Earthquake-Resistant Coupling Beams without Diagonal Reinforcement

Strain-hardening fiber-reinforced concrete provides means to simplify detailing

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For some time, there has been an inherent challenge in the design and construction of coupling beams in earthquake-resistant coupled wall systems. The failure of coupling beams with traditional longitudinal and transverse reinforcement during the 1964 Alaska earthquake indicated the need for new designs that would allow these beams to sustain large shear reversals without a substantial degradation of strength and stiffness. Experimental research conducted in New Zealand showed that the use of well-confined diagonal reinforcement cages—designed to resist the total shear demand—ensures stable behavior under displacement reversals such as those induced during a strong earthquake.[1] Current design provisions in the ACI 318-08 Building Code[2] are largely based on this research.

The reliance on diagonal reinforcement to resist the entire coupling beam shear demand, however, often translates into the use of large diameter diagonal bars—for example, No. 11 (No. 35) or larger—that require long development lengths and are difficult to handle on site. The construction of coupling beams is further complicated by the need for column-type transverse reinforcement to confine each diagonal reinforcement cage or the entire beam to maintain the integrity of the concrete and prevent premature buckling of the diagonal bars. Figure 1 shows a typical design of a coupling beam in an earthquake-prone region.

Fig. 1: Typical diagonally reinforced coupling beam *(photo courtesy of Rémy Lequesne)*

In recent years, the use of relatively slender coupling beams (that is, beams with span-to-overall-depth ratios [aspect ratios] on the order of 3) has become popular due to limitations in story heights. In these beams, however, the effectiveness of diagonal reinforcement to resist shear significantly decreases because of the shallow angle with the beam longitudinal axis (less than

20 degrees). For example, approximately twice the amount of diagonal reinforcement is needed to resist the same amount of shear when the angle between the diagonal reinforcement and the beam axis is reduced from 45 to 20 degrees. While the increase in beam aspect ratio allows flexure to play a more significant role compared to that in shorter coupling beams, results from recent research indicate that diagonal reinforcement, combined with column-type confinement, is still needed in the more slender coupling beams to ensure stable behavior under earthquake loading.[3]

It's possible that the addition of randomly oriented steel fibers to the concrete will allow significant simplifications in the design and construction of slender coupling beams, including the complete elimination of diagonal bars and substantial reductions in confinement reinforcement. This article presents the results from two large-scale tests that support that possibility.

EVALUATION OF NEW DESIGN FOR SLENDER COUPLING BEAMS

In the context of this article, the term "slender coupling beams" refers to those with span-to-overall-depth ratios of about 3. The proposed design builds on previous research conducted at the University of Michigan on the use of high-performance fiber-reinforced concrete (HPFRC) in short coupling beams (with a span-to-overall-depth ratio < 2).[4,5] Contrary to traditional fiber-reinforced concretes, HPFRC materials exhibit a strain-hardening behavior under direct tension and a compression behavior that resembles that of well-confined concrete; thus, they are able to provide shear resistance with reduced confinement reinforcement under large displacement reversals.

From previous research, it was concluded that the use of HPFRC allows a substantial reduction in both diagonal and confinement reinforcement in short coupling beams. It was also found that a shear stress of $5\sqrt{f_c'}$ psi ($0.42\sqrt{f_c'}$ MPa) represented a lower bound for the contribution of fiber-reinforced concrete to member shear strength, where f_c' is the concrete compressive strength.

Because of the increased role played by flexure, the use of HPFRC was believed to offer the potential for a complete elimination of diagonal reinforcement in slender coupling beams. To evaluate this possibility, two large-scale coupling beams with aspect ratios of 3.3 and 2.75 were tested under simulated earthquake loading.

The test specimens consisted of a coupling beam connected to stiff reinforced concrete blocks intended to represent structural walls. The two test coupling beams were 66 in. (1676 mm) long and 6 in. (152 mm) wide. The overall depths of Specimens 1 and 2 were 20 and 24 in. (508 and 610 mm); their aspect ratios were 3.3 and 2.75, respectively. The design of the two coupling beams is shown in Fig. 2. Flexural reinforcement was provided

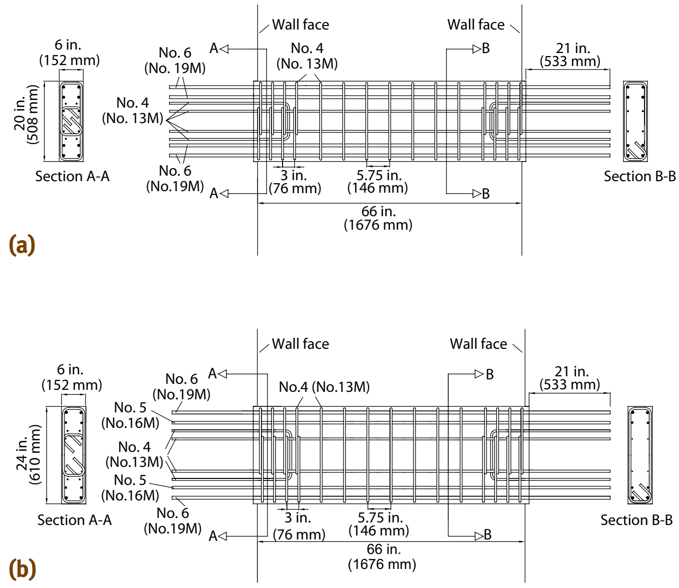

Fig. 2: Reinforcement details in test coupling beams: (a) Specimen 1 had an aspect ratio of 3.3; and (b) Specimen 2 had an aspect ratio of 2.75

such that the shear corresponding to the expected moment capacity being reached at both ends of the coupling beam would be close to the upper limit in ACI 318-08[2] ($10\sqrt{f_c'}$ psi [$0.83\sqrt{f_c'}$ MPa]). The expected beam flexural capacity was calculated considering overstrength and strain-hardening of the steel but neglecting the contribution of fiber-reinforced concrete in tension. Assuming that the HPFRC provides a shear resistance of $5\sqrt{f_c'}$ psi ($0.42\sqrt{f_c'}$ MPa), based on previous work,[5] transverse reinforcement was designed to resist the remaining shear demand. Because large inelastic rotations were expected at the beam ends, it was decided to provide column-type confinement over a distance of half the beam depth from the wall faces; this reinforcement was greater than that required for shear strength purposes. The transverse reinforcement outside of the plastic hinge region consisted of two-legged single stirrups.

We envisioned the use of a precast coupling beam design. This would allow the use of regular concrete in the structural walls and speed up the construction of the coupled wall system. Interference with wall reinforcement was prevented by extending the precast portion of the beam only into the wall cover. Moment transfer at the connection between the precast coupling beam and the cast-in-place walls was to be achieved by extending the flexural reinforcement a full development length beyond the precast portion. Also, to prevent excessive inelastic deformations at the precast coupling beam-wall interface, intermediate U-shaped reinforcement was provided, which extended approximately 8 in. (200 mm) into the span (Fig. 2).

Figure 3 is a schematic of the test setup. As shown in Fig. 3, the beam was rotated 90 degrees for testing convenience. The coupling beams were loaded to induce double curvature. For this purpose, lateral displacements were applied to the top block, which was prevented from rotating by two steel links. These links also provided some axial restraint to the coupling beams to simulate the restraint provided by structural walls in an actual coupled wall system. The specimens were subjected to lateral drift cycles of increasing magnitude until the beams exhibited substantial strength degradation.

MATERIALS

Based on previous research,[5] concrete reinforced with a 1.5% volume fraction of high-strength hooked-steel fibers was selected for use in the coupling beams. Concrete proportions by weight were as follows: 1.2 (Type III cement): 0.3 (fly ash): 0.6 (water): 1.7 (sand): 1.0 (coarse aggregate): 0.01 (high-range water-reducing admixture): 0.0095 (viscosity-modifying admixture). Coarse aggregate consisted of crushed limestone with a maximum size of 1/2 in. (13 mm). Commercially available hooked-steel fibers were used. These fibers were 1.2 in. (30 mm) long and 0.015 in. (0.38 mm) in diameter, made of a wire with a specified tensile strength of 330 ksi (2300 MPa). The concrete mixture was designed such that a highly workable composite with a compressive strength of about 10,000 psi (70 MPa) would be obtained. Results from 4 x 8 in. (100 x 200 mm) cylinder tests, as well as from ASTM 1609-06 four-point bending tests on beams with a 6 in. (150 mm) square cross section and 18 in. (450 mm) span, are shown in Table 1.

All reinforcing bars were Grade 60 (420) steel. The yield and ultimate strengths for the various reinforcing bars are shown in Table 2.

Table 1:
Compressive and flexural strength of fiber-reinforced concrete

Specimen number (aspect ratio)	f_c', ksi (MPa)	f_{p1},* psi (MPa)	$f_{150,\,0.75}$,* psi (MPa)	$f_{150,\,3.0}$,* psi (MPa)
1 (3.3)	9.9 (68)	1000 (6.9)	1080 (7.4)	650 (4.5)
2 (2.75)	9.8 (68)	1030 (7.1)	1280 (8.8)	980 (6.8)

*Obtained using a four-point bending test per ASTM 1609-06; f_{p1} is first peak flexural strength; $f_{150,\,0.75}$ is equivalent flexural strength at 0.03 in. (0.75 mm) deflection; $f_{150,\,3.0}$ is equivalent flexural strength at 0.12 in. (3.0 mm) deflection.

Table 2:
Properties of reinforcing bars

Specimen number (aspect ratio)	Bar size, No.	Yield strength f_y, ksi (MPa)	Tensile strength f_u, ksi (MPa)
1 (3.3)	6 (19)	79 (544)	100 (689)
	4 (13)	77 (531)	96 (661)
2 (2.75)	6 (19)	76 (524)	94 (648)
	5 (16)	64 (441)	97 (668)
	4 (13)	85 (586)	101 (696)

BEHAVIOR OF HPFRC COUPLING BEAMS WITHOUT DIAGONAL BARS

Figure 4 shows the average shear stress versus drift response for the two test coupling beams. Drift is

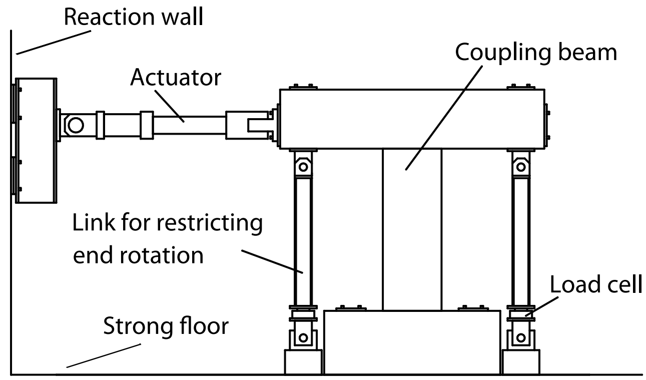

Fig. 3: Schematic of test setup. Each precast HPFRC coupling beam was anchored to two stiff concrete blocks. The links maintained the two blocks parallel during loading and simulated the axial restraint provided by coupled walls in an actual system

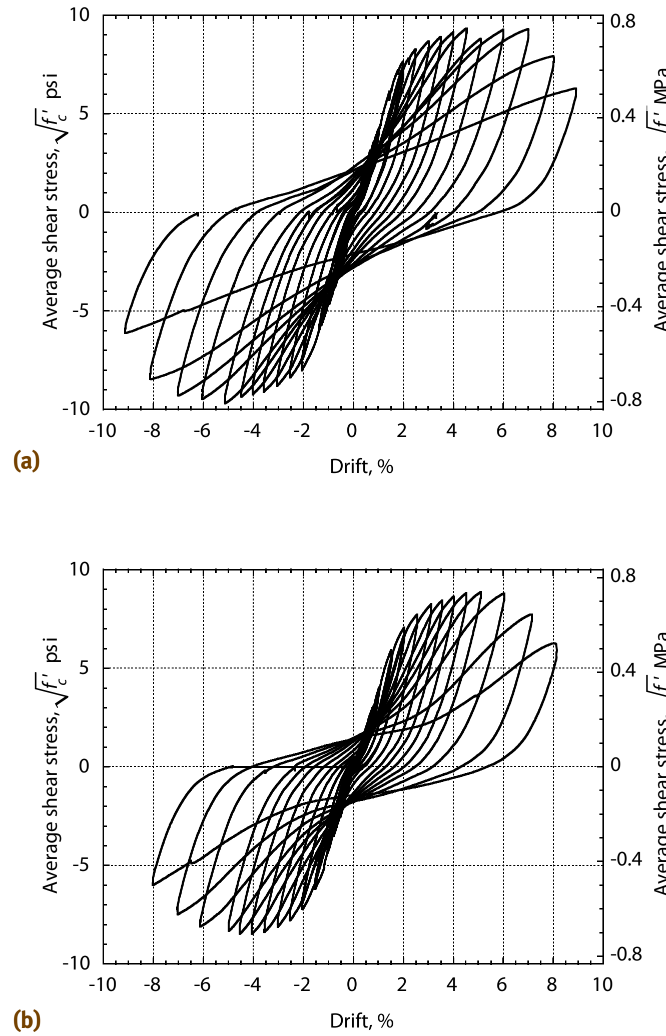

Fig. 4: Shear stress versus drift response for coupling beam specimens: (a) Specimen 1 (aspect ratio = 3.3); and (b) Specimen 2 (aspect ratio = 2.75)

defined as the applied lateral displacement divided by the clear beam span. As shown in Fig. 4, both specimens exhibited large drift capacities: Specimen 1 failed during the cycle at 9% drift and Specimen 2 failed during the cycle at 8% drift. The behavior of both specimens was governed by flexural hinging at both ends with negligible shear-related damage, even though the peak shear stress was close to $10\sqrt{f_c'}$ psi ($0.83\sqrt{f_c'}$ MPa).

As expected, the more slender coupling beam (Specimen 1) exhibited slightly wider hysteresis loops, indicating higher energy dissipation compared with Specimen 2. The achieved drift capacities under such high shear stress demand are a clear indication of the adequacy of the proposed design for use in regions of high seismicity.

In terms of damage tolerance, only minor damage was observed up to approximately 5% drift because of the tension and compression ductility exhibited by the HPFRC material (Fig. 5). This indicates that HPFRC coupling beams are substantially less likely to require repairs than regular concrete coupling beams after a major earthquake. Ultimately, failure occurred due to concrete crushing at the beam ends.

CONCLUSIONS

The experimental results support the potential use of strain-hardening HPFRC as a means to substantially simplify the reinforcement detailing in coupling beams with aspect ratios on the order of 3. In particular, the test results clearly indicate that a complete elimination of diagonal reinforcement is possible while achieving drift capacities as large as 8% under shear stresses close to the upper limit allowed in ACI 318-08. Furthermore, the results show that column-type confinement reinforcement is required only at the beam ends—the remainder of the beam can be reinforced with regular stirrup reinforcement. The HPFRC provided excellent damage

Fig. 5: Specimen 1 at 5% drift

tolerance with only minor damage observed at drifts as large as 5%. The proposed coupling beam design applies to both precast and cast-in-place construction, which provides greater flexibility for contractors.

Acknowledgments

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References

1. Paulay, T., and Binney, J.R., "Diagonally Reinforced Coupling Beams of Shear Walls," *Shear in Reinforced Concrete,* SP-42, V. 2, American Concrete Institute, Farmington Hills, MI, 1974, pp. 579-598.

2. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary," American Concrete Institute, Farmington Hills, MI, 2008, 473 pp.

3. Naish, D.; Wallace, J.W.; Fry, J.A.; and Klemencic, R., "Reinforced Concrete Link Beams: Alternative Details for Improved Construction," UCLA-SGEL *Report* 2009-06, Structural & Geotechnical Engineering Laboratory, University of California at Los Angeles, Los Angeles, LA, 2009, 103 pp.

4. Canbolat, B.A.; Parra-Montesinos, G.; and Wight, J.K., "Experimental Study on Seismic Behavior of High-Performance Fiber-Reinforced Cement Composite Coupling Beams," *ACI Structural Journal*, V. 102, No. 1, Jan.-Feb. 2005, pp. 159-166.

5. Lequesne, R.; Parra-Montesinos, G.; and Wight, J.K., "Test of a Coupled Wall with High-Performance Fiber-Reinforced Concrete Coupling Beams," *Thomas T.C. Hsu Symposium: Shear and Torsion in Concrete Structures*, SP-265CD, American Concrete Institute, Farmington Hills, MI, 2009, 18 pp.

Note: Additional information on the ASTM standard discussed in this article can be found at **www.astm.org**.

Selected for reader interest by the editors.

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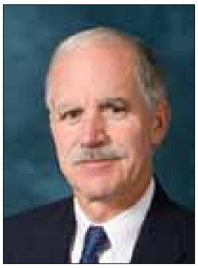

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