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SEISMIC DETAILING AND BEHAVIOR OF COUPLED-WALL SYSTEMS WITH HIGH-PERFORMANCE FIBER-REINFORCED CONCRETE

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ABSTRACT

A series of tests has been conducted to investigate the use of strain-hardening, high-performance fiber-reinforced concrete (HPFRC) in the critical regions of coupled-wall systems. A component test phase, consisting of tests of three large-scale precast coupling beam specimens with aspect ratios of 1.75 subjected to reversed cyclic loading, was conducted to evaluate various reinforcement details for HPFRC coupling beams. A new design approach for HPFRC coupling beams, developed from these tests, has been shown to result in a ductile flexurally-dominated failure mode. These tests confirm that HPFRC can fully confine the diagonal reinforcement and demonstrate the significant contribution to shear capacity and damage tolerance that HPFRC provides. The viability of precasting the coupling beam and connecting it to adjacent structural walls without interfering with wall boundary reinforcement was demonstrated.

Subsequently, two large-scale, four-story, coupled-wall specimens, designed based on the conclusions drawn from the component test phase, were tested under lateral displacement reversals. Each coupled-wall consisted of four precast coupling beams linking two T-shaped reinforced concrete structural walls. The reinforcement details of the precast beams were slightly varied, allowing a comparison of the response of different detailing schemes when integrated into a coupled wall system. The second coupled-wall specimen also incorporated fiber reinforcement in the wall plastic hinge region, which allowed for a reduction in confinement reinforcement and a higher contribution from concrete to wall shear capacity. The response of both coupled-wall specimens showed good strength and stiffness retention, and energy dissipation beyond a system drift of 2.5%.

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. The efficiency of a structural wall system can be improved by proper coupling of two or more consecutive walls through the

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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 and strength.

For satisfactory performance of a coupled-wall system during a seismic event, the short coupling beams must retain a significant strength and stiffness through large displacement reversals. To ensure that adequate coupling beam ductility is achieved, the ACI Building Code (ACI 318-08) requires the use of diagonal reinforcement 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, Parra-Montesinos, and Wight 2005; Wight, Parra-Montesinos, and Lequesne 2009) have demonstrated that precasting coupling beams with strain-hardening, high performance fiber-reinforced concrete (HPFRC) can simplify the construction process without sacrificing overall performance under large displacement reversals. HPFRC coupling beams tested recently exhibited a ductile flexurally-dominated behavior, despite requiring significantly simpler reinforcement detailing.

The behavior of a coupled-wall system with precast HPFRC coupling beams, however, has not previously been studied experimentally. The tests described herein seek 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 imposed on each system component. In addition, the impact of reinforcing the wall plastic hinge region with fibers was evaluated in the second coupled-wall test. The inclusion of fiber reinforcement permitted a substantial reduction of the boundary element confinement reinforcement, simplifying construction, and allowed for a higher nominal shear stress to be resisted by the concrete, with no observed detriment to the response of the system.

Research Significance

An experimental investigation of the impact of using precast HPFRC coupling beams on the design, constructability, and seismic performance of coupled walls is presented. More specifically, this project seeks to: 1) demonstrate the ease with which precast coupling beams can be connected to cast-in-place structural wall systems, 2) evaluate the effectiveness of reduced boundary element confinement in HPFRC walls for increasing concrete ductility and providing longitudinal bar support, 3) compare the behavior of coupling beams with various details when subjected to similar deformation demands, and 4) study the interaction between HPFRC coupling beams, slabs, and structural walls.

Material Properties

Details on the development of the HPFRC mix design used in this study are reported by Liao et al. (2006). This HPFRC includes a 1.5% volume fraction of high-strength hooked steel fibers and coarse aggregate with a maximum nominal size of 0.5 in. (13 mm). The hooked steel fibers used are 1.2 in. (30 mm) long, have an aspect ratio of 80, and are manufactured out of high strength steel with a tensile strength of 330 ksi (2300 MPa). The conventional concrete used

throughout this study was either mixed in the laboratory or delivered by local suppliers, and had a specified compressive strength (f'_c) of 6 ksi (41 MPa). Results from compressive tests of 4 in. by 8 in. (100 mm by 200 mm) cylinders and of bending tests performed in accordance with ASTM C1609–05, have been reported previously (Lequesne, Parra-Montesinos and Wight 2009). The bending tests of the HPFRC showed pronounced deflection hardening behavior, with peak bending stresses that exceeded the first cracking stress by more than 30% occurring near deflections of L/800.

Coupled-Wall System Tests

Two four-story coupled wall specimens were built at approximately one-third scale and subjected to earthquake-type lateral displacement reversals. A completed specimen and testing setup is shown in Fig. 1. Each of the coupling beams had a slightly different reinforcement detail, allowing 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 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 specified 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. Steel tube sections embedded through each wall above the second story slab transferred force from the external tendons into the walls. Hydraulic jacks were used to apply this vertical force before any lateral displacement was 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.

Lateral displacements were pseudo-statically applied 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.

HPFRC Coupling Beams

The design approach for the HPFRC coupling beams used in the coupled-wall tests was developed through two series of tests on HPFRC coupling beam components (Canbolat, Parra-Montesinos, and Wight 2005; Wight, Parra-Montesinos, and Lequesne 2009). The most recent

series of tests involved three large-scale precast HPFRC coupling beams with an aspect ratio (ℓ_n/h) of 1.75. A sample result from this series of tests is shown in Fig. 2. The results indicate that HPFRC contributes appreciably to the shear capacity of the coupling beam, provides confinement to diagonal reinforcement, and results in improved damage tolerance evidenced by reduced crack widths and wide shear force vs. member drift hysteresis loops. Two different connection details were evaluated for transferring moment and shear between the precast section and the adjacent structural walls without interfering with wall boundary reinforcement. The connection details involved either U-shaped or straight dowel bars near mid-depth, which successfully forced flexural rotations to localize away from the beam-wall interface. This permitted the precast HPFRC section to penetrate into the wall face only as deeply as the cover, thus allowing wall boundary reinforcement to continue, uninterrupted.

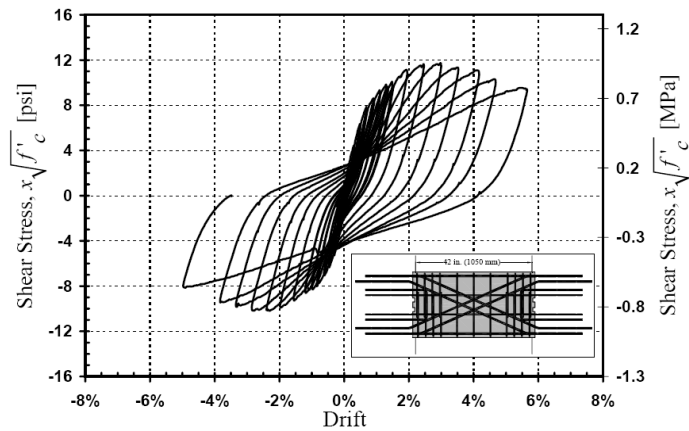Figure 1.   Photo of test setup

Figure 2.   Shear stress vs. drift for coupling beam

Based on these component tests, a design approach for HPFRC coupling beams has been proposed that represents a major change from the current state of the art. This new approach accounts for the improved damage tolerance exhibited by HPFRC elements by assuming the full coupling beam section maintains its integrity and remains active in resisting shear and moment through large displacement reversals, reducing the need for diagonal and transverse reinforcement for resisting applied shear. This is in contrast to the current approach which assumes the full beam capacity is uniquely controlled by two intersecting diagonal reinforcement cages.

Accordingly, the following approach for HPFRC coupling beam design was adopted for the beams used in the coupled-wall systems, given an expected beam shear demand, V_u, and a corresponding moment demand, M_u ($M_u = V_u \cdot L/2$, where L is the clear span of the beam).

1) Select cross-sectional beam dimensions that satisfy architectural requirements while ensuring that $V_u \leq 10\sqrt{f'_c} A_{cw}$, [psi] ($0.83\sqrt{f'_c} A_{cw}$, [MPa]), where f'_c is the specified concrete compressive strength concrete cylinder strength, and A_{cw} is the cross-sectional area of the coupling beam.

2) Assume diagonal reinforcement resists approximately 30-40% of the expected shear demand, V_u, and select appropriate diagonal bar orientation angle and bar sizes.

3) From Step 2, the contribution of the diagonal reinforcement to the beam probable moment strength, M_{pr}, is determined. Select flexural reinforcement, A_s, to ensure $\phi M_{pr} \geq M_u$. This procedure is a capacity-based approach, so $\phi = 1.0$ is recommended.

4) Place a reasonable fraction of this longitudinal flexural reinforcement as short dowel bars to ensure failure will not localize at the precast beam-wall interface.
   - Based on the beam components tested, "reasonable fraction" corresponds to:
      - 30-40% of A_s for short coupling beams
      - 15-20% of A_s for longer coupling beams

5) In this project, a moment-curvature analysis was used to verify that $\phi M_{pr} \geq M_u$, and the flexural design was modified if required. For this moment-curvature analysis, the constitutive model selected for the HPFRC has an important impact on the results.

6) Shear design: Use the probable coupling beam moment capacity, M_{pr}, to determine the expected shear demand, V_{pr}. The HPFRC coupling beam shear capacity, V_n, should be designed to satisfy $\phi V_n \geq V_{pr}$, where $\phi = 0.85$. For HPFRC coupling beams, $V_n = V_d + V_s + V_c$, where V_d is the contribution from the diagonal reinforcement, V_s is the contribution from the stirrups, which is suggested to be 30-40% of V_{pr}, and V_c is the contribution from the HPFRC. $V_c \leq 5\sqrt{f'_c}A_{cw}$, [psi] $(0.42\sqrt{f'_c}A_{cw}$, [MPa]) appears to be appropriate, based on test results.

7) Detailing: Provide column-type confinement to the ends of the coupling beam, extending approximately $h/2$ away from the face of the walls, where h is the coupling beam height. Due to the confinement provided by the HPFRC, no special confinement reinforcement is required for the remaining portion of the span.
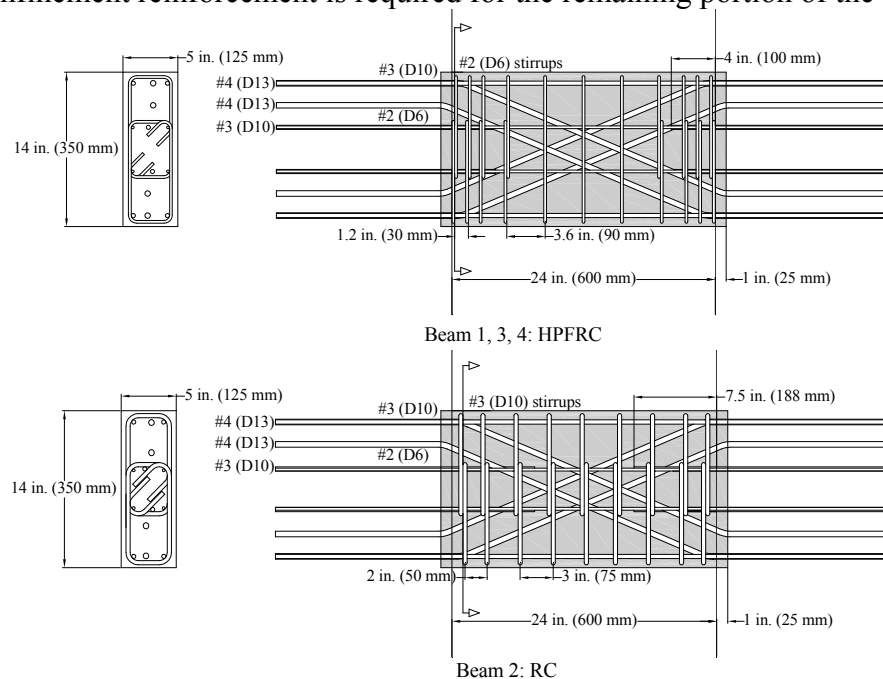

Figure 3.   Coupling beam reinforcement

Typical reinforcement details used in the coupling beams of the test specimens are shown in Fig. 3. The first, third, and fourth story coupling beams were constructed with HPFRC, and the second story beam in both specimens was cast with conventional concrete. The flexural and diagonal reinforcement were identical for the HPFRC and RC coupling beams to provide similar stiffnesses and flexural capacities. However, the transverse reinforcement ratio was increased from 0.55% for the HPFRC coupling beams to 1.5% to provide additional shear resistance and confinement to the RC beam.

The flexural reinforcement of the coupling beams used in the first coupled-wall system was terminated 3 in. (75 mm) into the wall to be more consistent with current design requirements. This resulted in an undesirable localization of damage along the interface between the structural wall and precast coupling beam section, and is not recommended. All of the coupling beam reinforcement was fully developed in the second coupled-wall system, which resulted in damage occurring within the precast section and full development of the coupling beam capacity.

Coupled-Wall Design

The coupling beam dimensions and detailing were of special interest in this project, and thus, they were the initial focus of the coupled wall design. The structural walls in the first coupled-wall specimen were subsequently designed in accordance with the ACI Building Code (ACI 318-08) to provide the required overturning moment capacity, shear strength, and ductility for the coupled-wall system to behave realistically. Fiber reinforcement was used in the first two stories of the second coupled-wall system, as shown in Fig. 4. The HPFRC contributed to confinement of the wall boundary reinforcement, which allowed for an increase in the spacing of the hoops from $b_w/3$. Stirrups were spaced at $b_w/2$ in one HPFRC wall, and at b_w in the other.

The wall shear design was based on the expected ultimate capacity of the system, assuming that a mechanism controlled by flexural hinging in the base of both walls and in each of the beams would develop. To resist the expected shear demand, the wall concrete in the first coupled-wall system was assumed to carry a shear stress of $2\sqrt{f'_c}$, [psi] $(0.17\sqrt{f'_c}$, [MPa]), and wall transverse (horizontal) reinforcement, anchored by alternating 90- and 135-degree hooks, was provided to resist the remaining shear. For the RC walls the transverse reinforcement ratio was 0.45%. The concrete in the second coupled-wall system, which was fiber reinforced, was assumed to carry a shear stress of $4\sqrt{f'_c}$, [psi] $(0.34\sqrt{f'_c}$, [MPa]), resulting in the same wall transverse reinforcement ratio, despite an increased shear stress demand. This higher shear demand was a result of the higher overturning moment capacity of the second specimen caused by full development of the coupling beam reinforcement.

Coupled Wall Test Results

A plot of the overturning moment vs. drift responses of the coupled-wall specimens is shown in Fig. 5. Both specimens exhibited the high strength and stiffness characteristic of coupled-walls, with excellent strength retention. The first specimen maintained more than 80%

of the peak overturning moment capacity to beyond 2.5% drift in both directions. The second specimen did even better, retaining more than 80% of the peak capacity to beyond 3% drift. Furthermore, the hysteresis loops show no appreciable pinching in either test, because the responses were governed by flexural hinging in the bases of the walls and at the ends of the coupling beams, as intended in design.

Figure 4.   Coupled-wall system reinforcement

Assuming this flexurally-dominated plastic mechanism developed, and accounting for the measured concrete and reinforcing steel properties, the system overturning moment capacity for the first specimen was under-predicted by only approximately 5%. The design coupling ratio of 0.4 was attained at approximately 0.75% drift, and largely maintained until the test was terminated when the integrity of the structure had been compromised by the fracturing of diagonal reinforcement in the coupling beams and flexural reinforcement in the walls. Similar predictability was exhibited by the second coupled-system. The second test was terminated once the compression wall failed in shear at a system drift of 3%, and the opposing wall exhibited a flexural failure upon load reversal at a drift of 3.3%. Throughout this paper, "compression" wall is used to refer to the wall that, due to the coupling action of the beams, was subjected to increased compression within a particular loading half-cycle.

Both specimens exhibited predominantly diagonal-shear cracking in the base of the walls in early cycles, with flexural cracking becoming more prevalent at larger drifts. This indicates that the ACI Building Code requirements for structural wall design provided adequate shear resistance and confinement of longitudinal reinforcement to accommodate a ductile flexural mechanism in the first specimen. Likewise, the stable response of the second specimen indicates that the complement of reduced reinforcement and HPFRC also provided adequate shear resistance and confinement of longitudinal reinforcement. Furthermore, the second specimen, with fiber reinforced walls, exhibited a much higher number of cracks spaced at approximately 2 in. (50 mm), less than half of the typical diagonal crack spacing in the first specimen. This is typical of HPFRC members, and results in greater damage tolerance.
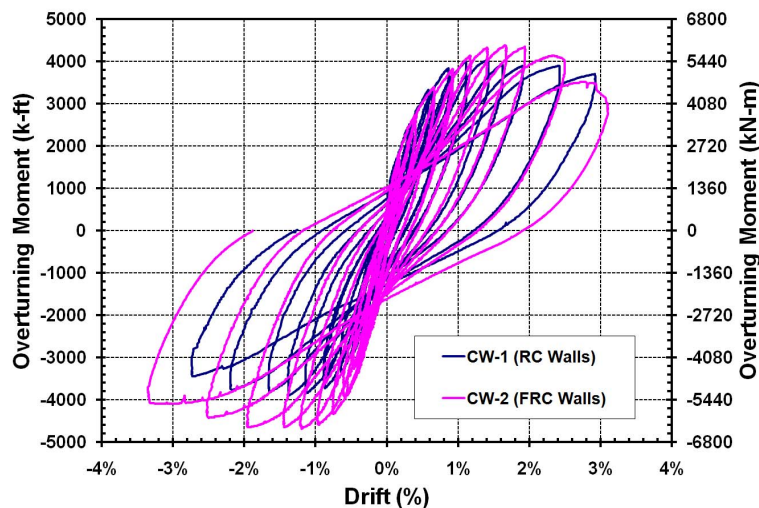
Figure 5.   Overturning moment vs. drift for coupled-wall specimens

Coupling-Beam Performance

In both specimens, diagonal-shear cracking was first observed in all four coupling beams near system drifts of 0.5%. As drifts increased in the first coupled-wall system, damage began to localize near the precast beam-wall interface due to the termination of coupling beam flexural reinforcement only 3 in. (75 mm) into the wall. The result was crushing and spalling of the wall concrete near the interface at system drifts exceeding 2.5%, which precluded the development of a more desirable damage pattern within the precast beams.

The second specimen, with the fully developed coupling beam reinforcement, showed coupling beam damage localizing within the precast beam away from the wall connection. This allowed for a better comparison of HPFRC coupling beam reinforcement details. Fig. 6 shows a photo of the first two coupling beams at a system drift of 3.3%, and illustrates the substantial improvement in damage tolerance achieved by HPFRC beams relative to the RC coupling beam.

Coupling Beam Chord Rotations

For typical coupled-wall systems, the chord rotation expected in coupling beams (as defined in ASCE/SEI 41/06), can be roughly predicted as the coupled-wall rotation times the

ratio of the distance between the wall centroidal axes to the effective length of the coupling beam (2.1 for the test specimens). The effective coupling beam length, L_{eff}, was defined as $L_{eff} = L + h$, where L is the clear span and h is the height of the beam. The experimentally observed relationship between coupling beam chord rotations and inter-story wall rotations are shown in Fig. 7, plotted along with the predicted value. In both systems tested, when the system drift exceeded 2.5%, nearly all of the coupling beams were subjected to chord rotations exceeding 4.0% (ratio = 1.6). Although these large deformation demands emphasize the need for highly ductile coupling beams, the observed ratio is lower than the calculated value of 2.1, indicating that this rough predictor may over-estimate coupling beam chord rotation demands.
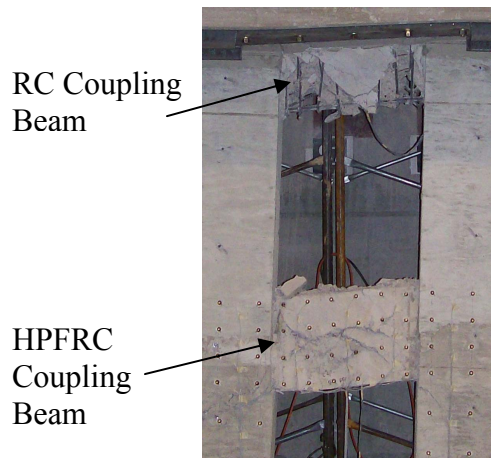
RC Coupling Beam

HPFRC Coupling Beam
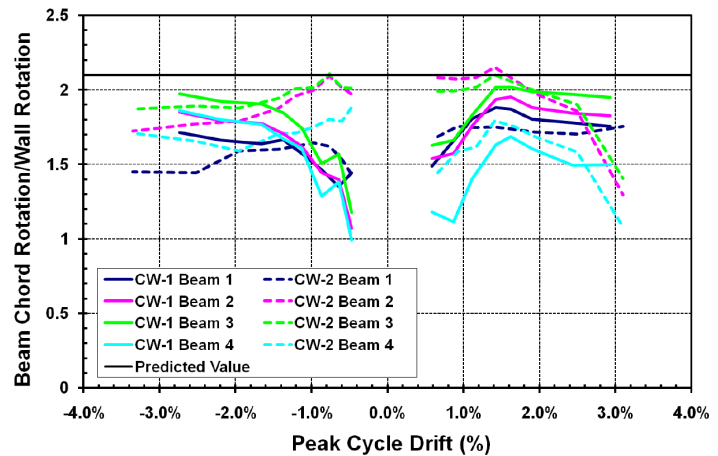

Figure 6.   CW-2 beam damage

Figure 7.   Coupling beam chord rotations

Summary and Conclusions

Two large-scale four-story coupled-wall specimens were tested to investigate the impact that precast HPFRC coupling beams have on the design, construction, and behavior of coupled-wall systems. The design of the specimens was based on results from a series of coupling beam component tests that demonstrated the utility of HPFRC in coupling beam design. The following conclusions can be made regarding the performance of the systems tested.

- The coupled-wall specimens exhibited the high strength and stiffness characteristic of coupled-walls, with excellent strength retention beyond system drifts of 2.5%. The HPFRC regions exhibited narrower crack spacing and significantly improved damage tolerance, despite simplified detailing.
- Placing the precast coupling beams, with beam reinforcement threading through adjacent wall reinforcement, proved to be simple and is believed to be a viable alternative method for assembling a coupled-wall system.
- Terminating longitudinal reinforcement within the wall near the coupling beam-wall interface is not recommended. This detail caused a concentration of damage along the precast/cast-in-place interface in the first specimen, a damage pattern that was not observed in the second system with fully developed reinforcement.
- Coupling beam chord rotations, relative to the inter-story wall rotations, were generally less than predicted, implying that coupling beam chord rotations may generally be over-estimated by existing procedures.

- Spacing the wall boundary element ties at $b_w/2$ or b_w, as compared to $b_w/3$ for the RC walls, did not lead to premature failure in the HPFRC walls, indicating that effective confinement was achieved.
- A design value of $4\sqrt{f'_c}$, [psi] ($0.34\sqrt{f'_c}$, [MPa]) for the contribution of HPFRC to the nominal shear stress capacity of structural walls seems appropriate.
- Although not a focus of the current study, the ACI Building Code requirements for structural wall design provided adequate shear resistance and confinement of longitudinal reinforcement to allow for a stable flexural mechanism to develop.

Acknowledgments

References

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

ASCE/SEI 41/06, 2007. *Seismic Rehabilitation of Existing Buildings*, American Society of Civil Engineers, Reston, Virginia.

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

Canbolat, B. A.; Parra-Montesinos, G. J.; and Wight, J. K., 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.

Lequesne, R. D.; Parra-Montesinos, G. J.; and Wight, J. K. 2009. "Test of a Coupled Wall with High Performance Fiber Reinforced Concrete Coupling Beams," *Thomas T. C. Hsu Symposium: Shear and Torsion of Concrete Structures,* SP-265, American Concrete Institute, Farmington Hills, MI, 16 pp.

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

Wight, J. K., Parra-Montesinos, G. J. and Lequesne, R. D. (2009), "The Design of Coupled Wall Systems for Earthquake Motions with High-Performance Fiber Reinforced Concrete." *Proceedings of WCCE-ECCE-TCCE Joint Conference Earthquake & Tsunami*, Istanbul, Turkey, June 22-24, 13 pp.