SP-265—1

Test of a Coupled Wall with High-Performance Fiber-Reinforced Concrete Coupling Beams

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Synopsis: Results from the test of a large-scale coupled-wall specimen consisting of two T-shaped reinforced concrete structural walls joined at four levels by precast coupling beams are presented. Each coupling beam had a span length-depth ratio (ℓ_n/h) of 1.7, and was designed to carry a shear stress of $7\sqrt{f_c'}$ [psi], ($0.59\sqrt{f_c'}$ [MPa]). One reinforced concrete coupling beam was included along with three strain-hardening, high-performance fiber-reinforced concrete (HPFRC) coupling beams to allow a comparison of their behavior. When subjected to reversing lateral displacements, the system behaved in a highly ductile manner characterized by excellent strength retention to drifts of 3% without appreciable pinching of the lateral load versus drift hysteresis loops. The reinforced concrete structural walls showed an excellent damage tolerance in response to peak average base shear stresses of $4.4\sqrt{f_c'}$ [psi], ($0.34\sqrt{f_c'}$ [MPa]). This paper presents the observed damage patterns in the coupling beams and the structural walls. The restraining effect provided by the structural walls to damage-induced lengthening of the coupling beams is discussed and compared with that observed in component tests. Finally, the end rotations measured in the coupling beams relative to the drift of the coupled-wall system are also presented.

Keywords: coupling beam; coupled wall; high-performance fiber-reinforced concrete (HPFRC); precast; seismic; shear.

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INTRODUCTION

Concrete structural walls are commonly used as the primary lateral force-resisting system for both medium-and high-rise concrete and steel frame structures. Due to their stiffness and strength, structural walls attract a considerable amount of lateral force when subjected to earthquake-induced displacement reversals. The efficiency of a structural wall system can be improved by proper coupling of two or more consecutive walls through the use of short coupling beams. This coupling action reduces the demand for flexural stiffness and strength from the individual walls by taking advantage of their axial stiffness, strength, and the distance between the centroidal axes of adjacent walls to provide additional resistance to overturning moment.

For satisfactory performance of a coupled-wall system during a seismic event, the short coupling beams must retain a significant, and predictable, strength and stiffness through large displacement reversals. To ensure adequate coupling beam ductility is achieved, ACI 318-08 (ACI Committee 318 2008) requires that diagonal reinforcement be provided to resist all of the shear demand in short and highly stressed coupling beams. This reinforcement detail has been shown by many researchers to provide a stable behavior under earthquake-type displacement reversals, but can be difficult and time consuming to construct. Recent coupling beam component tests (Canbolat et al. 2005; Lequesne et al. 2009) have demonstrated that precasting coupling beams with strain-hardening, high-performance fiber-reinforced concrete (HPFRC) can simplify the construction process without sacrificing performance. The HPFRC coupling beams have exhibited a highly ductile behavior when subjected to large displacement reversals, despite requiring significantly simpler reinforcement detailing than comparable reinforced concrete beams.

The impact that the ductility exhibited by HPFRC coupling beams at the component level has on a coupled-wall system, however, has not been studied experimentally. The test described herein seeks to study the interaction between precast HPFRC coupling beams and structural walls, with special attention being paid to the ductility and strength retention of the system and the deformation demands that each system component is subjected to.

In addition to the performance of the system, the construction process required for this specimen is considered to be of great importance. The most significant limitation of the current design approach is the cumbersome reinforcement that is time consuming and difficult to assemble on-site. For the specimen described herein, the precast coupling beams were inserted between the wall's longitudinal reinforcing bars, and supported by wall formwork. The remaining wall reinforcement was subsequently placed without significant interference from beam reinforcement, which protruded horizontally from the ends of the precast member and through the wall boundary element, as shown in Fig. 1. The construction of this specimen demonstrated that HPFRC coupling beams can be precast and easily embedded into cast-in-place structural walls with minimal reinforcement interference.

Fig. 1—Precast coupling beam embedment detail.

RESEARCH SIGNIFICANCE

An experimental investigation of the implications of using precast HPFRC coupling beams on the design, constructability, and performance of coupled walls is presented. More specifically, this project seeks to: 1) demonstrate the ease with which precast coupling beams can be embedded in cast-in-place structural wall systems, 2) provide a comparison between various coupling beam details when subjected to similar deformation demands, and 3) study the interaction between HPFRC coupling beams, slabs, and structural walls.

GENERAL DESIGN AND TEST SETUP

A diagram of the coupled-wall specimen is shown in Fig. 2. Each of the coupling beams had a slightly different reinforcement layout, shown in Fig. 3, which allowed for a comparison of various reinforcement layouts. Slabs were built at the second and fourth levels to facilitate lateral load application. Slab reinforcement placed perpendicular to the loading direction was continuous through the structural walls, but not the precast coupling beams. The slabs provided an opportunity to observe the interaction between the precast coupling beams and the adjacent slab, and to evaluate the need for design modifications to minimize damage at this connection.

For design of the specimen, the base of each wall was assumed to be fixed, which was achieved experimentally through the use of deep reinforced concrete foundation elements bolted directly to the laboratory strong floor. A vertical force, equivalent to an axial stress of 7% of the design f_c', based on the gross area of the walls, was applied at the second story through external prestressing tendons anchored at the bottom of the foundation elements. Steel tube sections cast through each wall above the second story slab transferred the force from the external tendons into the walls. Hydraulic jacks were used to apply this vertical force before any lateral displacement was

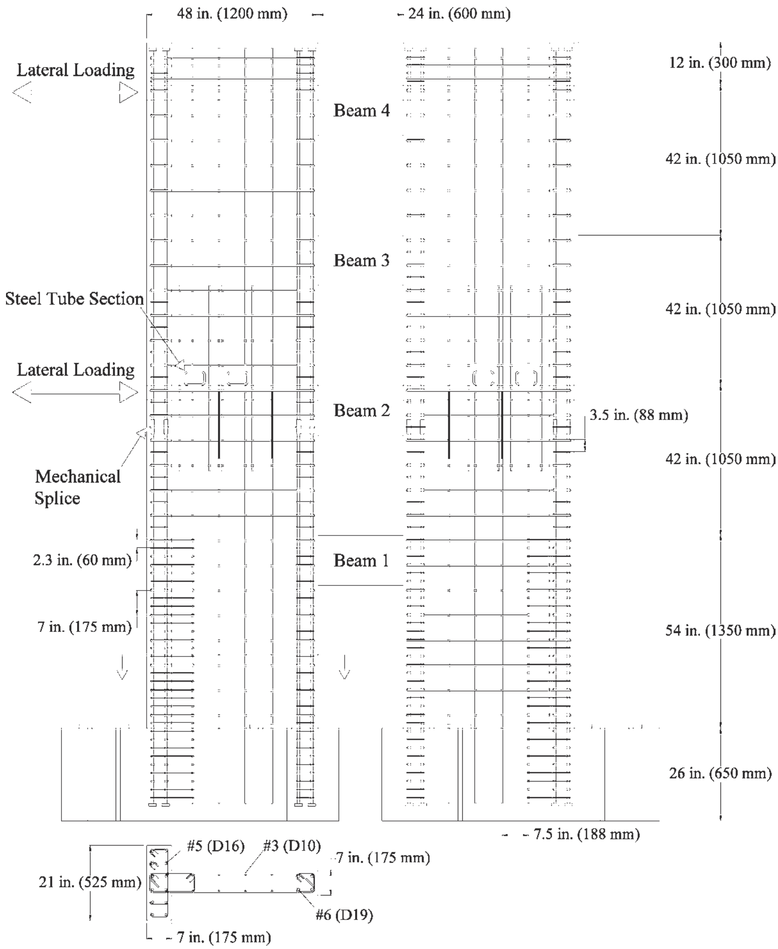

Fig. 2—Coupled-wall specimen reinforcement.

applied, and held it constant throughout the duration of the test. This level of gravity load is consistent with current design practice for structural walls, and was sufficient to offset a majority of the uplift force resulting from the coupling of the walls.

The test setup, shown in Fig. 4, was used to pseudo-statically apply lateral displacement (and load) through the slabs cast at the second and fourth levels. The actuator mounted on the fourth level applied a predetermined sequence of reversing lateral displacements, while the actuator at the second level applied a force equivalent to 60% of the force applied by the top actuator. These lateral forces were transferred to the coupled walls through a yolk and four channel sections that were attached to the top and bottom of the outer edges of the slabs. This was intended to allow for a distribution of lateral force to each of the structural walls that is similar to the load transfer mechanism that develops in a normal building system.

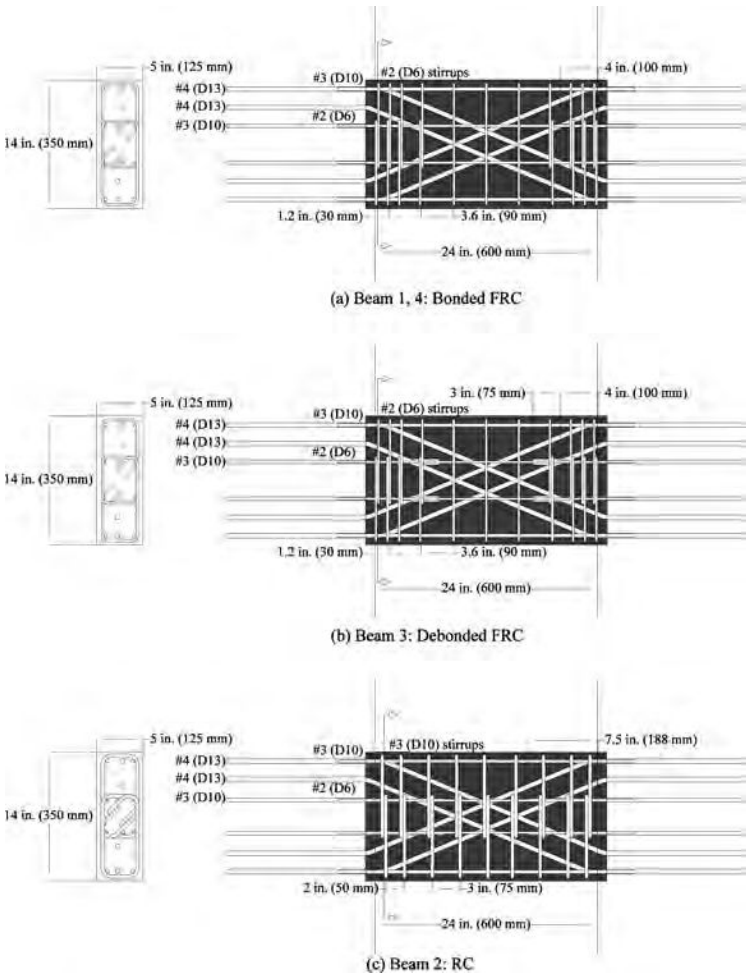

Fig. 3—Coupling beam reinforcement.

Fig. 4—Photo of test setup and specimen.

Efforts were made throughout the construction of the specimen to be as realistic as possible in terms of both construction methods and sequencing. It was felt that this approach was critical for gauging the possible construction scheduling advantages gained by incorporating precast coupling beams. The construction process consisted of first precasting the coupling beams and storing them, ready for placement. The construction of each wall story began with tying the wall reinforcement into position and then placing enough of the wall formwork to support the precast beam. The beam was then slid into position with an overhead crane and placed on the formwork that supported it fully until the wall concrete was placed. Overlapping U-shaped stirrups were used to provide confinement to the wall boundary element in the region where the coupling beam reinforcement intersected the longitudinal wall reinforcement, as shown in Fig. 1. Ensuring adequate anchorage for the special transverse reinforcement is critical, yet the preferred detail is dependent on the layout of the wall boundary element. The detail selected for this specimen consisted of overlapping U-shaped stirrups anchored by 135-degree bends around the longitudinal reinforcement. Finally, the wall formwork assembly was completed, and the concrete was placed. The formwork was then removed, moved up the wall, and the previously described sequence was repeated. The process proved to be efficient.

At the second and fourth levels, where a slab was also present, the top of the precast coupling beam was placed to be flush with the top of the slab. Although the slab concrete was cast against the precast beam, no reinforcement crossed this cold joint. It has been demonstrated that the influence of a slab on the cyclic response of coupling beams is limited to minor stiffening in early drift cycles (Paulay and Taylor 1981; Gong and Shahrooz 2001). This is due to the much lower stiffness observed in slabs relative to coupling beams when subjected to reversed cyclic loads. After the influence of the slab degrades, the slab-beam is expected to behave very similarly to a beam component without a slab. Therefore, no attempt was made to encourage interaction between the slab and precast coupling beam, which simplified casting and placement of the precast coupling beam without sacrificing performance.

INSTRUMENTATION

To record the performance of the coupled-wall specimen, strain gauges, linear potentiometers, inclinometers, load cells, and optical position sensors were used. Strain gauges were placed on coupling beam reinforcement, wall longitudinal reinforcement at every level, and transverse reinforcement within the first story of the wall. Linear potentiometers measured deformations in the second, third and fourth coupling beams, monitored the lateral displacement at the second and fourth levels, and measured vertical deformations along the edges of both structural walls. Inclinometers were used to monitor wall rotations at the second, third, and fourth levels. Load cells measured the forces applied laterally to the specimen, and the vertical force applied at the second level was monitored by hydraulic pressure gauges. Finally, an optical system was used to track the location of 144 independent markers placed in a grid covering the first story of both walls and the first coupling beam. This optical system provided reliable data that were used to calculate deformations, rotations, and displacements.

MATERIAL PROPERTIES

Recent work at the University of Michigan (Liao et al. 2006) has led to the development of a flowable HPFRC with a 1.5% volume fraction of high-strength hooked steel fibers. The mixture also includes coarse aggregate with a maximum nominal size of 0.5 in. (13 mm). The properties of the fibers, as specified by the manufacturer, are summarized in Table 1. This mixture was selected for the precast HPFRC coupling beams used in this study.

Results from compressive tests of 4 by 8 in. (100 by 200 mm) cylinders performed at 28 days and near the test dates are shown in Table 2. The test day values of f_c' are used throughout this paper. A representative compressive constitutive response, based on cylinder tests, is shown in Fig. 5, and was used for design purposes. A parabola was assumed to represent the ascending branch, followed by a shallow linear descending branch that accounted for the confinement provided by the distributed fiber reinforcement. A maximum useable concrete compressive strain of 0.8% was assumed.

Average tensile properties previously determined for this HPFRC mixture (Liao et al. 2006) were used for design of the test specimens. The representative tensile stress-strain response, shown in Fig. 6, has a peak tensile stress of 500 psi (3.4 MPa) at 0.5% strain, 25% higher than the first cracking stress. This peak is followed by a gradual decrease in tensile stress capacity. On average, tensile specimens still carried 50% of their peak tensile stress at 1.4% strain.

Table 1—Hooked steel fiber properties

Length (in./mm)		Diameter (in./mm)		L/d	Minimum tensile strength (ksi/MPa)	
1.2	30	0.015	0.38	80	330	2300

Table 2—Concrete properties

Portion of specimen	Fibers?	28-day tests								Test day f'_c (ksi/MPa)	
		f'_c (ksi/MPa)		ASTM C1609/C1609M-05 flexural tests							
				σ_{fc}* (psi/MPa)		σ_{peak}† (psi/MPa)		$\sigma_{(\sigma=L/150)}$‡ (psi/MPa)			
CB-1	Y	5.5	38	765	5.3	1090	7.5	540	3.7	10.4	72
CB-2	N	5.3	37	—	—	—	—	—	—	9.8	68
CB-3	Y	5.5	38	765	5.3	1090	7.5	540	3.7	10.4	72
CB-4	Y	6.0	41	850	5.9	1120	7.7	610	4.2	10.8	74
Foundation	N	5.0	34	—	—	—	—	—	—	7.8	54
Wall 1st lift	N	5.3	37	—	—	—	—	—	—	7.0	48
Wall 2nd lift	N	4.1	28	—	—	—	—	—	—	6.7	46
Wall 3rd lift	N	5.5	38	—	—	—	—	—	—	6.6	45
Wall 4th lift	N	6.9	48	—	—	—	—	—	—	9.5	65

*bending stress at first crack; †bending stress at peak stress; ‡bending stress at a deflection of L/150.

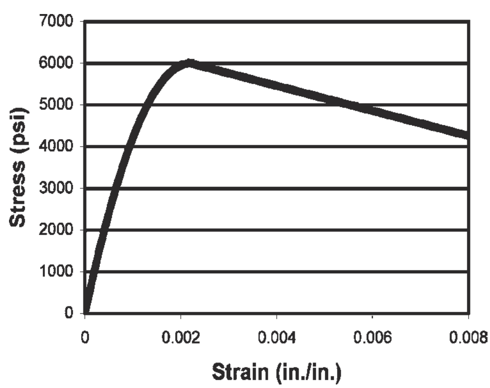

Fig. 5—Compression constitutive model. (Note: 1 in. = 24.5 mm; 1 psi = 0.0069 MPa.)

Bending tests were also performed on short beams with dimensions of 6 x 6 x 20 in. (150 x 150 x 500 mm), in accordance with ASTM C1609/C1609M-05. These tests were performed 28 days after casting each HPFRC coupling beam to characterize the bending properties of the HPFRC used in this study. The results of these tests are summarized in Table 2 by the following three values: the equivalent bending stress at first crack (σ_{fc}), peak stress (σ_{peak}), and at a deflection of $L/150$, where L is the beam span length of 18 in. (450 mm). All tests showed pronounced deflection hardening behavior, with peak bending stresses occurring near deflections of $L/800$, which exceeded the first cracking stress by more than 30%.

The tensile stress-strain properties of the reinforcing steel used for the construction of this specimen were evaluated by direct tensile tests on representative coupons. The measured yield and ultimate stresses from these tests are summarized in Table 3.

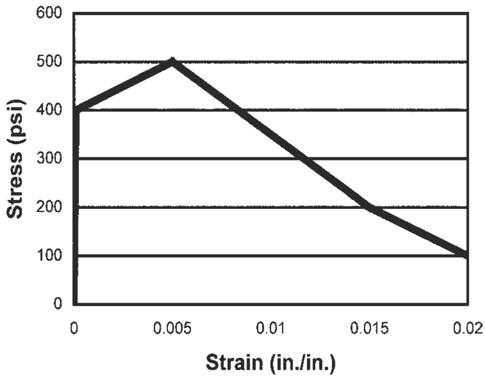

Fig. 6—Tension constitutive model. (Note: 1 in. = 24.5 mm; 1 psi = 0.0069 MPa.)

Table 3—Steel reinforcement properties

Portion of specimen	Bar size	Yield stress, ksi (MPa)	Ultimate stress, ksi (MPa)
Coupling beams	No. 3 (D10)	79.4 (550)	121 (830)
	No. 4 (D13)	76.9 (530)	115 (790)
Structural wall	No. 3 (D10)	74.2 (510)	112 (770)
	No. 5 (D16)	67.2 (460)	109 (750)
	No. 6 (D19)	68.0 (470)	109 (750)

SPECIMEN DESIGN

Coupling beam design

Three different reinforcement details, shown in Fig. 3, were selected for the coupling beams in this system. Care was taken to ensure that all of the designs exhibited similar initial stiffnesses and calculated ultimate flexural capacities, which resulted in shear stresses near $7\sqrt{f_c'}$ [psi] ($0.55\sqrt{f_c'}$ [MPa]). This was done to prevent any particular beam from attracting more shear than the others. One unavoidable design issue is the contribution of the fiber reinforcement to the moment capacity of fiber reinforced coupling beams, compared with conventional reinforced concrete beams. Although this contribution will increase the beam capacity near and at the first yield point, the impact will diminish at larger deformations. Significant yielding was observed in the diagonal reinforcement of all four coupling beams by the time the system drift, defined as the lateral displacement of the top slab divided by the wall height off of the foundation, reached 1%. Thus, it is reasonable to assume that shears were relatively evenly distributed between the beams at these larger drifts.

The first coupling beam design, which was used as Beams 1 and 4 in the coupled wall, is labeled "Bonded FRC," and is shown in Fig. 3(a). This design is comparable to the component tests reported by Lequesne et al. (2009). The following notes should be made on this design:

- Diagonal bars were provided to carry approximately half of the expected ultimate shear capacity and to improve the rotational ductility at the beam ends, where plastic hinges were expected to develop. The diagonal bar contribution to shear was originally targeted to be close to 40% of ultimate, but due to scaling issues, a slightly larger diagonal bar contribution resulted. No special transverse reinforcement, except for the beam ends, was provided to prevent buckling of the diagonal bars because strain-hardening HPFRC composites have been shown to confine diagonal reinforcement and arrest any tendency to buckle (Canbolat et al. 2005).
- Longitudinal reinforcement was provided, but only embedded 3 in. (75 mm) into the walls. This is commonly done to limit the contribution of the longitudinal reinforcement to the flexural capacity of the coupling beam.
- To strengthen the interface between the precast fiber-reinforced beam and the structural wall, and to encourage plastic hinging to develop inside the fiber reinforced section, dowel bars were provided across the beam-wall interface and terminated 4 in. (100 mm) into the beam. The high bond stress developed between fiber-reinforced concrete and reinforcing bars, addressed by Chao (2005), made this very short development length sufficient to yield the dowel bars near the interface.
- Transverse reinforcement was provided in the beam for the first $h/2$ away from the wall face to confine the beam plastic hinge regions. Stirrups were provided throughout the remaining span to carry approximately one-half of the expected shear.

The second coupling beam design, which was used as Beam 3 in the coupled wall, is labeled “Debonded FRC,” and is shown in Fig. 3(b). This design was identical to the previous design, with one detailing change. Within the beam, the dowel bars were extended 3 in. (75 mm) beyond the 4 in. (100 mm) development length and debonded over that added length. The term “debonding” is used herein to describe the use of mechanical means to prevent the fiber reinforced concrete from bonding with the reinforcing bar. This was accomplished by wrapping the bar with a few layers of plastic sheeting, and sealing it with tape. The intent was to delay the development of a single failure plane by eliminating the disturbance resulting from the physical discontinuity of the terminated bar. The motivation for this detail came from the observation that the dowel bars in previous component tests were successful in moving the ultimate failure plane away from the interface to the plane where the dowel bars were terminated. If possible, it would be advantageous to spread that flexural yielding out through a larger portion of the coupling beam, thus further delaying the localization of rotations.

The third coupling beam design, which was used as Beam 2 in the coupled wall, is labeled “RC,” and is shown in Fig. 3(c). This reinforced concrete beam design was unique because it investigated the potential for precasting non-fiber-reinforced concrete coupling beams, which could offer construction time-savings if proven to be successful. To account for the precasting and embedment of this coupling beam, ACI 318-08 requirements were modified, and a detail more similar to the “Bonded FRC” design discussed previously was selected. The following modifications to the “Bonded FRC” design were made to account for the lack of fiber reinforcement.