Seismic Detailing and Behavior of Coupling Beams With High-Performance Fiber Reinforced Concrete

By Rémy Lequesne, Monthian Setkit, Gustavo Parra-Montesinos, and James K. Wight

Synopsis: This paper summarizes a series of tests performed on strain hardening High-Performance Fiber Reinforced Concrete (HPFRC) coupling beams with span length-to-depth ratios (ℓ_n / h) of 1.75 and 2.75. These tests show that incorporating HPFRC simplifies the detailing required to ensure a stable response of coupling beams subjected to earthquake induced displacement reversals. Results from five tests of precast coupling beams, three with $\ell_n / h = 1.75$ and two with $\ell_n / h = 2.75$, are reported herein. Strategies for embedding the precast HPFRC coupling beams into the structural walls without interfering with boundary element reinforcement were explored. Test results confirm that HPFRC can reliably confine diagonal reinforcement and ensure stable hysteresis behavior. HPFRC was also found to significantly increase shear strength, thereby forcing a flexurally dominated failure mode with modest stiffness degradation and excellent energy dissipation. A revised coupling beam design philosophy is outlined in order to ensure ductile flexural behavior.

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INTRODUCTION

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

For satisfactory performance of this structural system during a seismic event, the short coupling beams must retain a significant strength through large displacement reversals, while suffering only a modest degradation of stiffness. For coupling beam span-to-depth ratios of less than two, it has been shown[1] that special diagonal reinforcement is required to eliminate the development of a sliding shear failure, and thus, provide adequate ductility and stiffness retention to achieve the design goals of the system. However, the reinforcement detailing required to ensure a stable coupling beam behavior is difficult to construct, and often fails to maintain the integrity of the full concrete element through large displacement reversals. Other alternatives have been proposed and investigated[2,3,4], including various reinforced concrete, steel, and hybrid steel-concrete coupling beam designs. Of these potential solutions, steel and hybrid steel-concrete coupling beams exhibit the most favorable response to reversed cyclic loading, and have begun to find their way into design practice. However, despite the improved hysteresis behavior achievable with steel coupling beams, providing proper anchorage of the steel section without disrupting reinforcement in the structural wall remains a significant challenge to engineers and contractors.

Recent work on very short coupling beams ($\ell_n / h = 1.0$)[5,6] demonstrated the potential for strain hardening, High-Performance Fiber Reinforced Concrete (HPFRC) to provide improved ductility over more traditional concrete coupling beams, while simplifying detailing requirements. Fiber reinforced concrete takes advantage of the inclusion of randomly disbursed fibers within the concrete matrix to provide enhanced transfer of tensile stresses across cracks. When used in sufficient amounts some types of fibers may lead to a strain hardening tensile response, in which case the material is referred to as a high-performance fiber reinforced concrete. On a structural scale, this strain hardening property translates into increased shear resistance, damage tolerance, and better stiffness retention[7]. Further, HPFRC has been shown to provide confinement to steel reinforcement and superior bond with reinforcing bars[8].

RESEARCH SIGNIFICANCE

The implications of using precast HPFRC coupling beams on the constructability of coupled walls, and on the ductility and damage tolerance of these systems when subjected to displacement reversals, may be appreciable. The experimental work described herein seeks to quantify these benefits and to provide engineers with a practical design alternative to current code compliant coupling beam designs. Test results show that HPFRC adequately confines the diagonal reinforcement, eliminating the need for cumbersome special confinement for these bars. These results also show that the area of diagonal reinforcement can be significantly reduced when HPFRC is used, and that much of the difficult detailing requirements for coupling beams can be relaxed without sacrificing performance.

PREVIOUS RESEARCH

In 1971, Paulay[9] demonstrated that the development of a sliding shear failure mechanism could not be adequately controlled in short, highly stressed coupling beams with moment frame-type reinforcement. The use of diagonal reinforcement was then investigated to resist shear in coupling beams[1,10], and a remarkable improvement in both strength and ductility was demonstrated experimentally. This diagonal reinforcement approach has been accepted by the ACI Building Code[11], and is required for short and highly stressed coupling beams, where $\ell_n / h < 2.0$ and $V_u > 4\lambda\sqrt{f_c'}A_{cw}$, [psi] $(0.33\lambda\sqrt{f_c'}A_{cw}$, [MPa]) , where: $\ell_n =$ clear span of coupling beam, $h =$ height of coupling beam, $V_u =$ shear demand, $\lambda =$ correction factor for lightweight concrete, $f_c' =$ compressive strength of concrete, and $A_{cw} =$ gross cross-sectional area of coupling beam. Unfortunately, this diagonal reinforcement scheme requires substantial confinement of either the diagonals themselves or of the coupling beam section as a whole. These complications[12] often cause the construction of coupling beams and coupled walls to control the construction schedule in seismic zones.

a) Special moment frame detailing   b) Full-length/cut-off dowels

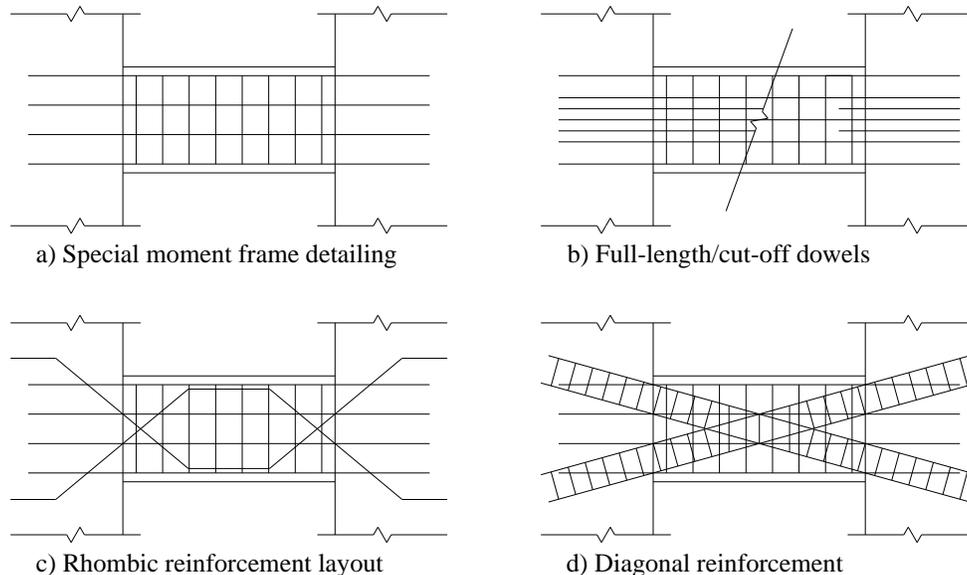c) Rhombic reinforcement layout   d) Diagonal reinforcement

Fig. 1– Alternative reinforcement layouts

To resolve these issues, many alternative approaches have been evaluated. Two relatively recent studies, performed by Galano and Vignoli[2] and Tassios, Moretti, and Bezas[4], sought to compare the relative strength, ductility, and damage tolerance of several proposed alternatives to diagonally reinforced concrete coupling beams. The alternatives considered, shown in Fig. 1, included a) special moment frame detailing, b) full-length and cut-off dowel bars crossing the coupling beam/wall interface to prevent sliding shear failures, c) "rhombic" type reinforcement, which provides unconfined diagonal reinforcement through the plastic hinge, and, d) confined and unconfined diagonally reinforced coupling beams. These tests were performed on short coupling beams with ℓ_n / h ratios of 1.0, 1.5, and 1.66. The results of these studies illustrate the following key points:

- Special moment frame detailing is not appropriate for short coupling beams subjected to high shear stresses.

• Dowel bars crossing the coupling beam/wall interface were shown to help prevent sliding shear failures, but could not prevent stiffness degradation and severe pinching in the hysteresis response.

• A rhombic layout of diagonal reinforcement requires less complicated detailing than diagonally reinforced coupling beams, and exhibited less stiffness degradation than coupling beams detailed as special moment frames. However, pinching of the hysteresis loops was still present.

• Of the reinforced concrete options considered, confined diagonal reinforcement appeared to provide the most stable behavior and highest energy dissipation. It was noted that providing sufficient confinement to ensure stability of the diagonal bars is "indispensable", despite the significant field placement difficulties.

Steel and hybrid steel-concrete alternatives have also been evaluated, as discussed by Harries, Gong, and Shahrooz[3]. When properly detailed and fully anchored to the adjoining structural walls, these alternatives have been shown to respond very favorably to cyclic shear, showing stable behavior with wide hysteresis loops. Unfortunately, the steel elements require a long embedment into the adjoining structural walls to ensure a full development of flexural and shear capacities, which inevitably interferes with critical transverse reinforcement required for confinement in the boundary regions of the structural walls.

MATERIAL PROPERTIES

Recent work at the University of Michigan[13] has led to the development of a highly flowable HPFRC with a 1.5% volume fraction of high-strength hooked steel fibers. The properties of the fibers, as specified by the manufacturer, are summarized in Table 1. This mixture was selected for use in all of the coupling beam tests described herein. Results from compressive tests on 4 in. by 8 in. (100 mm by 200 mm) cylinders at 28 days and near the test dates are shown in Table 2. These test day values of f'_c are used throughout this paper. The compressive constitutive response shown in Fig. 2 was used for modeling purposes. A parabolic ascending branch was assumed, followed by a linear descending branch representative of the confinement provided by the distributed fiber reinforcement. Crushing was assumed to occur at 0.8% 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	333	2300

Average tensile properties previously determined for this mixture[13] were used for design of the test specimens. This tensile stress-strain response, shown in Fig. 3, is represented by a peak tensile stress of 500 psi (3.4 MPa), 25% higher than the first cracking stress, occurring around 0.5% strain. 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.
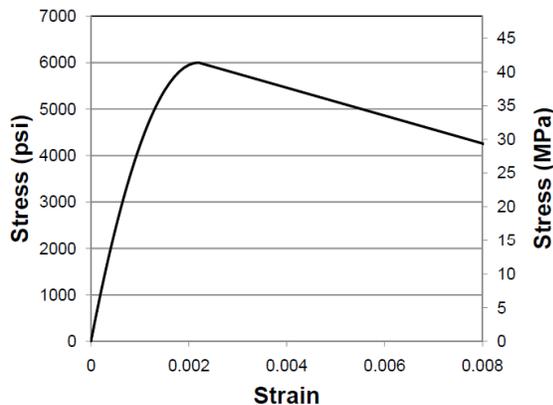

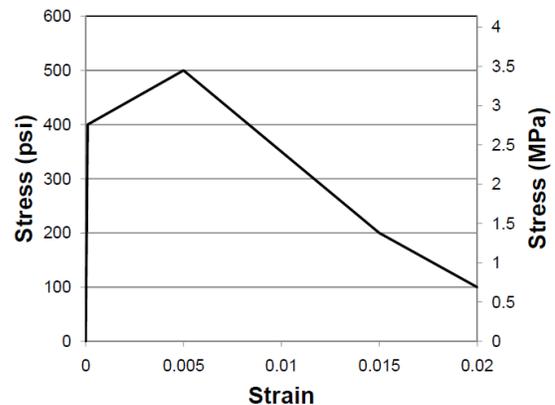Fig. 2 – Compressive constitutive model

Fig. 3 – Tensile constitutive model

Bending tests were also performed on 6 in. by 6 in. by 20 in. (150 mm by 150 mm by 500 mm) prisms following ASTM 1609[14] at 28-days to characterize the bending properties of HPFRC. These results are summarized in Table 2, where the equivalent bending stress at first crack (σ_{fc}), peak (σ_{peak}), and at a deflection of $L/150$ are reported, where L is the beam span length (18 in. or 460 mm). Note that all tests showed pronounced deflection hardening behavior, with peak bending stresses exceeding the first cracking stress by more than 30%.

Table 2 – Concrete properties

	Specimen Number	Aspect Ratio	28-Day Tests								Test Day f'c (ksi/MPa)	
			f'c (ksi/MPa)		ASTM 1609 Flexural Tests							
					σ_{fc} (psi/MPa)		σ_{peak} (psi/MPa)		$\sigma_{(\delta=L/150)}$ (psi/MPa)			
Phase I	CB-1	1.75	5.5	38	640	4.4	930	6.4	540	3.7	6.5	45
	CB-2	1.75	6.1	42	630	4.3	890	6.1	400	2.8	7.6	52
	CB-3	1.75	5.0	34	690	4.8	860	5.9	330	2.3	5.0	34
Phase II	1	2.75	5.0	34	700	4.8	1000	6.9	450	3.1	7.0	48
	2	2.75	7.4	51	800	5.5	1100	7.6	500	3.4	8.8	61

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

Table 3 – Steel reinforcement properties

	Specimen	Aspect Ratio	Bar Size	Yield Stress (ksi/MPa)		Ultimate Stress (ksi/MPa)	
Phase I	CB-1	1.75	#3 (D10)	76.5	527	116	799
			#4 (D13)	72.5	500	107	737
			#5 (D16)	62.6	431	98.7	680
	CB-2	1.75	#3 (D10)	68.8	474	108	744
			#4 (D13)	63.5	438	99.6	686
			#5 (D16)	62.6	431	98.7	680
	CB-3	1.75	#3 (D10)	60.4	416	92.8	639
			#4 (D13)	63.5	438	101	696
			#5 (D16)	61.2	422	104	717
Phase II	1	2.75	#3 (D10)	53.7	370	87.6	604
			#4 (D13)	61.4	423	101	696
			#5 (D16)	61.1	421	99.1	683
			#6 (D19)	74.6	514	95.1	655
	2	2.75	#3 (D10)	53.7	370	87.6	604
			#4 (D13)	60.0	413	99.9	688
			#5 (D16)	60.6	418	99.0	682
			#6 (D19)	82.6	569	102	703

EXPERIMENTAL PROGRAM, PHASE I: HPFRC COUPLING BEAMS WITH ℓₙ/h = 1.75

Discussion of tests

Three coupling beams with $\ell_n / h = 1.75$ were precast out of HPFRC described above, and connected to stiff reinforced concrete blocks simulating wall boundary elements. The intent of this series of tests was to evaluate the applicability of HPFRC in coupling beams with relatively large aspect ratios, where flexural deformations contribute more significantly to drift capacity. In particular, this study sought to: 1) confirm that HPFRC can be relied on to confine diagonal reinforcement, 2) quantify the shear stress capacity that can safely be attributed to the HPFRC in design, 3) determine whether the response of coupling beams with $\ell_n / h = 1.75$ can be controlled by flexure, thereby improving ductility, 4) evaluate the degree of stiffness degradation at various drift levels, and 5) compare alternatives for embedding the precast HPFRC section into the adjoining structural walls without interfering with the boundary element reinforcement.

For these tests, the coupling beam specimen was precast and embedded into stiff reinforced concrete elements simulating the interface with adjacent structural walls. The test setup, shown in Fig. 4, consisted of fixing one of these "wall" elements to the laboratory strong floor, and restraining the rotation of the other wall with pinned steel links designed to control vertical movement. These steel links had the added benefit of providing passive resistance to elongation of the coupling beam resulting from damage due to cyclic displacements, similar to the restraint provided by stiff structural walls. The effect of the resulting axial load on the strength and behavior of the coupling beam can be significant and should be considered in design if a reasonable method for estimating this force can be determined.
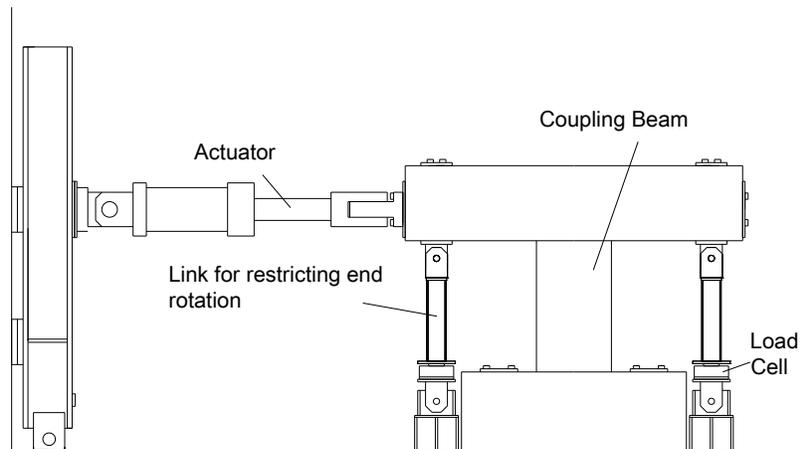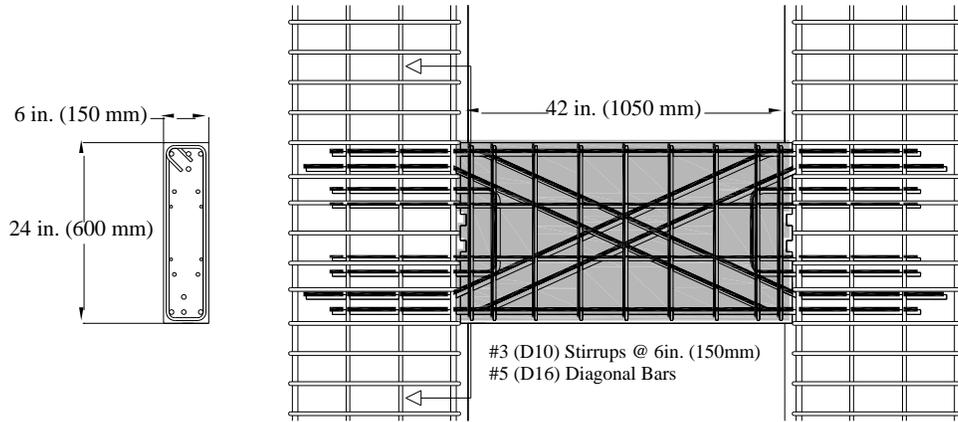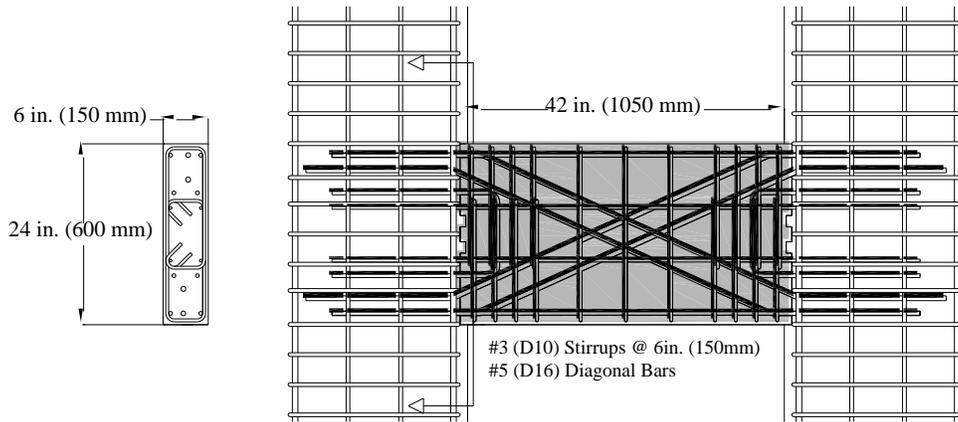

Fig. 4 – Test setup

The philosophy behind the design of these specimens, shown in Fig. 5, represents a major change from the current state of the art. It is believed that the improved damage tolerance exhibited by HPFRC elements should be used advantageously in design. Rather than assuming that the capacity of the coupling beam is uniquely controlled by two intersecting diagonal reinforcement cages, the full coupling beam section is assumed to remain intact and therefore active in resisting shear and moment. At the current aspect ratio, diagonal reinforcement is still considered critical for ensuring a ductile response to cycling, but the diagonal steel contribution to shear capacity is substantially reduced and accounts approximately for only 1/3 of the ultimate shear capacity. The remaining shear is resisted by the transverse reinforcement and the HPFRC. With the combination of diagonal reinforcement, HPFRC, and stirrups resisting shear, it was assumed that a more reliable and ductile flexural mechanism would develop and control the response. With this assumption, the capacity of the coupling beam could be controlled by providing flexural reinforcement at the coupling beam/wall interface that corresponds to the desired level of shear demand.
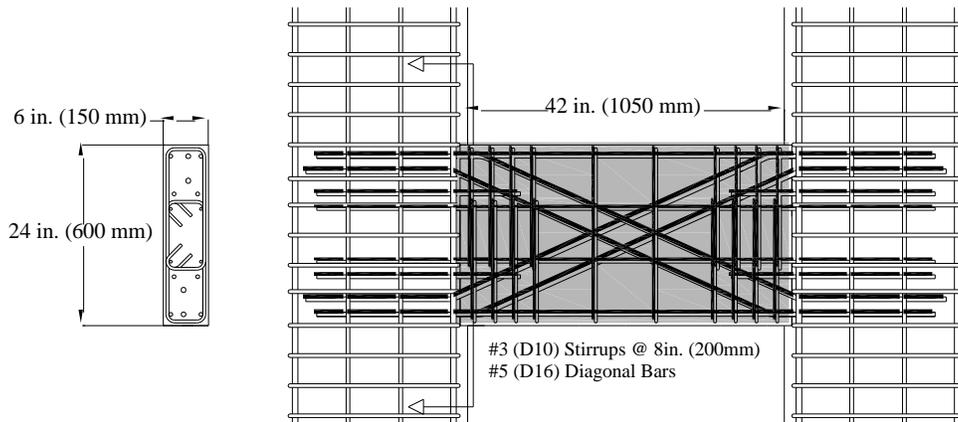
It should also be noted that the coupling beam specimens were precast and embedded into the adjacent walls in an effort to incorporate the more costly HPFRC only in regions where its use is most advantageous. To limit interference with the structural wall boundary element, the HPFRC section was embedded only as deep as the concrete cover for the wall reinforcement, which for the scale considered was equal to 1 in. (25 mm). Two different connection reinforcement details, each consisting of either U-shaped or straight dowel bars, were successful at forcing flexural rotations to localize away from the coupling beam-wall interface, as intended.

6 in. (150 mm)

24 in. (600 mm)

42 in. (1050 mm)

#3 (D10) Stirrups @ 6in. (150mm)
#5 (D16) Diagonal Bars

(a) Specimen CB-1

6 in. (150 mm)

24 in. (600 mm)

42 in. (1050 mm)

#3 (D10) Stirrups @ 6in. (150mm)
#5 (D16) Diagonal Bars

(b) Specimen CB-2

6 in. (150 mm)

24 in. (600 mm)

42 in. (1050 mm)

#3 (D10) Stirrups @ 8in. (200mm)
#5 (D16) Diagonal Bars

(c) Specimen CB-3

Fig. 5 – Reinforcement details for phase I tests with $\ell_n / h = 1.75$

The performance of Specimen CB-1, shown in Fig. 6a, was initially favorable. Relatively wide loops, indicative of yielding in the reinforcement, indicated good energy dissipation. The shear stress capacity of approximately $12\sqrt{f_c'}$, [psi] $(1.0\sqrt{f_c'}$, [MPa]) near 2% drift was remarkably high. As noted earlier, the diagonal reinforcement accounted for only roughly 1/3 of this strength though, which indicates that the HPFRC was effective in enhancing the shear resistance of the coupling beam. Specifically, the HPFRC affects the shear resistance mechanisms in three ways: 1) buckling is prevented in the diagonal reinforcement, allowing for both the tension and compression diagonal steel to resist shear, 2) damage in the beam is limited to multiple narrow cracks, resulting in a section whose integrity is maintained sufficiently for a truss mechanism to develop even at large drifts, and 3) the fibers bridging cracks contribute directly to transferring tensile stresses across cracks and enhancing aggregate interlock.

To understand the detailing issue that caused the premature degradation of the hysteresis loops near 2% drift, first note that the diagonal reinforcement was bent before entering the wall. This was done to ease placement of the precast section and to increase the efficiency of the diagonal reinforcement for resisting shear by increasing the angle of inclination. This resulted in a force component that tended to burst through the top and bottom of the coupling beam if not sufficiently restrained by ties. The experimental results indicated that the transverse reinforcement provided was insufficient to both adequately restrain this force component and confine the plastic hinge region. In subsequent tests, this deficiency was addressed by providing a tighter spacing of ties in the plastic hinge region, and by providing additional tie legs through the center of the hinge to restrain lateral expansion.

Fig. 5b shows that additional confinement of the plastic hinge region, over a length of $h/2$, was provided in Specimen CB-2. This change resulted in a significantly more stable hysteresis response than observed previously, which is shown in Fig. 6b. Despite the very high shear stresses imposed, which exceeded $11.5\sqrt{f_c'}$, [psi] $(0.87\sqrt{f_c'}$, [MPa]), this second specimen performed in a very stable manner, characterized by a high level of strength and stiffness retention, with minimal observed pinching, even at drifts exceeding 5%.

The results from the test of Specimen CB-2 confirmed that it is not reasonable to assume that the behavior of an HPFRC coupling beam with $\ell_n/h = 1.75$ is governed by the diagonal reinforcement alone. As in Specimen CB-1, diagonal steel was provided to only carry approximately 1/3 of the shear force applied to the section. The high shear capacity, bolstered by the observation that no diagonal cracks widened throughout the test, proves that the stirrups and HPFRC were active in resisting shear. This allowed for plastic hinges to develop near the ends of the coupling beam, as intended. The special column-type confinement provided in the plastic hinge region complemented the confinement provided by the HPFRC to allow for a very ductile and stable flexural mechanism to control the failure. Late in the test, flexural cracks from opposing drift half-cycles coalesced along a stirrup in the plastic hinge region, creating a plane with a reduced ability to resist sliding shear. Failure ultimately occurred along this plane at drifts exceeding 5.5%.

In Specimen CB-3, a simpler connection detail, consisting of straight dowel bars placed across the beam-wall interface to encourage failure to occur within the beam, was evaluated. Also, the transverse reinforcement ratio at midspan was reduced by 25% relative to the previous specimens. Despite the unsymmetrical loading history imposed due to an unforeseen issue with the test setup, the response of Specimen CB-3 again showed good stiffness and strength retention with minimal pinching of the hysteresis response out to drifts exceeding 5%. Fig. 6c shows that shear stresses of up to $14\sqrt{f_c'}$, psi $(1.17\sqrt{f_c'}$, MPa) were imposed in the positive loading direction. As in the other two tests, the response of Specimen CB-3 was dominated by flexural rotations near the ends of the coupling beam, which ultimately led to a sliding shear failure plane at drifts exceeding 5% in the positive loading direction. This ductile result is possible when diagonal shear cracking is controlled, which was achieved despite a 25% reduction in the stirrup area provided.

HPFRC Contribution to Shear

Strain gauges were applied to the diagonal reinforcement and stirrups to evaluate how active these bars were throughout the test. These measured strains were used to estimate the shear force carried by these bars near maximum loading using constitutive relationships derived experimentally. The remaining shear was assumed to be resisted by the HPFRC.
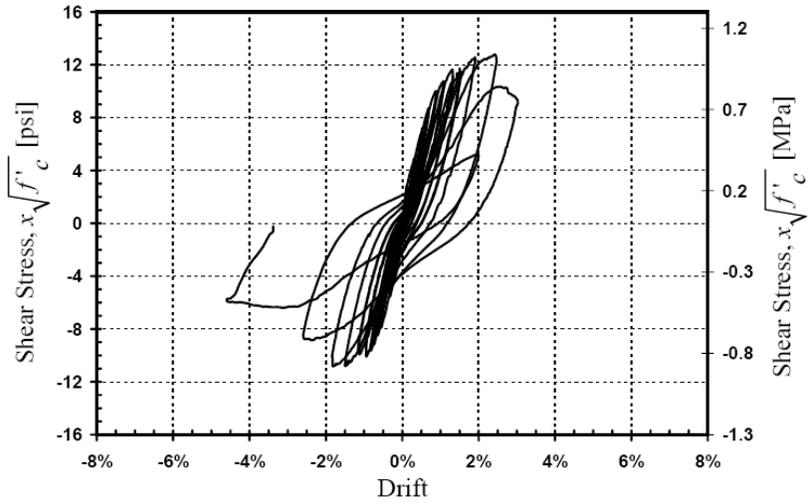
(a) Specimen CB-1
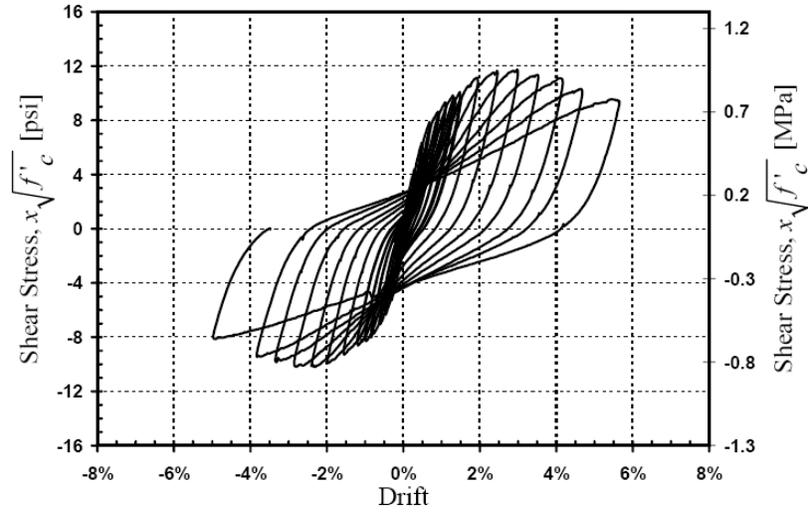
(b) Specimen CB-2
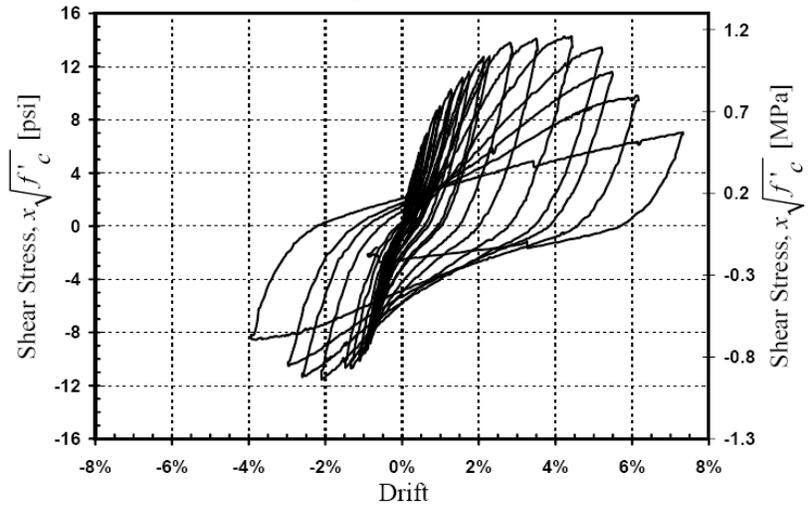
(c) Specimen CB-3

Fig. 6 – Shear stress vs. drift for coupling beams with $\ell_n / h = 1.75$

Specimen CB-1 was subjected to a peak shear stress of $12.7\sqrt{f_c'}, [psi]$ $(1.06\sqrt{f_c'}, [MPa])$. Approximately 40% of this shear stress, or $4.8\sqrt{f_c'}, [psi]$ $(0.4\sqrt{f_c'}, [MPa])$, was resisted by the HPFRC. It was estimated that 45% of the shear applied to Specimen CB-2, or $5.3\sqrt{f_c'}, [psi]$ $(0.44\sqrt{f_c'}, [MPa])$, and 40% of the shear applied to Specimen CB-3, or $5.1\sqrt{f_c'}, [psi]$ $(0.42\sqrt{f_c'}, [MPa])$, was resisted by the HPFRC.

Because diagonal tension did not control the failure of any of these specimens, it is not possible to place an upper bound on the HPFRC contribution to the shear strength of the coupling beams tested herein. Based on the observed HPFRC contribution to shear resistance, and considering that no widening of diagonal shear cracking was observed, a level of shear stress of $5\sqrt{f_c'}, [psi]$ $(0.42\sqrt{f_c'}, [MPa])$ was attributed to the HPFRC for the design of subsequent specimens. This is considered a reasonable value, given the results, but further experimental evidence is necessary to improve confidence in this value for design.

Discussion of Embedment Details

HPFRC has been shown to improve the ductility and damage tolerance of coupling beams while alleviating congestion, yet the cost of incorporating fiber reinforcement throughout a structure remains prohibitive. It has been proposed[5] that precasting the HPFRC coupling beam and embedding it into the adjacent structural walls could allow for HPFRC to be utilized where it is most beneficial, without negatively impacting the cost of the entire structure. This precasting approach has the added benefit of providing a more controlled environment for manufacturing, thereby ensuring high quality of construction of the coupling beams. Further, if a connection detail that does not interfere with reinforcement in the wall boundary element can be validated, placement of these precast beams could prove to be less disruptive than comparable steel and hybrid coupling beams.

Two alternative connection details were tested. Both details assumed that the precast concrete is only embedded as deep as the wall concrete cover, which for the scale considered was only 1 in. (25 mm). Coupling beam reinforcement was extended beyond the precast section and into the walls to transfer shear and moment across this interface. This deep embedment of only straight horizontal reinforcing bars allows for placement of the precast beam through the wall boundary reinforcement. To enhance the sliding shear resistance at the interface, shear keys were provided in the first two coupling beam specimens, as shown in Fig. 5. Note that removing these shear keys in Specimen CB-3 did not negatively impact the performance of the connection.

Both connection details were intended to force plastic rotations to occur within the precast element where the fiber reinforcement and column-type confinement should ensure the development of a stable flexural hinge. To achieve this, the first detail, adopted in Specimens CB-1 and CB-2, consisted of two U-shaped reinforcing bars extending 6 in. (150 mm) into the coupling beam before being bent for anchorage. The result is that the ratio of moment capacity to moment demand in the coupling beam, at the termination of the U-shaped bar, was similar to that at the cold joint. Test results, shown in Fig. 6, verified that this detail prevented localization of rotations at the coupling beam-wall interface.

The second proposed detail, used in Specimen CB-3, consisted of straight dowel bars crossing the cold joint. It was believed that this simpler detail would better distribute plastic rotations throughout the hinge than the previous detail. If it is assumed that the stress in a dowel bar varies linearly from zero at the point where it is terminated to yield at the cold joint, then the development of the dowel bar would result in a gradually diminishing contribution of the dowel bar to the moment capacity of the coupling beam at cross sections away from the interface. As a result, the moment capacity and demand would follow a similarly sloped decline throughout the plastic hinging zone, which encourages a more desirable distribution of flexural yielding. A satisfactory distribution of plastic rotations was observed in the test of Specimen CB-3. Note that only 8 in. (200 mm) of development was provided for a #4 (D13) bar. This reduced development length was possible due to the enhanced bond strength developed between deformed steel reinforcement and HPFRC, as discussed by Chao[8].

EXPERIMENTAL PROGRAM, PHASE II: HPFRC COUPLING BEAMS, $\ell_n/h = 2.75$

The 2008 ACI Building Code[11] made changes to Chapter 21, which governs the design of coupling beams, to clarify the reinforcement requirements for coupling beams with $\ell_n/h = 2$ to 4. The new code language provides two options to engineers: 1) Design coupling beams according to Section 21.9.7.4, identical to very short coupling beams where diagonal reinforcement is assumed to be the only shear resisting mechanism, or, 2) Design coupling beams according to Sections 21.5.2 through 21.5.4, which govern the design of special moment frames.

It is the opinion of the writers that neither option provides an optimal design for these moderate length coupling beams, particularly when HPFRC is used. At these longer aspect ratios, the angle of inclination of the diagonal reinforcement is necessarily often below 20 degrees, making the diagonal reinforcement inefficient for

resisting shear. As a result, an impractically large area of confined diagonal reinforcement is required for coupling beams expected to resist high shears. If the second alternative is selected, there is no provision for including a moderate amount of diagonal reinforcement, which has the effect of preventing a sliding shear failure and improving coupling beam ductility when subjected to high shear stress reversals. These observations are particularly pertinent to the design of HPFRC coupling beams, where the HPFRC resists a significant amount of the applied shear.

To offer engineers an alternative design option for coupling beams with $\ell_n / h = 2$ to 4, two precast HPFRC coupling beams with an aspect ratio of 2.75 were tested under displacement reversals. To accommodate these longer beams without reducing the cross section size, the test setup shown in Fig. 4 was modified with extensions to the steel links.

Based on the results from the Phase I tests described above, the following revised design philosophy for HPFRC coupling beams was adopted for the design of the coupling beams with longer aspect ratios.

1) Assume that diagonal reinforcement provides approximately 30-40% of the required shear strength (V_n), and select appropriate diagonal bar orientation angle and bar sizes.
2) The contribution of the diagonals to moment strength is now known. Assuming that the HPFRC carries roughly 15% of the total moment strength, select flexural reinforcement (A_s) to provide the remaining design flexural strength.
3) Place a reasonable fraction of this longitudinal flexural reinforcement as dowel bars to ensure that failure will not localize at the interface with the wall.
    -In the tests presented herein, "reasonable fraction" corresponds to:
        -30-40% of A_s for coupling beams with an aspect ratio less than 2
        -15-20% of A_s for longer coupling beams
4) Shear design: consider the contribution of the diagonal reinforcement selected in Step 1 to V_n; then provide 30-40% of V_n through transverse reinforcement, and attribute the remainder to the HPFRC. Verify that the shear carried by the HPFRC does not exceed $5\sqrt{f'_c}$, [psi] $(0.42\sqrt{f'_c}$, [MPa]$)$.
5) Provide column-type confinement to the ends of the coupling beam, extending approximately $h/2$ away from the face of the walls. No special confinement is required in addition to the confinement provided by the HPFRC for the remaining portion of the span.
6) Through a moment-curvature analysis, check that (and modify flexural design if required):

$$M_n \geq \frac{M_u}{\phi}$$

    -For this moment-curvature analysis, the constitutive model selected for the HPFRC has an important impact on the validity of the results. In this study, the constitutive models shown in Figs. 2 and 3 were used for analysis.

The reinforcement provided for the two specimens tested in this series is shown in Fig. 7. The expected shear stress for these beams was $10\sqrt{f_c}'$, [psi] $(0.83\sqrt{f_c}'$, [MPa]$)$ and $7.1\sqrt{f_c}'$, [psi] $(0.59\sqrt{f_c}'$, [MPa]$)$, respectively. The shear stress versus drift responses for both of these specimens, shown in Fig. 8, demonstrate good agreement with the predicted capacities. In addition, the excellent ductility, wide hysteresis loops, and good stiffness retention typical of previous HPFRC coupling beam tests were again observed. It should be noted that both specimens exhibited this response to drift levels exceeding 5% in both loading directions. Finally, the simplified connection detail utilizing dowel bars was effective at transferring the shear and moment between the precast HPFRC section and the wall and ensuring that plastic rotations do not localize at the beam-wall interface. This was all achieved without disrupting the wall boundary element reinforcement.

CONCLUSIONS

The tests of HPFRC coupling beams described herein demonstrate that HPFRC can significantly simplify the design and construction of coupling beams, while ensuring adequate ductility and damage tolerance. The following observations and conclusions have been drawn from the test results:

• Steel fiber HPFRC can be relied on to provide confinement of diagonal reinforcement. None of the coupling beams tested showed signs of diagonal bar instability. This significantly simplifies the detailing and construction of coupling beams.

• HPFRC contributes significantly to shear resistance. In the tests, HPFRC was estimated to carry shear stresses of approximately $5\sqrt{f'_c}$, $[psi]$ $(0.42\sqrt{f'_c}$, $[MPa])$, which allowed for a ductile flexural mechanism to develop. This high contribution to shear resistance reduces the required area of diagonal reinforcement appreciably without requiring excessive transverse reinforcement to resist the applied shear.

• Two alternative details were proven effective for adequately transferring moment and shear between the precast coupling beam and the adjacent structural wall without interfering with the wall boundary element. The first detail, consisting of U-shaped dowel bars, successfully forced rotations to localize away from the cold joint, as intended. The second detail, consisting of straight dowel bars, proved to be an even simpler alternative for achieving the same desired relocation of plastic deformations. From a construction viewpoint, the success of these connection details, which do not interfere with wall reinforcement, makes precast HPFRC a more attractive option for coupling beams than structural steel members.

• HPFRC coupling beams were shown to exhibit excellent ductility, often showing no significant strength loss up to 5% drift.

• The proposed design philosophy for HPFRC coupling beams was shown to result in a desirable and predictable behavior of the tested coupling beams.

6 in. (150 mm)

24 in. (600 mm)

66 in. (1650 mm)

#3 (D10) Stirrups @ 8in (200mm)
4 #5 (D16) Top and Bottom Bars
#6 (D20) Diagonal Bars

7.5 in. (188 mm)

(a) Specimen 1

6 in. (150 mm)

24 in. (600 mm)

66 in. (1650 mm)

#3 (D10) Stirrups @ 6.5in (163mm)
2 #5 (D16) Top and Bottom Bars
#6 (D20) Diagonal Bars
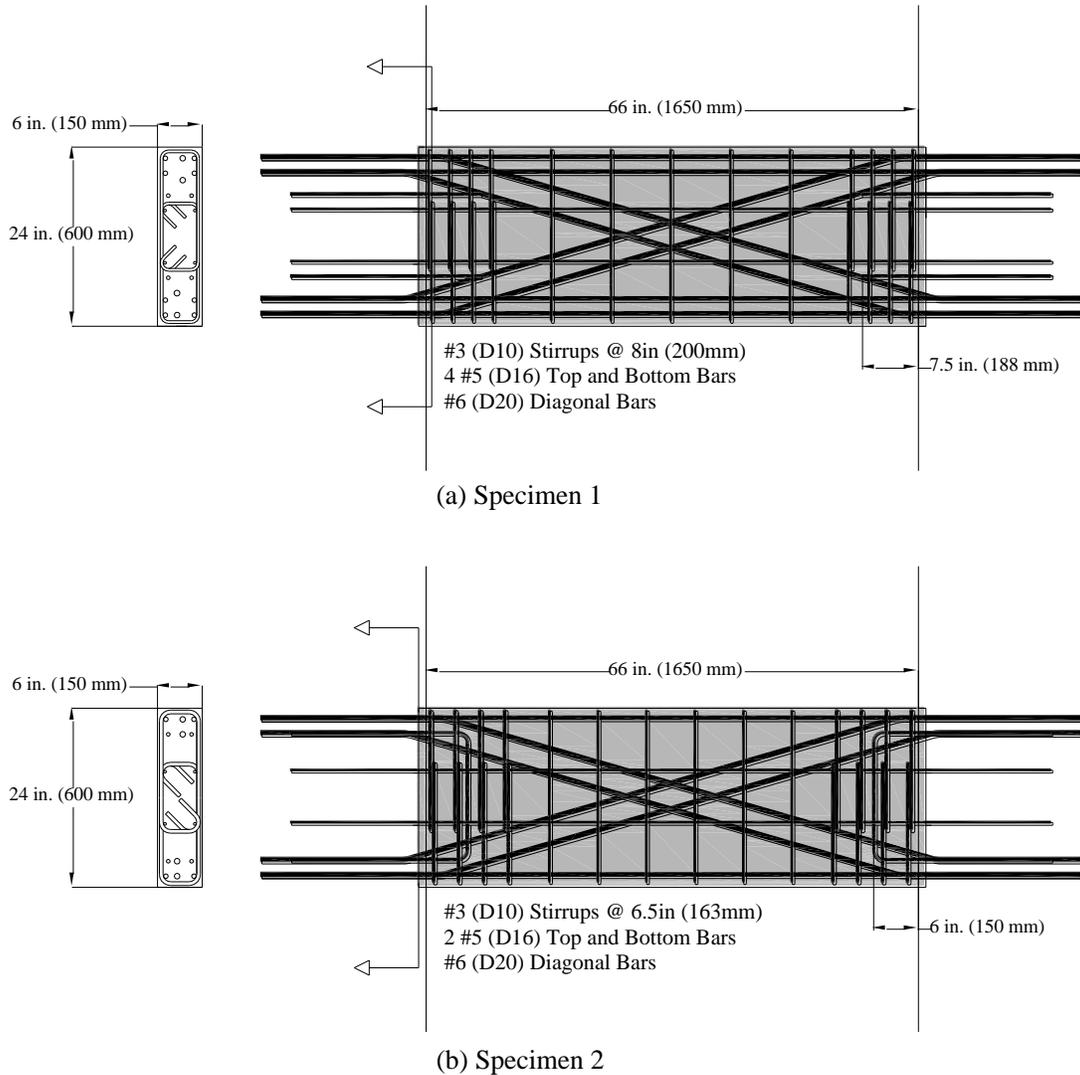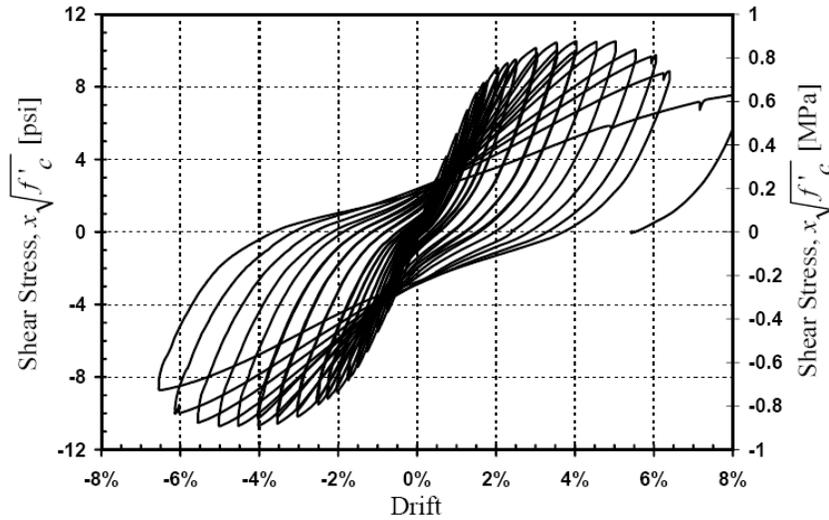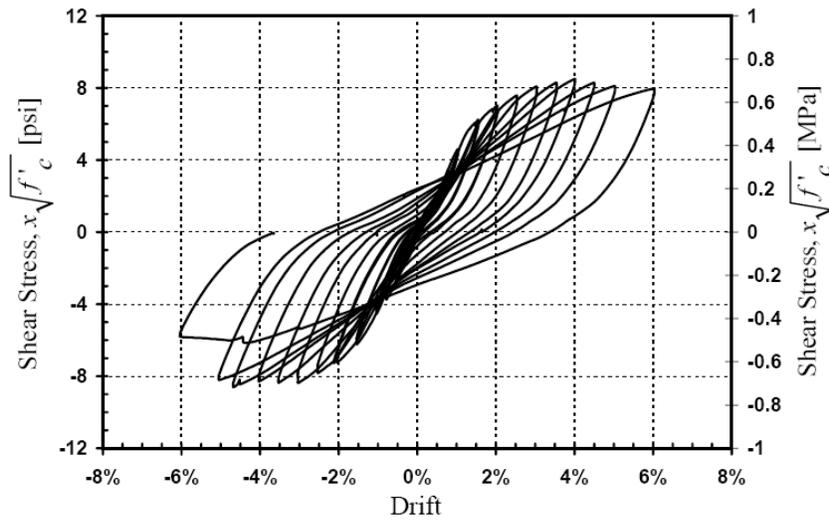
6 in. (150 mm)

(b) Specimen 2

Fig. 7 – Reinforcement layout for phase II coupling beams with $\ell_n / h = 2.75$

(a) Specimen 1

(b) Specimen 2

Fig. 8 – Shear stress vs. drift response for coupling beams with $\ell_n / h = 2.75$

REFERENCES

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

[2] Galano, L.; Vignoli, A., 2000, "Seismic Behavior of Short Coupling Beams with Different Reinforcement Layouts," *ACI Structural Journal*, V. 97, No. 6, pp. 876-885.

[3] Harries, K.; Gong, B.; and Shahrooz, B., 2000, "Behavior and Design of Reinforced Concrete, Steel, and Steel-Concrete Coupling Beams," *Earthquake Spectra*, V. 16, No. 4, pp. 775-799.

[4] Tassios, T.; Moretti, M.; and Bezas, A., 1996, "On the Behavior and Ductility of Reinforced Concrete Coupling Beams of Shear Walls," *ACI Structural Journal*, V. 93, No. 6, pp. 711-720.

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

[6] Canbolat, B.A., 2004, "Seismic Behavior of High-Performance Fiber Reinforced Cementitious Composite Coupling Beams," Ph.D. Dissertation, Report No. UMCEE 04-11, University of Michigan, Ann Arbor, MI, 193 pp.

[7] Parra-Montesinos, G.J., 2005, "High-Performance Fiber-Reinforced Cement Composites: An Alternative for Seismic Design of Structures," *ACI Structural Journal*, V. 102, No. 5, pp. 668-675.

[8] Chao, S.H., 2005, "Bond Characterization of Reinforcing Bars and Prestressing Strands in High Performance Fiber Reinforced Cementitious Composites Under Monotonic and Cyclic Loading," Ph.D. Dissertation, University of Michigan, Ann Arbor, MI, 475 pp.

[9] Paulay, T., 1971, "Coupling Beams of Reinforced Concrete Shear Walls," *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers*, V. 97, No. ST3, pp. 843-862.

[10] Luisoni, C.J.; Somenson, H.M.; Ungaro, M.A., 1970, "Verification Experimental de un Calculo Plastico y Otro Elastico do una Patrad de Corte," *XIV Jornades Sudamericanas de Ingerieri a Estructural: y IV Simposio Panamericano de Estructuras*, pp. 230-286.

[11] ACI Committee 318, 2008, "Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary," American Concrete Institute, Farmington Hills, MI, 465 pp.

[12] Harries, K. A.; Fortney, P. J.; Shahrooz, B. M.; Brienen, P. J., 2005, "Practical Design of Diagonally Reinforced Concrete Coupling Beams – Critical Review of ACI 318 Requirements," *ACI Structural Journal*, V. 102, No. 6, pp. 876-882.

[13] Liao, W.C.; Chao, S.H.; Park, S.Y.; and Naaman, A.E., 2006, "Self-Consolidating High Performance Fiber Reinforced Concrete (SCHPFRC) – Preliminary Investigation," Technical Report No. UMCEE 06-02, University of Michigan, Ann Arbor, MI, 68 pp.

[14] ASTM C 1609/C 1609M – 05, "Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam With Third-Point Loading)," West Conshohcken, PA., 2005.