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Seismic Shear Response of Structural Concrete Elements

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ABSTRACT

Presents results from 20 tests of structural concrete beam and column elements subjected to reversed cyclic shear. The parameters investigated include the maximum shear span (maximum bending moment to shear force ratio), the quantity of transverse reinforcement, the distribution of longitudinal reinforcement, and the magnitude of axial load.

INTRODUCTION

Traditional shear design rules assume that the shear resistance of a structural concrete member is equal to the sum of a stirrup contribution V_s, which is calculated using a 45 degree truss model, and a concrete contribution V_c, which is taken as the shear to cause initial diagonal cracking. In the seismic shear design of new structures, the concrete contribution is usually assumed to be zero in regions of the structure which need to develop considerable ductility. Experimental results have shown this approach is conservative, however, the additional cost (to add more stirrups during construction) is negligible.

In the seismic evaluation of existing structures, the conservative approach of neglecting the concrete contribution may mean that an expensive retrofit is needed. For example, many concrete bridges in British Columbia were constructed more than thirty years ago and have very little transverse reinforcement in the columns and pier-cap beams. If the concrete contribution of these members is totally neglected, most of these members will need to be strengthened. Thus a less conservative empirical approach, in which the concrete contribution depends on the specific ductility demand, is often used in evaluating existing structures.

In order to better understand the degradation of shear strength with increased ductility demand, 20 reinforced concrete specimens were tested using a specially developed element tester at the University of British Columbia. This paper summarizes some of the important results from these tests.

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EXPERIMENTAL PROGRAM

Element Tester

An element tester was developed by Adebar (1994) to apply axial load, bending moment and shear to short segments (elements) of structural concrete beams (or columns). The sectional forces are applied to the elements using yokes bolted to loading plates which are cast into the specimens (see Fig. 1). Three 1000 kN hydraulic actuators provide the active loads, while three rigid links provide the required reactions. Two loading yokes (one at each end) are used to transmit the loads from the actuators and rigid links to the structural concrete specimens.

To apply transverse shear, the two actuators at one end of the specimen (i.e., at the top of Fig. 1) are made to apply a bending moment which is not equal and opposite to the bending moment produced by the actuator and rigid link at the other end (bottom of Fig. 1). Moment equilibrium is not satisfied by the longitudinal forces so the transverse rigid links must provide the equilibrating force couple. Note that the force in the transverse rigid links is equal to the shear force applied to the specimen. Almost any combination of maximum bending moment and transverse shear can be applied to the elements by adjusting the relative bending moments applied at the two ends of the specimen. The maximum transverse shear results when the actuators apply equal moments (in magnitude and direction) at the two ends of the specimen.

The connection detail that has been developed for specimens subjected to reverse cyclic shear is shown in Fig. 2. While in monotonic shear tests the critical section is at approximately d from the specimen—tester interface, in reverse cyclic tests the important plastic hinge region initiates at the location of maximum moment. Thus the "test region" is not connected directly to the loading yoke but is attached via a structural concrete "interface," which forces the plastic hinge away from the yoke and allows for bond slip of the longitudinal steel. As shown in Fig. 2, the longitudinal reinforcement is welded directly to the loading plate and shear studs are used to transfer the applied shear force uniformly over the depth of elements thereby reducing the compression strut (arch) action.

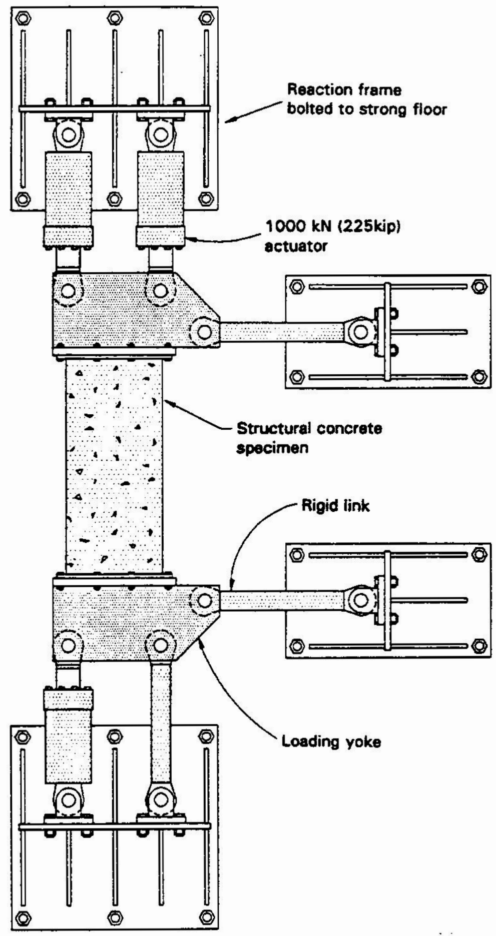

Fig. 1 — Plan view of element tester.

The first series of tests conducted using the element tester involved four identical deep beams (152 mm wide × 610 mm deep) without stirrups subjected to monotonic shear using different maximum bending moment to shear force ratios (i.e., different shear spans, $a=M_{max}/V$). The beam elements were identical to a series of deep beams tested by Kani (1967) using the traditional loading arrangement in which a simply supported beam is loaded by two concentrated loads. For shear span to depth ratios *(a/d)* larger than 2.5, the two different testing methods gave very similar results, while for *a/d* less than about 2.5, the two testing methods gave very different results (see Fig. 3). In the traditional loading arrangement used by Kani, a direct compression strut (tied-arch) can form more easily when *a/d* is reduced below 2.5 since the inclination of the compression strut is steeper. In the element tester *a/d* does not effect the inclination of the potential compression strut, and as a result does not significantly effect the shear strength. Further details of the element tester are given by Adebar (1994).

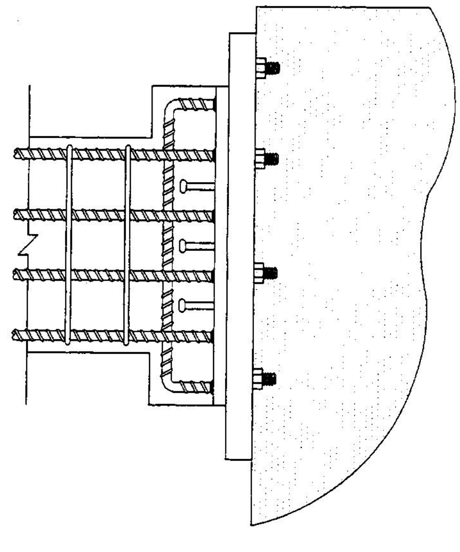

Fig. 2 — Specimen boundary conditions.

Test Specimens

In order to investigate the behaviour of concrete bridge components (beams and columns) subjected to reverse cyclic shear, 20 specimens have been tested (see Table 1). The 20 specimens can be grouped into three main series. The CS series of eight specimens (CS1 to CS8) were meant to represent approximately full-scale elements of a small bridge column, while the first eight specimens of the SR series (SR1 to SR8) were similar size column elements, but were considered to be a smaller scale version of a larger column. All sixteen specimens had cross-sectional dimensions of 400 mm × 400 mm, however the concrete cover was smaller in the SR series (20 mm versus 40 mm) and the transverse reinforcement consisted of more closely spaced smaller bars in the SR series (see Table 1). The third series of four specimens (SR9 to SR12) were meant to represent elements of a typical pier-cap beam and had dimensions of 300 mm × 500 mm. No transverse reinforcement was provided in the four beam element specimens. All 16 column elements and all 4 beam elements were 1.5 metres long.

The CS series specimens typically contained 5 — No. 15M bars (f_y = 485 MPa) near each flexural face, except that CS8 also contained 4 — No. 10 (f_y = 440 MPa) distributed over both side faces. The transverse reinforcement in the CS series had a yield strength (f_y) of 440 MPa. Specimens SR1 to SR8 contained 5 — No. 15M bars (f_y = 482 MPa) on the top and bottom faces, plus 2 — No. 15M on each side face.

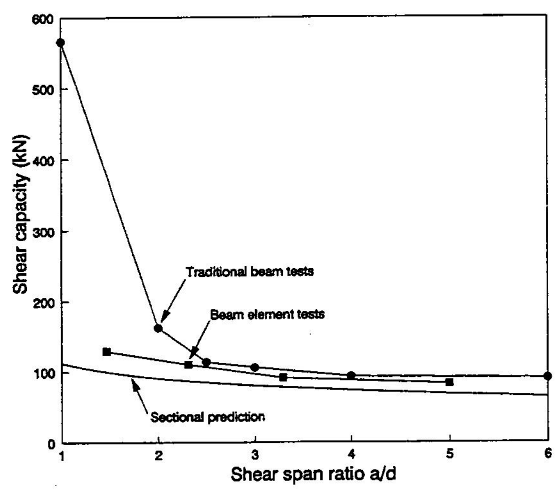

Fig. 3 — Influence of shear span on beam tests.

Table 1 — Summary of experimental program.

Specimen	N/V	M_r (kNm)	$(M/V)_{max}$ (mm)	A_v (mm^2)	s (mm)
CS1	0	192	686	2 × 100	305
CS2	0	192	1600	2 × 100	305
CS3	0	192	1029	2 × 100	305
CS4	-1.86	256	1029	2 × 100	305
CS5	+1.64	154	1029	2 × 100	305
CS6	+1.10	154	686	2 × 100	305
CS7	0	163	1029	2 × 100	305
CS8	0	217	1029	2 × 100	305
SR1	0	235	1100	3 × 33	76
SR2	0	235	900	3 × 33	76
SR3	0	235	800	3 × 33	76
SR4	0	235	725	3 × 33	76
SR5	0	235	900	3 × 33	51
SR6	0	235	900	3 × 33	102
SR7	-1.0/+1.0	309/220	900	3 × 33	76
SR8	-1.5/+1.5	336/204	900	3 × 33	76
SR9	0	189	1620	0	-
SR10	0	189	1335	0	-
SR11	0	189	1900	0	-
SR12	0	189	1900	0	-

The transverse reinforcement consisted of 1/4 inch deformed wire with a yield strength of approximately 250 MPa. Specimens SR9 to SR11 contained 3 — No. 15M (f_y = 460 MPa) bars plus 2 — No. 10M (f_y = 450 MPa) on the top and bottom faces and 2 — No. 10M on both side faces, while specimen SR12 contained 5 — No. 15M on the top and bottom faces without any reinforcement on the side faces. All concrete used to construct the specimens had a strength of about 30 MPa.

In addition to the differences between the various groups of specimens mentioned above, the variables considered in the study include: the maximum shear span (i.e., the ratio of the maximum applied moment to the applied shear force); the corresponding axial load (compression "-", and tension

"+"); the quantity and distribution of longitudinal reinforcement; and the quantity and spacing of stirrups (see Table 1). Further details of the CS series are given by Webster (1995), while further details of the SR series are given by Roux (1995).

DISCUSSION OF RESULTS

The CS series specimens were the first elements to be subjected to reverse cyclic shear using the element tester, therefore these specimens served as a pilot series. The specimen boundary conditions were adjusted a couple of times during the testing of the CS series, and from the experience gained with the CS series, the connection detail shown in Fig. 2 was finally developed and used in all SR series specimens.

As mentioned above, the CS series specimens were considered full-scale elements of a small bridge column. In many existing bridge columns in British Columbia, the hoops (transverse reinforcement) consist of No. 3 reinforcing bars at 12 inches. Thus the transverse reinforcement in the CS series were spaced at 305 mm (12 inches). As the overall dimension of the column elements was only 400 mm, in some cases a diagonal crack formed between the stirrups. Also, a clear concrete cover of 40 mm was used in these "full-size" specimens. In a number of tests, the cover spalled and the compression reinforcement buckled, significantly reducing the flexural ductility of these elements.

While many sub-groups within the CS series showed an interesting trend (Webster, 1995), perhaps one of the most interesting comparisons is specimens CS1 and CS6, which were identical in all regards except that CS6 was subjected to an axial tension of 1.1 times the shear force and CS1 was not subjected to any axial load. Table 2 compares the shear capacity versus the shear demand for these two specimens. The axial tension reduced the flexural capacity from 192 kNm to 154 kNm. As a result, the shear demand (V_{dem}), which is equal to the flexural capacity divided by the maximum shear span (M/V), reduced from 280 kN to 225 kN. The shear capacity of the two specimens was predicted by the ACI Building Code (1989), as well as the modified compression field theory (Collins and Mitchell, 1991). These calculations ignore any degradation of the shear resistance due to ductility demand.

Table 2 — Influence of axial tension on capacity - demand ratios.

Spec	N/V A	M_r B (kNm)	$(M/V)_{max}$ C (mm)	V(dem) B/C (kN)	ACI Code V_i(cap) (kN)	ACI Code $\frac{V_i(cap)}{V(dem)}$	M.C.F.T. V_i(cap) (kN)	M.C.F.T. $\frac{V_i(cap)}{V(dem)}$
CS1	0	192	686	280	241	0.86	253	0.90
CS6	+1.10	154	686	225	191	0.85	246	1.10

* $N \approx +0.05 f_c'A_g$

The ratio of the shear capacity to the shear demand is a good indicator of the likelihood of a brittle shear failure. According to the ACI shear strength calculations, the shear capacity/demand ratio was a little smaller for CS6. That is, the axial tension decreased the shear resistance of CS6 a little more than the flexural resistance. According to the modified compression field theory, the reduction in shear strength less than the reduction in flexural resistance so that the capacity demand ratio increases considerably. Figure 4 shows the hysteresis loops for the two specimens. Specimen CS1 suffered a brittle shear failure during the first cycle of loading, while specimen CS6 was considerably more ductile.

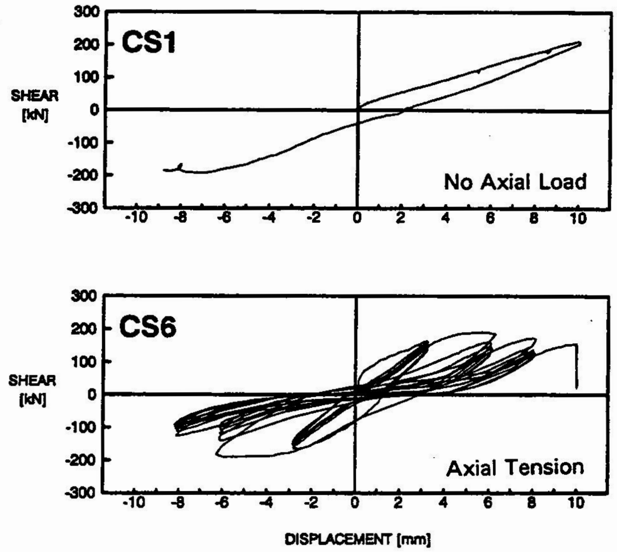

Fig. 4 — Measured response.

The first six specimens of the SR series (SR1 to SR6) were designed to investigate the influence of flexural ductility on the seismic shear response of elements with significant flexural ductilities (smaller cover as a percentage of the overall dimension and smaller hoop spacings). All six specimens had identical flexural reinforcement (i.e., identical flexural capacities), and SR1 to SR4 also had identical shear reinforcement (i.e., identical shear resistances). These four specimens were tested with varying maximum shear spans (see Table 1) which resulted in different shear demands. Figure 5 summarizes the different shear demands for the four specimens and shows the peak shear force at each cycle of loading for both directions.

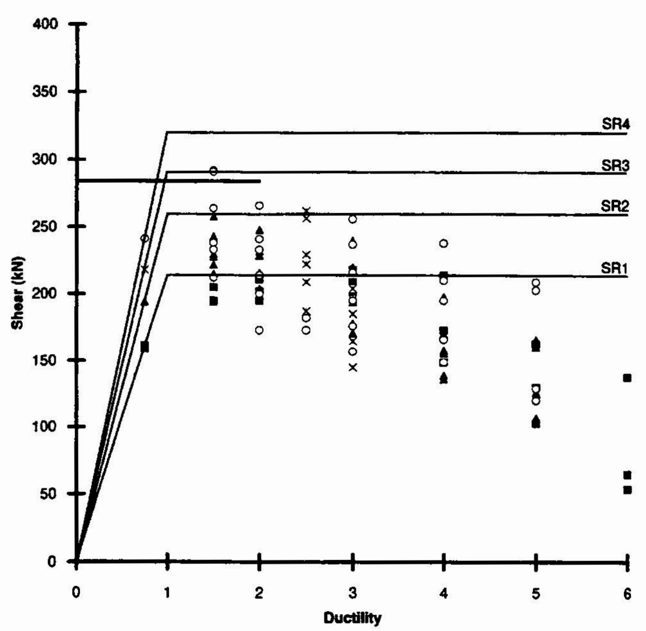

Fig. 5 — Peak shear force values at each load cycle.

Specimens SR2, SR5 and SR6 were all tested at the same shear span (i.e., same shear demand), but had different shear resistances resulting from the transverse reinforcement being spaced differently (see Table 1). Figure 6 shows the peak shear force at each load cycle for these specimens.

To better understand the shear strength degradation due to increase ductility demand, the deformations of the specimens were measured using a photographic technique. Target points were placed on the specimen in a 100 mm × 100 mm grid. At the peak of each load cycle, a photograph was taken of the specimen surface. The photographic images were enlarged and the distances between the target points were measured. Figure 7 shows the deformed shape of specimen SR5 at the first and third cycle during the loading at a displacement ductility of 5. The peak shear forces reached during these two cycles was 243 kN and 201 kN (i.e., the peak shear reduced by 17%). Figure 7 shows that during the two cycles of loading considerable dilation occurred in the plastic hinge region accompanied by an overall shortening of the specimen. These deformations are believed to be due to slippage along the diagonal shear cracks.

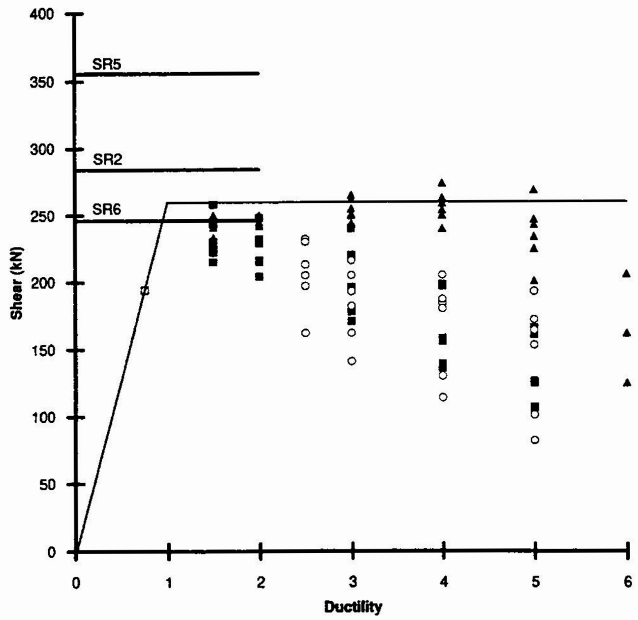

Fig. 6 — Peak shear force values at each load cycle.

CONCLUDING REMARKS

In general, the experimental results demonstrated that a good indicator of the available ductility is the ratio of maximum shear (at formation of a plastic hinge) to monotonic shear capacity; however, the shear capacity must be predicted with an appropriate shear design method. For example, traditional shear design rules predict that axial tension will reduce the ductility of a concrete element with little or no transverse reinforcement because it degrades the shear capacity (i.e., reduces V_c) more than it reduces the flexural capacity. The modified compression field theory predicts, and the experimental results confirm, that axial tension actually enhances the flexural ductility of structural concrete members with well distributed longitudinal reinforcement.

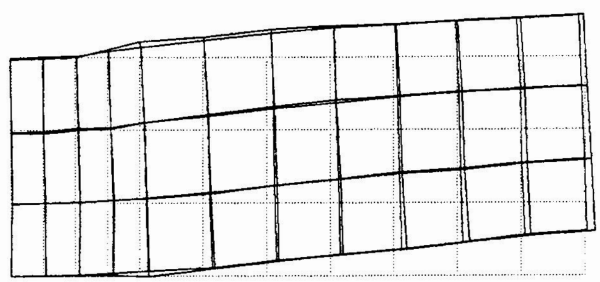

Fig. 7 — Deformations of SR5 at a displ. ductility of 5.

Another example of the importance of an appropriate shear design model is given by specimen SR11, which was more ductile than SR12. The two specimens had the same flexural capacity (and thus the same shear demand), but unlike SR11, SR12 did not contain any distributed longitudinal reinforcement on the side faces. While traditional shear design methods neglect the influence of longitudinal reinforcement, the modified compression field theory predicts that the distributed reinforcement provides better crack control and as a result better shear response.

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