Reference:
Lequesne, R. D., Setkit, M., Kopczynski, C., Ferzli, J., Cheng, M.-Y., Parra-Montesinos, G. J., & Wight, J. K. (2011). Implementation of High-Performance Fiber Reinforced Concrete Coupling Beams in High-Rise Core-Wall Structures. In SP-280: Advances in FRC Durability and Field Applications. Farmington Hills, MI: American Concrete Institute. 12 pp.

Implementation of High-Performance Fiber Reinforced Concrete Coupling Beams in High-Rise Core-Wall Structures in the Seattle Area

by Rémy Lequesne, Monthian Setkit, Cary Kopczynski, Joe Ferzli, Min-Yuan Cheng, Gustavo Parra-Montesinos, and James K. Wight

Synopsis:
Experimental and analytical studies that led to the incorporation of strain-hardening, high-performance fiber reinforced concrete (HPFRC) coupling beams in the design of a high-rise core-wall structure in Seattle, WA, are described. A total of eight HPFRC coupling beams with span-to-depth ratios ranging between 1.75 and 3.3 were tested under large displacement reversals. The tension and compression ductility of HPFRC materials allowed an approximately 70% reduction in diagonal reinforcement in beams with a 1.75 aspect ratio and a total elimination of diagonal bars in beams with a 2.75 and 3.3 aspect ratio. Further, special column-type confinement reinforcement was not required except at the ends of the beams. When subjected to shear stress demands close to the upper limit in the 2008 ACI Building Code ($0.83\sqrt{f_c'}$ [MPa] $\left(10\sqrt{f_c'}\text{ [psi]}\right)$) the coupling beams with aspect ratios of 1.75, 2.75 and 3.3 exhibited drift capacities of approximately 5%, 6% and 7%, respectively.

The large drift and shear capacity exhibited by the HPFRC coupling beams, combined with the substantial reductions in reinforcement and associated improved constructability, led Cary Kopzcynski & Co. to consider their use in a 134 m (440 ft) tall reinforced concrete tower to be constructed in the city of Seattle, WA. Results from inelastic dynamic analyses indicated adequate structural response with coupling beam drift demands below the observed drift capacities. Also, cost analyses indicated 20-30% savings in material costs, in addition to much easier constructability and reduced construction time.

Keywords: structural walls, coupled walls, coupling beams, shear, drift, fiber reinforced concrete, steel fibers, precast

ACI member **Rémy Lequesne** is a PhD student at the University of Michigan, Ann Arbor. He was the 2007-2008 ACI Pankow Foundation Fellowship recipient, and is an Associate Member of ACI Committee 374, Performance-Based Seismic Design of Concrete Buildings. His research interests include the behavior and design of reinforced concrete members, the mechanical and structural behavior of high-performance fiber-reinforced concrete, and the earthquake resistant design of reinforced concrete structures.

ACI member **Monthian Setkit** is a PhD student at the University of Michigan, Ann Arbor. His research interests include the behavior and design of reinforced concrete, the earthquake-resistant design of reinforced concrete structures, and fiber- reinforced concrete structures.

ACI member **Cary Kopczynski**, P.E., S.E., FACI, is CEO of Cary Kopczynski & Company, Inc. P.S. (CKC), a structural engineering firm based in Bellevue, Washington. CKC provides leading edge structural designs for major buildings throughout the United States and beyond. Kopczynski served on the ACI Strategic Planning Committee, is a member of ACI Committee 318, Structural Concrete Building Code, and a past member of ACI 352 and ACI 368. He chairs the Technical Advisory Board of the Post-Tensioning Institute and is a past President of the Washington Chapter of ACI. Mr. Kopczynski was also selected by ENR magazine as one of the Top 25 Newsmakers in 2007 based on his pioneering work in the field of high-strength steel and reinforcing bar.

ACI Member **Joe Ferzli**, P.E., S.E. is a Senior Associate at Cary Kopczynski & Company, Inc. P.S. (CKC), a structural engineering firm based in Bellevue, Washington. He is a registered professional engineer with more than 12 years' experience in a wide range of projects. He received his B.S. and M.S. degree in Civil Engineering from the University of Oklahoma. He chairs the Educational Committee of SEAW.

ACI Member **Min-Yuan Cheng** is Assistant Professor at National Taiwan University of Science and Technology in Tapei, Taiwan. He received his Ph.D. degree in Civil Engineering from the University of Michigan, his M.S. degree in Civil Engineering from Cheng Kung University in Taiwan, and his B.S. degree in Marine Engineering from Sun-Yat-Sen University, also in Taiwan.

ACI member **Gustavo J. Parra-Montesinos** is an Associate Professor of civil engineering at the University of Michigan, Ann Arbor. He is Chair of ACI Committee 335, Composite and Hybrid Structures; and a member of ACI Committee 318, Structural Building Code, and Joint ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures. His research interests include the behavior and design of reinforced concrete, fiber reinforced concrete, and hybrid steel-concrete structures.

James K. Wight, FACI, is a Professor of civil engineering at the University of Michigan, Ann Arbor. He is Vice-President of ACI, Past Chair of ACI Committee 318, Structural Concrete Building Code, and ACI Committee 318-E, Shear and Torsion, and is a member of Joint ACI-ASCE Committees 352, Joints and Connections in Monolithic Concrete Structures, and 445, Shear and Torsion. His research interests include the earthquake-resistant design of reinforced concrete structures and the use of high-performance fiber-reinforced concrete in critical members or regions of such structures.

INTRODUCTION

Medium- and high-rise structures in regions of high seismicity typically rely on structural walls for lateral stiffness and strength. In order to maximize space usage flexibility, it is common practice to layout all structural walls around the elevators, forming what is referred to as a core wall. Access to the elevator halls as well as utility conduits, however, prevent these walls from being continuous along the core perimeter, leading to the splitting of an otherwise solid wall into several walls connected by relatively short and deep "coupling" beams.

The design of coupling beams, with span-to-depth ratios that often range between 1.5 and 3.5, requires special attention because of the large inelastic rotations and shear stresses coupling beams can be subjected to during a strong earthquake. In order to ensure adequate seismic performance, design provisions for coupling beams in the ACI Building Code (ACI Committee 318, 2008) include the use of diagonal reinforcement designed to resist the entire shear demand together with special column-type transverse reinforcement confining either the diagonal bars or the entire member. Fig. 1 shows a photo of a diagonally reinforced coupling beam. Although results from tests indicate that well-confined, diagonally reinforced coupling beams behave well under large displacement reversals (Paulay and Binney, 1974; Naish et al., 2009), the construction of these coupling beams is difficult and costly. Further, the need for large diameter bars (often No. 43M (No. 14) or even No. 57M (No. 18) and closely spaced transverse reinforcement at the wall boundaries create interference problems that require careful consideration.

In relatively slender coupling beams (i.e. beam span-to-depth ratio on the order of 3), current practice of using diagonal reinforcement to resist the entire shear demand becomes questionable due to the very shallow angle between the diagonal bars and the beam longitudinal axis (less than 15 degrees). At such shallow angles with the beam axis, diagonal reinforcement contributes little to shear resistance and thus, excessive amounts of diagonal steel are needed in highly-stressed slender coupling beams, with the associated increased reinforcement congestion and cost. Results from recent experimental research (Naish et al., 2009), however, indicate that diagonal reinforcement is needed in slender coupling beams to ensure adequate deformation capacity under large shear reversals.

In the past several years, research has been conducted at the University of Michigan to simplify the design of coupling beams without compromising their seismic performance. The simplification of reinforcement detailing translates not only into less material, but also substantial savings in labor and construction time. The approach followed consists of the use of high-performance fiber reinforced concrete (HPFRC) for increased shear resistance and confinement, which allows a reduction in both diagonal and transverse bar reinforcement. This is possible because of the post-cracking tensile hardening behavior of HPFRC materials, as well as their compression behavior that resembles that of well-confined concrete.

Recognizing the advantages of using HPFRC coupling beams from both a performance and economic viewpoint, Cary Kopczynski & Co., a design firm in Bellevue, WA, together with researchers at the University of Michigan, embarked in an effort to incorporate HPFRC coupling beams in high-rise building structures on the west coast. A summary of experimental and analytical work that led to the incorporation of HPFRC coupling beams in the design of a high-rise core-wall structure in Seattle, WA is presented in this paper.

Figure 1 - Diagonally reinforced coupling beam

RESEARCH SIGNIFICANCE

Results from experimental and analytical studies aimed at incorporating high-performance fiber reinforced concrete (HPFRC) coupling beams in earthquake-resistant high-rise core-wall construction are presented. Particular emphasis is placed on the application, for the first time, of HPFRC coupling beams in a high-rise structure in the Seattle area. Information presented is therefore useful not only to researchers but also to practicing engineers involved in the design of high-rise structures in earthquake-prone regions.

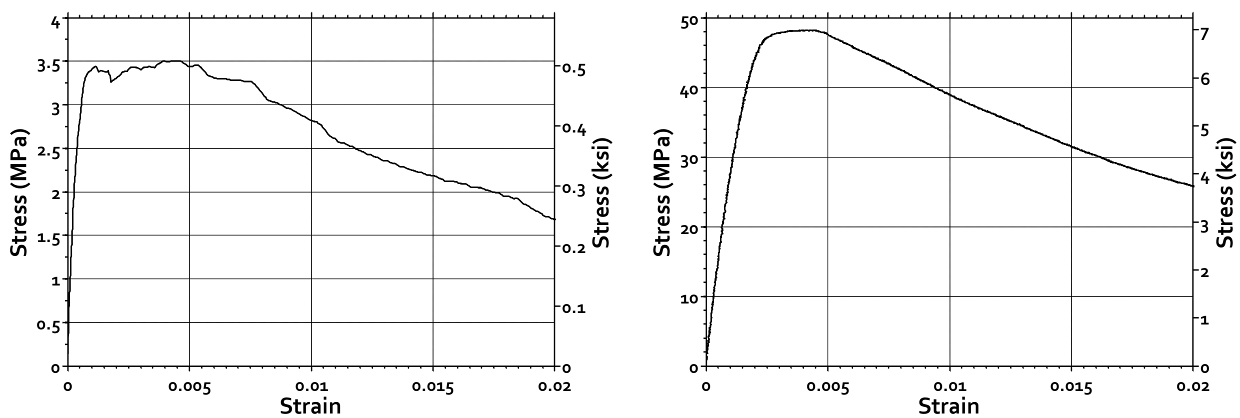
HPFRC PROPERTIES

As mentioned earlier, the tensile behavior of HPFRC materials is characterized by a strain-hardening behavior when subjected to direct tension. The HPFRC materials used in this investigation contained high-strength (2300 MPa or 330 ksi) hooked steel fibers in a 1.5% volume fraction. These fibers are 30 mm (1.2 in.) long and 0.38 mm (0.015 in.) in diameter. Mix proportions by weight for regular strength (41 MPa or 6 ksi) and high-strength (69 MPa or 10 ksi) concrete matrices (used in Specimens 1-4 and 5-8, respectively) were as shown in Table 1. Course aggregate consisted of crushed limestone with a maximum size of 13 mm (0.5 in.). A typical tensile stress-strain response obtained from direct tensile tests on "dog-bone" shaped specimens is shown in Fig. 2a.

Table 1 - HPFRC matrix mixture proportions by weight

	Cement (Type III)	Fly Ash	Sand	Agg.	Water	High-Range Water Reducer	Viscosity Modifying Agent
Regular Strength (41 MPa)	1	0.88	2.2	1.2	0.8	0.005	0.038
High Strength (69 MPa)	1	0.25	1.4	0.83	0.5	0.008	0.0083

In addition to improving material tensile response, fibers also provide confinement to the concrete and therefore, ductility. A representative compression stress-strain response obtained from cylinder tests is shown in Fig. 2b. As can be seen, the response was characterized by a shallow post-peak response with strain capacities greater than 1%.



a) Tensile response **b) Compression response**

Figure 2 - HPFRC stress versus strain response

TESTS OF HPFRC COUPLING BEAMS

Eight large-scale HPFRC coupling beams were tested under large displacement reversals to develop standard designs for various beam span-to-depth ratios. For this purpose, coupling beams with aspect ratios of 1.75, 2.75 and 3.3 were tested at the University of Michigan Structural Engineering Laboratory. Fig. 3 shows a sketch of the setup used in the experimental investigation. As shown in the figure, the test specimens consisted of an HPFRC beam connected to two stiff reinforced concrete blocks simulating structural walls. For testing convenience, the coupling beam-wall subassemblages were rotated 90 degrees. Beam rotations similar to those in an actual coupling beam were imposed by applying lateral displacements at the top block while maintaining the top and bottom blocks parallel through the use of vertical steel links. These steel links also provided some axial restraint to the coupling beam, which aimed to simulate that provided by structural walls and slabs to coupling beams in an actual coupled wall structure.

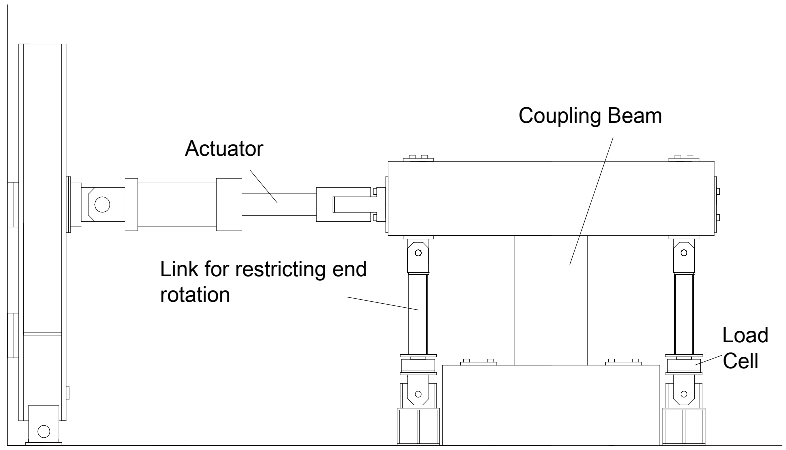


Figure 3 - Test setup

The coupling beams with aspect ratios of 1.75 and 2.75 had a 150 mm x 600 mm (6 in. x 24 in.) cross section, while the beams with a 3.3 aspect ratio had a 150 mm x 500 mm (6 in. x 20 in.) cross section. A summary of the main features of each test coupling beam is provided in Table 2. All test coupling beams with a 1.75 aspect ratio were reinforced with diagonal reinforcement to resist approximately 25-30% of the expected shear demand. The coupling beams with aspect ratios of 2.75 and 3.3 were tested both with and without diagonal reinforcement. Because of the large rotation demands at the beam ends, special column-type confinement was provided over half the beam depth from the wall faces. Transverse reinforcement in the remaining part of the beam consisted of single hoops.

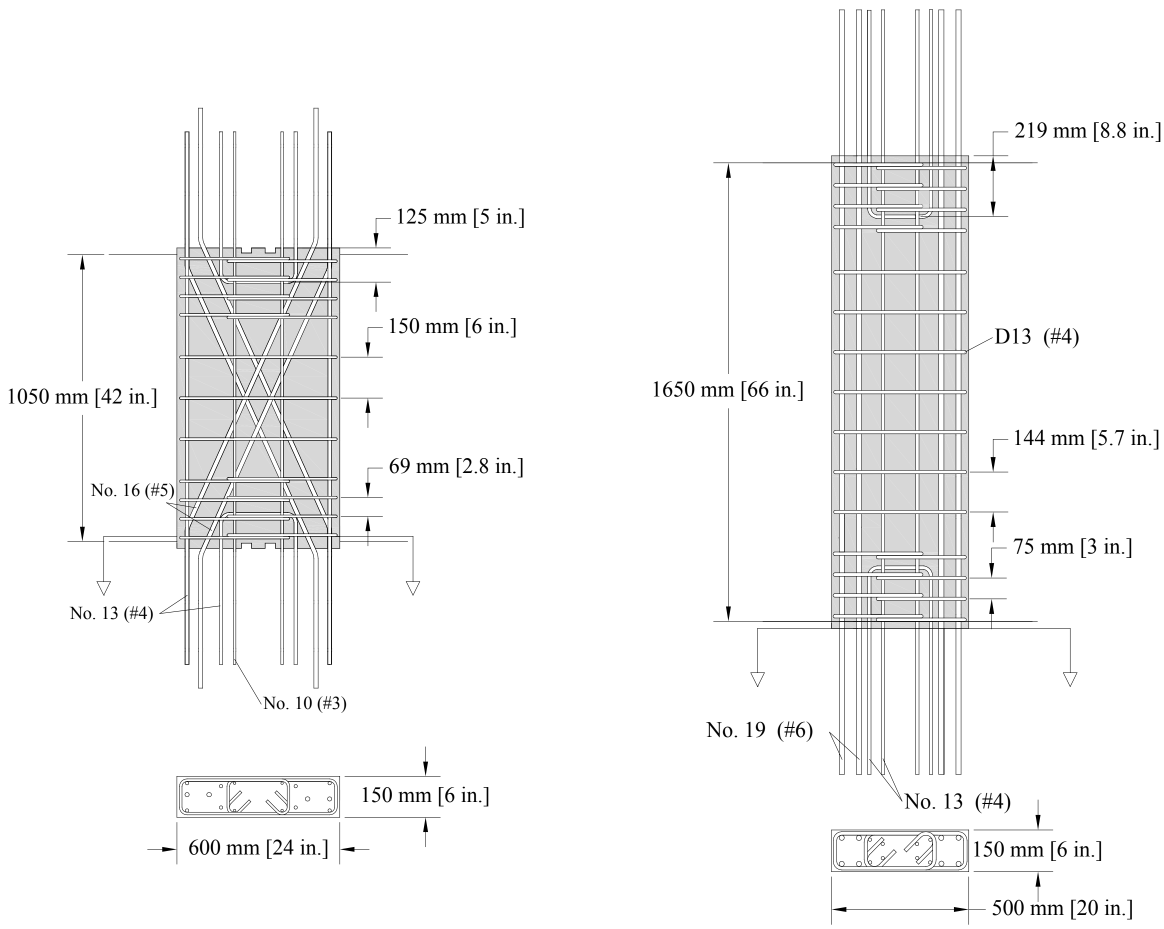
The reinforcement detailing in Specimens 2 and 8, with aspect ratios of 1.75 and 3.3, respectively, are shown in Fig. 4. These designs are proposed for use in short (aspect ratios between approximately 1.5 and 2.5) and slender (aspect ratios greater than 2.5) coupling beams. The substantial reduction in diagonal reinforcement for short coupling beams allows the use of smaller diameter bars which can be bent such that they enter the wall horizontally, allowing concrete in the walls to be cast up to the bottom level of the coupling beam prior to placing the beam reinforcement.

Table 2 - Summary of HPFRC test coupling beams

Specimen	Aspect ratio	Diagonal reinforcement (% shear strength)	Confinement volumetric ratio		Peak shear stress demand, MPa (psi)	Drift capacity (%)
			End*	Middle		
1	1.75	23	1.8%	0.89%	7.1 (1030)	2.5
2	1.75	27	2.9%	0.89%	7.0 (1020)	5.0
3	1.75	27	2.9%	0.67%	7.0 (1010)	5.0
4	2.75	25	4.8%	0.46%	6.2 (897)	5.8
5	2.75	30	4.1%	0.56%	5.5 (803)	5.8
6	2.75	0	4.4%	1.16%	6.0 (877)	6.5
7	3.3	25	4.4%	0.61%	6.6 (959)	5.2
8	3.3	0	4.4%	1.16%	6.7 (965)	7.0

*Over a distance of half the beam depth from the wall face.

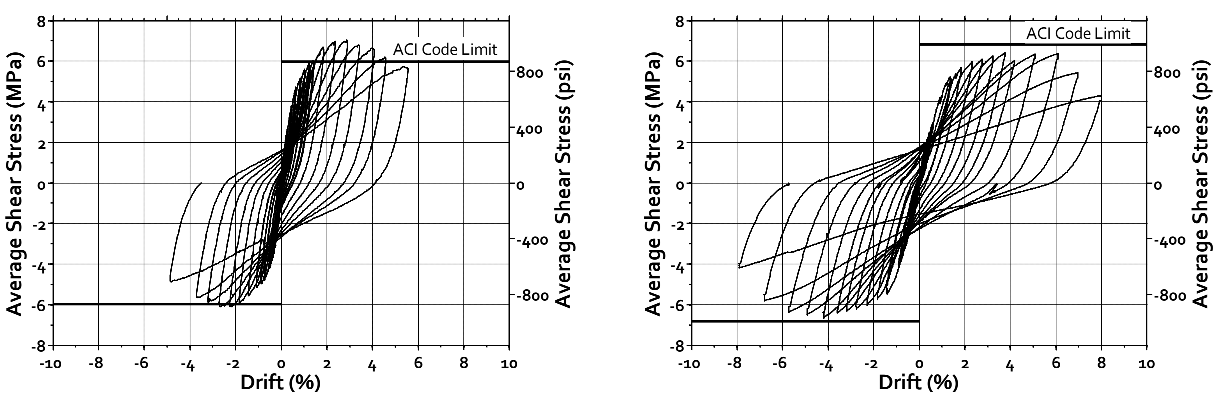
In order to further simplify the construction of the proposed HPFRC coupling beams, the designs were developed such that a precast coupling beam construction process could be followed. The use of precast coupling beams would allow greater quality control and quick assemblage in the field. Unlike steel coupling beams, the precast portion of the coupling beam would extend only into the wall cover to avoid interference with the wall reinforcement (see beam shaded region in Fig. 4). As can be seen in Fig. 4, the beam longitudinal and bent diagonal (if any) reinforcement extends into the wall for full anchorage. In order to strengthen the interface between the precast coupling beam and the wall and force inelastic deformations to occur within the beam, intermediate dowel reinforcement extending approximately half the beam depth into the span is provided.



a) Specimen 2 **b) Specimen 8**

Figure 4 – Reinforcement detailing for short and slender HPFRC coupling beam specimens

The average shear stress versus drift response obtained from the tests of coupling beams with aspect ratios of 1.75 (Specimen 2) and 3.3 (Specimen 8) is shown in Fig. 5. As can be seen, both coupling beams were subjected to peak shear stresses close to the upper stress limit in the ACI Building Code (ACI Committee 318, 2008) (i.e. $0.83\sqrt{f_c'}$ [MPa] $\left(10\sqrt{f_c'}\text{ [psi]}\right)$. Drift capacity for the shorter beam was on the order of 5%, while that for the slender coupling beam was approximately 7%.



a) Specimen 2 **b) Specimen 8**

Figure 5 - Hysteresis behavior of short and slender HPFRC coupling beam specimens

In terms of damage and failure mode, all HPFRC coupling beams exhibited negligible shear related damage with several hairline diagonal cracks. Flexural cracks developed early in the test and remained less than 1 mm (0.04 in.) wide until 2% drift. For larger drifts, flexural cracks near the termination section for the dowel reinforcement extended and widened, merging to form a continuous crack over the beam depth. Ultimately, substantial sliding occurred along this wide flexural crack. Some concrete crushing was also observed during the latter part of the tests. Fig. 6 shows photos of the two coupling beams described above at 3% and 5% drift.

From the results of the testing program, the following design recommendations are made. In short coupling beams, diagonal reinforcement should be provided to resist approximately 30% of the expected shear demand (based on the probable beam moment capacity). The HPFRC material, on the other hand, can be assumed to carry a shear stress of $0.42\sqrt{f_c'}$ [MPa] $\left(5\sqrt{f_c'}\text{ [psi]}\right)$, the remainder of the shear being resisted through transverse reinforcement. In slender coupling beams, because of the more dominant role played by flexure, shear capacity is provided by the HPFRC material and stirrups alone (i.e. no diagonal bars). If a precast construction process is used, then intermediate dowel reinforcement must be provided near mid-depth at the connection. Based on experimental results, a dowel bar area equal to 30-40% and 20-30% of the flexural reinforcement area is recommended for coupling beams with aspect ratios less than 2 and between 2 and 4, respectively.

INCORPORATION OF HPFRC COUPLING BEAMS INTO PRACTICE

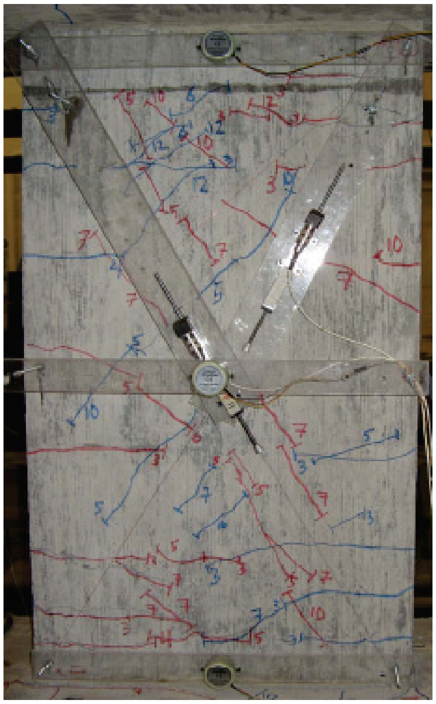
Cary Kopzcynski and Co. (CKC), a design firm in Bellevue, WA, is currently working on the structural design of a 134 m (440 ft) tall reinforced concrete tower (815 Pine) located in downtown Seattle. The 815 Pine tower is primarily a residential building with a central concrete shear wall core. The floor area at each level is approximately 930 m^2 (10,000 ft^2). The building is located on a square site of approximately 36 m by 34 m (118 feet by 113 feet). The tower will include five levels of sub grade parking and four levels of above grade parking. Figure 7a shows a rendering of the tower viewed from street level.

The central concrete shear wall core has dimensions approximately 12.2 m by 11.6 m (40 ft by 38 ft) with walls of varying thickness, from 76 cm (30 in.) at the base to 61 cm (24 in.) at the uppermost levels. There are six 76 cm (30 in.) deep coupling beams at each floor with the widths matching adjacent wall thicknesses. Figure 7b illustrates a plan view of a typical floor with the shear wall and coupling beam configuration. The structure was analyzed and designed using a performance-based design approach, which presented a unique opportunity to investigate innovative designs outside the prescriptive language of the code, provided the performance of the system met all applicable code requirements. The tower design was subject to a structural peer review due to the non-prescriptive structural design approach proposed by CKC. This allowed the CKC design team to investigate the suitability of incorporating HPFRC coupling beams in the core wall of 815 Pine Tower.

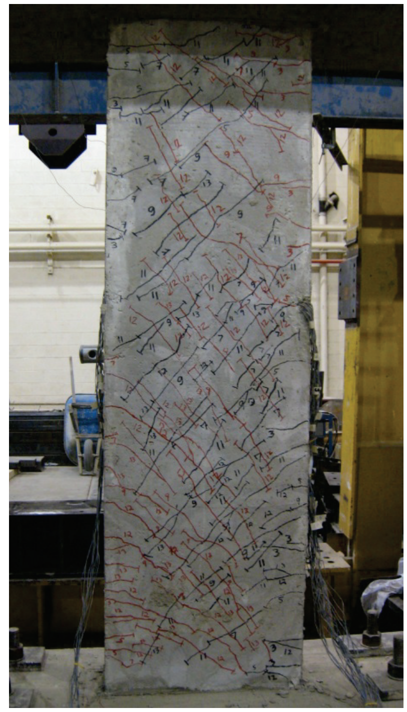
Nonlinear time history analyses of core-wall structure with HPFRC coupling beams

CKC developed two 3-D analytical models of the 134 m (440 ft) tall core-wall structure, one using code-compliant RC coupling beams and the other incorporating the proposed HPFRC coupling beams. For modeling purposes, the coupling beam non-linear parameters were selected based on experimental hysteresis loops in addition to the properties of the concrete and reinforcing steel to be used in the core-wall structure. The two analytical models were subjected to the same 7 pairs of earthquake ground motions. The three dimensional non-linear response history analysis results were used to compare the building response between RC and HPFRC coupling beam systems.

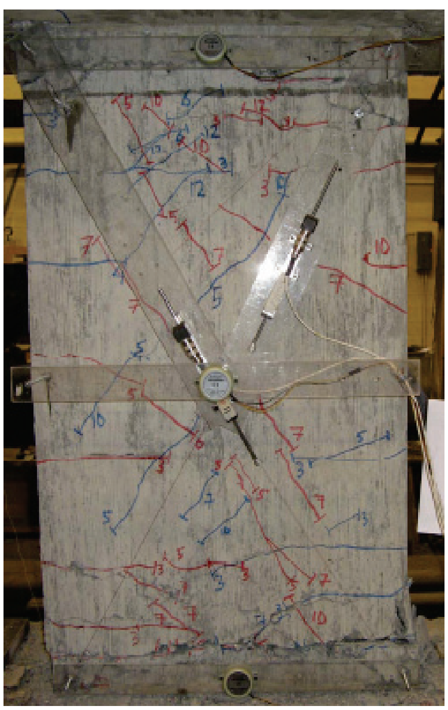
Evaluation of analytical results focused on tower story drift, coupling beam end rotations, and strains in the core wall boundary elements. The mean of the maximum values of the time history analyses were used in this comparative study. Both the RC and HPFRC coupled wall structures exhibited a maximum story drift well below the 2% limit currently allowed by the building code. The story drift results for a controlling ground motion record, over the height of the 134 m (440 ft) tower for both coupled wall systems, are shown in Figure 8. As can be seen, similar story drift responses were obtained for the structure with RC coupling beams (labeled as CVRC) and HPFRC coupling beams (labeled as SFRC). In terms of coupling beam deformation demands, chord rotations for the HPFRC model were in all cases below the 0.05 rad limit criterion set based on results from the tests of HPFRC coupling beams with a 1.75 aspect ratio. Flexural yielding in the wall of the two systems concentrated between the street level and the second floor. A slight decrease in the overall shear and moment at the core walls, however, was observed in the model with HPFRC coupling beams.



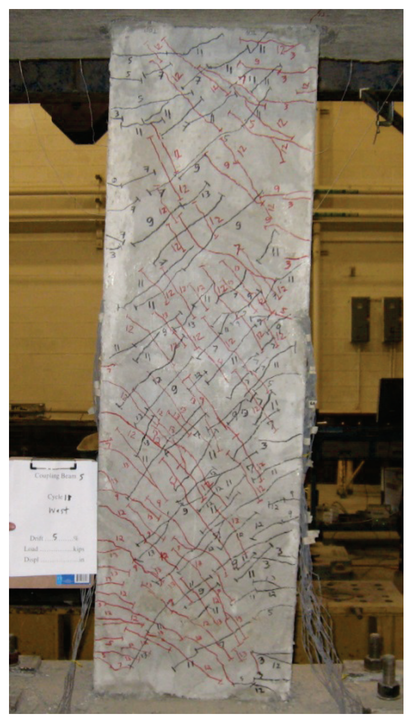
a) Specimen 2 (3% drift)



b) Specimen 8 (3% drift)



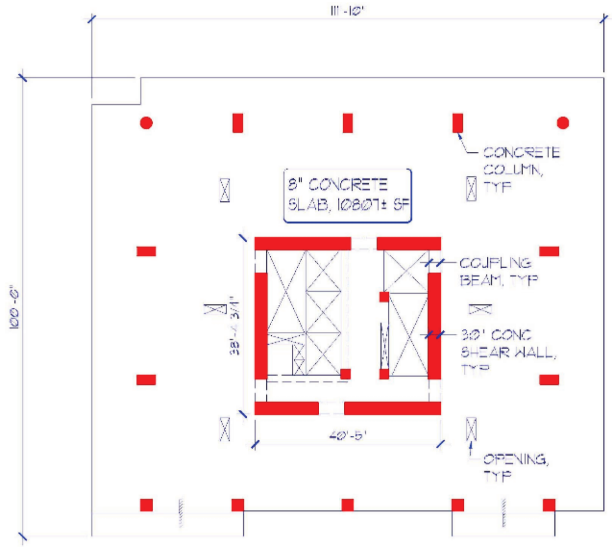
c) Specimen 2 (5% drift)



d) Specimen 8 (5% drift)

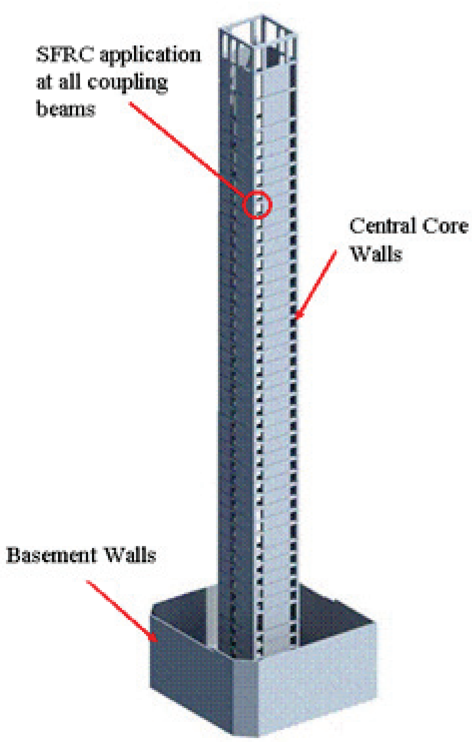
Figure 6 - Damage progression in short and slender HPFRC coupling beam specimens

a) 815 Pine rendering

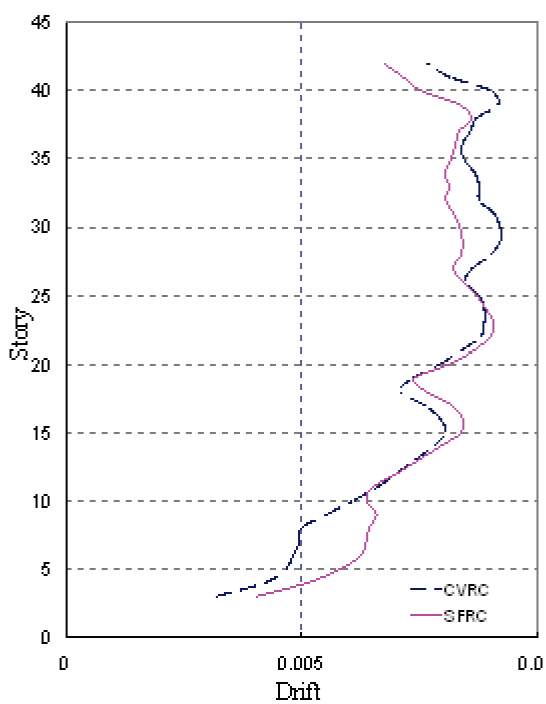


b) 815 Pine floor plan

Figure 7 - Rendering and floor plan of 815 Pine Tower



a) 815 Pine with HPFRC coupling beams

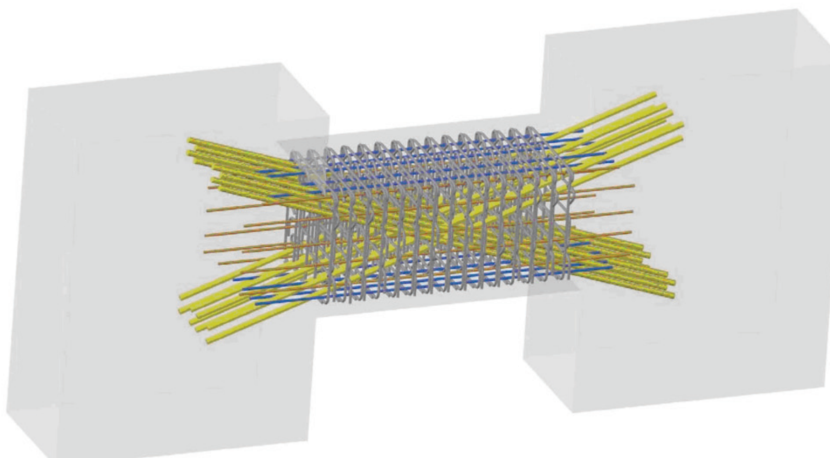


b) 815 Pine story drift distribution

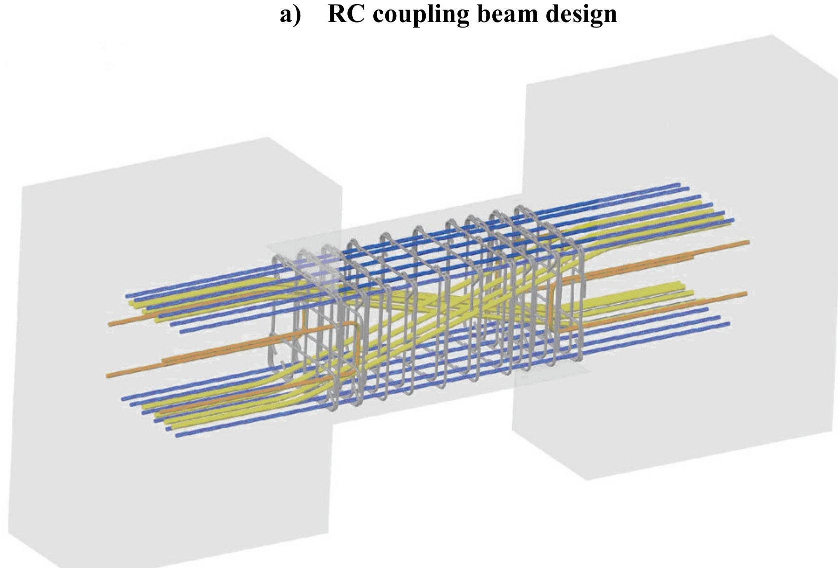
Figure 8 – HPFRC coupling beam application and story drift distribution for 815 Pine Tower

Cost analysis

Figure 9 presents a graphical comparison of coupling beams of equal capacity that were designed for 815 Pine Tower. The graphical comparison clearly illustrates the reduction of reinforcement between HPFRC and RC coupling beams. To better quantify the material and cost differences between RC and HPFRC coupling beams, Table 3 illustrates the reinforcing quantities for two highly stressed (i.e. shear stress nearly ACI Code upper limit) HPFRC and RC coupling beams with a 2.1 and 3.3 aspect ratio. For example, in the first scenario (3a), the diagonal and transverse reinforcement weight of the HPFRC coupling beams are reduced by approximately 65% and 45% respectively, but with an added 110 kg (240 lb) of steel fibers. Assuming an installed cost of \$2.20/kg (\$1.00/lb) of rebar and \$4.10/kg (\$1.85/lb) of high-strength hooked steel fibers, an approximate 30% cost savings can be achieved. A similar comparison for a slender coupling beam illustrates an approximate cost savings of approximately 20%. Although Table 3b shows only the material weights and costs corresponding to a slender beam with diagonal reinforcement, the total rebar weight is similar to that required for the same coupling beam if diagonal bars is eliminated. This is because additional flexural reinforcement needs to be provided to compensate for the reduction of flexural strength caused by the elimination of diagonal reinforcement. It should be kept in mind, however, that although both designs would require a similar total rebar weight, the construction of the coupling beam without diagonal reinforcement would be substantially simpler.



a) RC coupling beam design



b) HPFRC coupling beam design

Figure 9 - RC versus HPFRC coupling beam design with equal design strength

Constructability

In typical concrete shear wall and coupling beam towers, diagonally reinforced coupling beams and shear wall boundary element confining steel represent major constructability challenges and are often the cause of construction delays. As described above, the use of HPFRC leads to substantial reductions in quantity and size of conventional reinforcement in coupling beams. In addition, the use of smaller diagonal bars allows them to be internally bent so that they can exit the beam horizontally (Figures 4a and 9b). In a typical reinforced concrete coupling beam, the diagonal reinforcement extends at an angle into the boundary element of the shear wall (Figure 1 and 9a). These angled bars have proven challenging for contractors, as they cannot cast the wall below until the reinforcement for the coupling beam is installed. Thus, the ability to bend the diagonals and exit the beam horizontally into the shear wall represents a major improvement in coupling beam constructability. Reducing confinement steel will significantly ease reinforcing steel placement in heavily congested areas. In short, the use of HPFRC will generate the following improvements in the construction of coupling beams: 1) reduced total rebar tonnage; 2) reduced labor by simplifying the placement of reinforcement; and 3) accelerated construction schedule and reduced potential for delays.

Table 3: Quantity comparisons between conventional RC and SFRC coupling beam

(a) 2.1 Aspect ratio 750 x 750 x 1575 mm coupling beam

Materials	Conventional RC		HPFRC coupling beam	
	Rebar	Cost	Rebar	Cost
	kg (lb)	U.S. dollar	kg (lb)	U.S. dollar
Diagonal reinforcement	630 (1390)	1390	233 (513)	513
Straight reinforcement	93 (206)	206	93 (206)	206
U-Shape reinforcement			20 (45)	45
Shear reinforcement	288 (635)	635	159 (351)	351
Steel fibers			110 (243)	449
Sum	1012 (2230)	2231	616 (1358)	1564

(b) 3.3 Aspect ratio 600 x 600 x 2000 mm slender coupling beam

Materials	Conventional RC		HPFRC coupling beam	
	Rebar	Cost	Rebar	Cost
	kg (lb)	U.S. dollar	kg (lb)	U.S. dollar
Diagonal reinforcement	280 (618)	618	127 (279)	279
Straight reinforcement	64 (141)	141	64 (141)	141
U-Shape reinforcement			7 (16)	16
Shear reinforcement	266 (586)	586	131 (289)	290
Steel fibers			89 (196)	362
Sum	610 (1345)	1345	419 (923)	1088

CONCLUSIONS

Results from the experimental investigation described herein indicate that the use of strain-hardening, high-performance fiber reinforced concrete (HPFRC) in relatively short (beam span-to depth ratio of 1.75) and slender (beam span-to-depth ratios of 2.75 and 3.3) coupling beams allows an approximately 70% reduction and total elimination of diagonal reinforcement, respectively. In addition to contributing to shear strength, fiber reinforcement provides confinement to the concrete, eliminating the need for special column-type confinement, except at the beam ends. Coupling beam drift capacities when subjected to shear stresses close to the upper limit in the 2008 ACI Building Code ($0.83\sqrt{f_c'}$ [MPa] $\left(10\sqrt{f_c'}\text{ [psi]}\right)$) ranged between approximately 5% for coupling beams with aspect ratio of 1.75 and 7% for coupling beams with a 3.3 aspect ratio.

From a cost viewpoint, savings between 20% and 30% in material costs can be obtained with the use of the proposed HPFRC coupling beams compared to traditional reinforced concrete coupling beams. Further, the use of smaller diagonal bars in HPFRC coupling beams allows the bending of these bars at the beam ends such that they exit the beam horizontally. This greatly facilitates the wall construction process because concrete casting in the wall up to the bottom of each coupling beam can be performed prior to placing of the coupling beam reinforcement. For added flexibility, the proposed HPFRC coupling beams can be constructed either cast-in-place or precast.

The large drift capacity observed in the HPFRC coupling beam tests combined with the substantial savings in material costs and construction time make the proposed coupling beams ideal for use in high-rise core-wall construction. Nonlinear analyses conducted on a 134 m (440 ft) tall reinforced concrete tower to be constructed in the city of Seattle, WA indicated adequate structural response with coupling beam drift demands below the observed drift capacities.

REFERENCES

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

Lequesne, R. D., Setkit, M., Parra-Montesinos, G. J. and Wight, J. K. (2010). “Seismic Detailing and Behavior of Coupling Beams With High-Performance Fiber Reinforced Concrete,” *Antoine E. Naaman Symposium – Four decades of progress in prestressed concrete, fiber reinforced concrete, and thin laminate composites*, SP-272, American Concrete Institute, Farmington Hills, MI, pp. 205-222.

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.

Naish, D., Wallace, J.W., Fry, J.A., and Klemencic, R. (2009). “Reinforced concrete link beams: alternative details for improved construction,” *UCLA-SGEL Report 2009-06*, Structural & Geotechnical Engineering Laboratory, University of California at Los Angeles, 103 pp.

Paulay, T., and Binney, J.R. (1974). “Diagonally Reinforced Coupling Beams,” *Special Publication SP-42*, American Concrete Institute, Detroit, Michigan, pp. 579-598.

ACKNOWLEDGEMENTS

This research project was funded by NSF grant #CMS 0530383 and Bekaert Corporation. It is a part of the NEES research program. The ideas and conclusions expressed are those of the writers and do not necessarily represent the views of the sponsors.