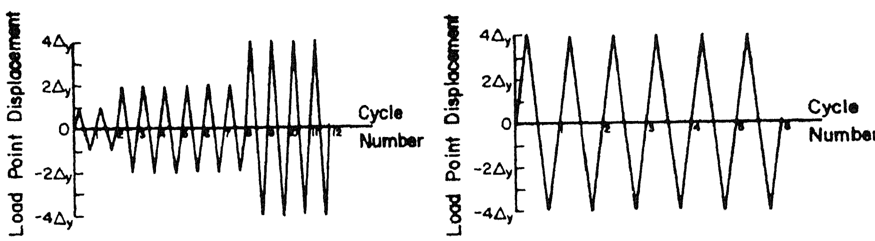


FIG. 1 TEST SETUP AND SPECIMEN DIMENSIONS



Loading Type A

Loading Type B

FIG. 2 LOADING PROCEDURES USED IN THIS EXPERIMENTAL PROGRAM

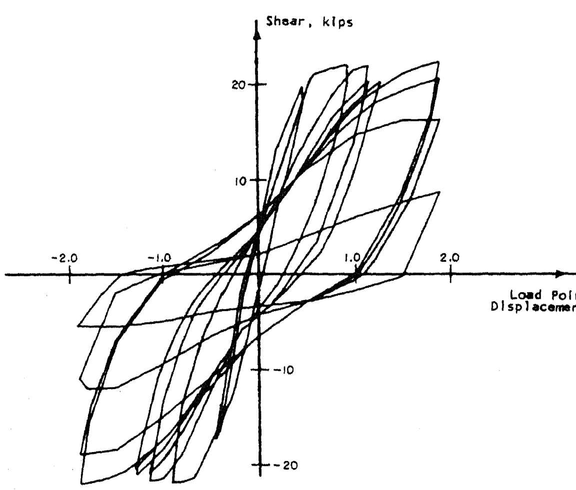


FIG. 3 SHEAR vs. DISPLACEMENT, SPECIMEN 40.048

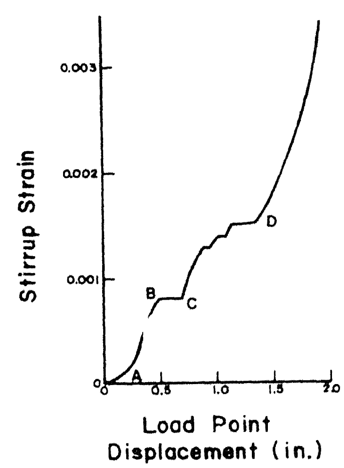


FIG. 4 STRAIN vs. DISPLACEMENT, SPECIMEN 00.105