16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 0555 (Abstract ID)

Registration Code: S-Q1464805856

USE OF HIGH PERFORMANCE REINFORCED CONCRETE SYSTEMS LOCATED IN SEISMIC REGIONS

M. Melek(1), T. Kang(2)

(1) *Associate, Arup, murat.melek@arup.com*
(2) *Professor, Seoul National University, tkang@snu.ac.kr*

Abstract

In the past decade, reinforced concrete became the most popular building material for tall buildings [1]. According to the Council of Tall Buildings and Urban Habitat, in 2013, 63% of the buildings 200 m or taller were built out of reinforced concrete. Shear demand on the core wall is one of the key parameters that govern the seismic design of reinforced concrete high-rise building systems. Higher mode affects that are not prominent in the design of regular buildings further amplify these demands controlling the core wall dimensions and wall thicknesses over the building height. For core wall systems with or without backing moment frame, the wall thicknesses required to resist these shear demands could be substantial. Increased wall thicknesses not only increase the material costs and reduce the efficiency of the floor plate by increasing the ratio of the core wall area to the floor area but also result in increased foundation forces and seismic forces due to increased weight.

Primary inelastic energy dissipation mechanism of core wall systems, is the flexural or shear yielding of coupling beams connecting the core wall pier together. Rotation and shear demands on these elements are generally substantial. These demands could lead design that require too much reinforcing steel creating constructability problems as well as substantial damage and repair time for the building during and after an earthquake.

High force and deformation demands controlling the core wall thicknesses and coupling beam design led us to investigate the feasibility of using High-Performance Fiber-Reinforced Cementitious Composites (HPFRCCs) as an alternative to the normal to high strength concrete that is currently the common construction practice for high rise buildings in seismic zones. As part of the study, four case study buildings were analyzed. Case study lateral load resisting systems consisted of a) core only, (b) dual system (core + backing moment frame), (c) core with outriggers at mid-height, and (d) core with outriggers at mid-height and roof. Construction cost and seismic performance of each structural system was compared.

The use of HPFRCC's enabled the relaxation of confinement reinforcement in columns and boundary elements of shear walls. In addition, reinforcement quantities in coupling beams were reduced using HPFRCC's. Savings in reinforcement quantities by using fiber reinforced concrete is calculated to be considerably less than the additional cost of using HPFRCC's.

Seismic performance was measured in terms of expected loss. Incremental dynamic analysis (IDA) results were used with ATC 58 methodology and PACT program to assess the expected loss due to seismic activity.

Considering the improved performance of the structural members and reduced cost, results of the study suggested that use of HPFRCC elements should be seriously considered for the design of tall buildings in seismic regions.

Keywords: Tall Buildings, Fiber Reinforced, High Performance

1. Introduction

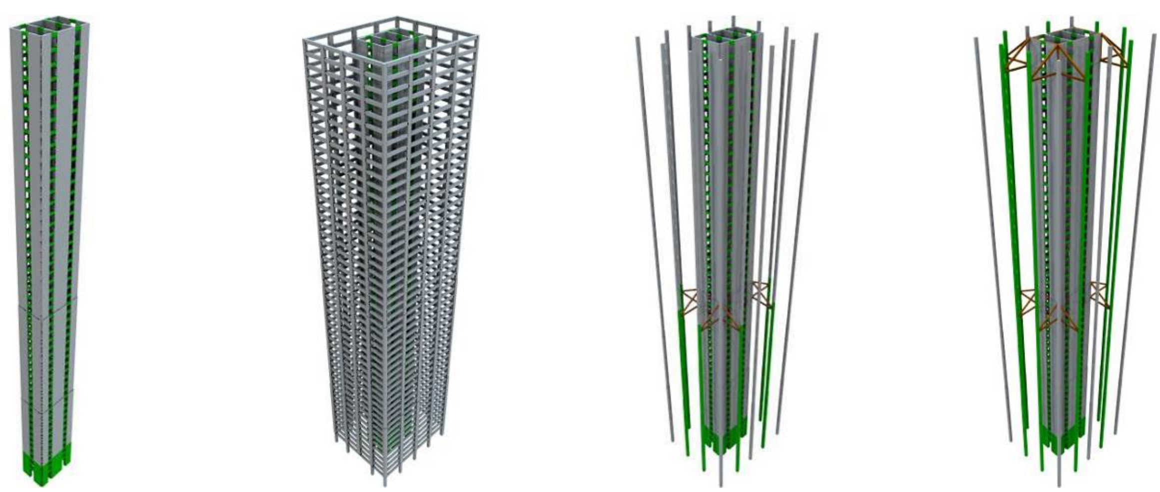
In the past decade, reinforced concrete became the most popular building material for tall buildings [1]. According to the Council of Tall Buildings and Urban Habitat, in 2013, 63% of the buildings 200 m or taller were built out of reinforced concrete. Shear demand on the core wall is one of the key parameters that govern the seismic design of reinforced concrete high-rise building systems. Higher mode affects that are not prominent in the design of regular buildings further amplify these demands controlling the core wall dimensions and wall thicknesses over the building height. For core wall systems with or without backing moment frame, the wall thicknesses required to resist these shear demands could be substantial. Increased wall thicknesses not only increase the material costs and reduce the efficiency of the floor plate by increasing the ratio of the core wall area to the floor area but also result in increased foundation forces and seismic forces due to increased weight.

Primary inelastic energy dissipation mechanism of core wall systems, is the flexural or shear yielding of coupling beams connecting the core wall pier together. Rotation and shear demands on these elements are generally substantial. These demands could lead design that require too much reinforcing steel creating constructability problems as well as substantial damage and repair time for the building during and after an earthquake.

High force and deformation demands controlling the core wall thicknesses and coupling beam design led us to investigate the feasibility of using High-Performance Fiber-Reinforced Cementitious Composites (HPFRCCs) as an alternative to the normal to high strength concrete that is currently the common construction practice for high rise buildings in seismic zones. As part of the study, four case study buildings were analyzed. Case study lateral load resisting systems consisted of a) core only, (b) dual system (core + backing moment frame), (c) core with outriggers at mid-height, and (d) core with outriggers at mid-height and roof. Construction cost and seismic performance of each structural system was compared.

The use of HPFRCC's enabled the relaxation of confinement reinforcement in columns and boundary elements of shear walls. In addition, reinforcement quantities in coupling beams were reduced using HPFRCC's. Savings in reinforcement quantities by using fiber reinforced concrete is calculated to be considerably less than the additional cost of using HPFRCC's.

Seismic performance was measured in terms of expected loss. Incremental dynamic analysis (IDA) [2] results were used with ATC 58 methodology [3] and PACT [4] program to assess the expected loss due to seismic activity.



Core Only Core + Moment Frames Core + One Outrigger Core + Two Outriggers

Fig. 1 – Structural Systems

2. Seismic Hazard

The case study buildings are located in Los Angeles. Per ASCE 7-10 [5] if either the short period or the 1-second spectral accelerations at the building site are greater than 0.75 and 0.3 respectively, assigned seismic design categoty of the building is D or greater. Based on this definition, the case study building falls into the seismic design category D.

Seven suits of time histories scaled to match the target spectrum. For the case study, Incremental Dynamic Analysis (IDA) was undertaken at 20%, 40%, 67% (Code Level), 80% and 100% MCE levels.

3. Loss Estimation Study

Incremental dynamic analysis (IDA) results were used with ATC 58 methodology and PACT program was to assess the expected loss due to seismic activity. In its current form PACT program makes use of engineering demand parameters: drift and acceleration as the input data for the loss estimation analysis. The expected loss calculated by the program is therefore linked to the calculated drift for a given seismic hazard level. For this reason, the tower with the dual system performed substantially better than the core only systems with normal and ultra-high strength concrete (Figure 2). Between the two core only systems, estimated repair costs were very similar.

It should be noted that PACT's current capabilities are very limited in terms of assessing the damage and possible loss of concrete core wall elements and coupling beams. The damage sustained by coupling beams is not considered by PACT and the fragility functions specified for the slender wall elements may not be representative for thick concrete walls with complex floor plan layouts. Regardless of the selected structural system, coupling beams act as the primary mechanism for energy dissipation (Figure 3). The coupling beams are expected to get damaged as they dissipate energy during the seismic event. Therefore the lack of fragility functions for coupling beams in PACT software, is expected to lead to an unrealistic estimation of the expected loss. On the other hand inelastic energy dissipation due to the axial/flexural yielding of core wall segments were observed to be limited. Based on the PACT analysis results, expected loss in shear wall elements were also limited.

At the time of this study, available literature was inadequate to improve the fragility functions use for the loss estimation calculations.

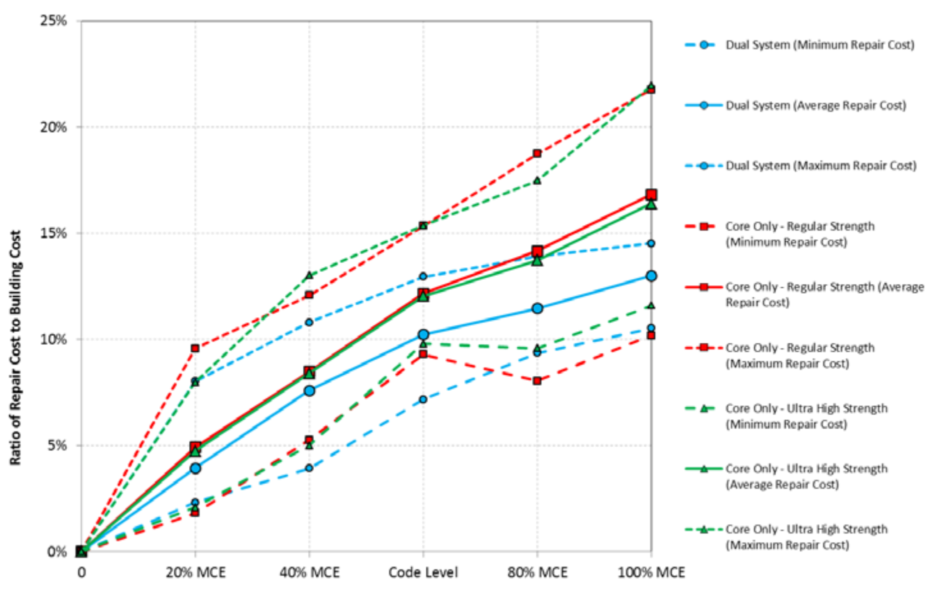


Fig. 2 –Ratio of Repair Cost to Building Cost

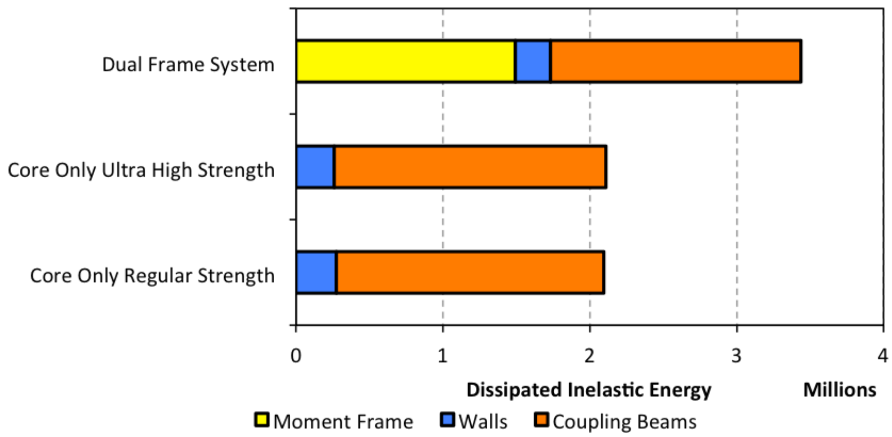


Fig. 3 –Inelastic Energy Dissipated by Structural Elements

4. Performance Improvement

IDA results revealed that use of ultra-high strength concrete did not have the positive effect on the seismic performance of the core only system that would justify the cost difference. For this reason, ultra-high strength concrete was only used at the columns of the system with outriggers. Figure 4 shows potential applications of HPFRCCs and Ultra-high strength concrete.

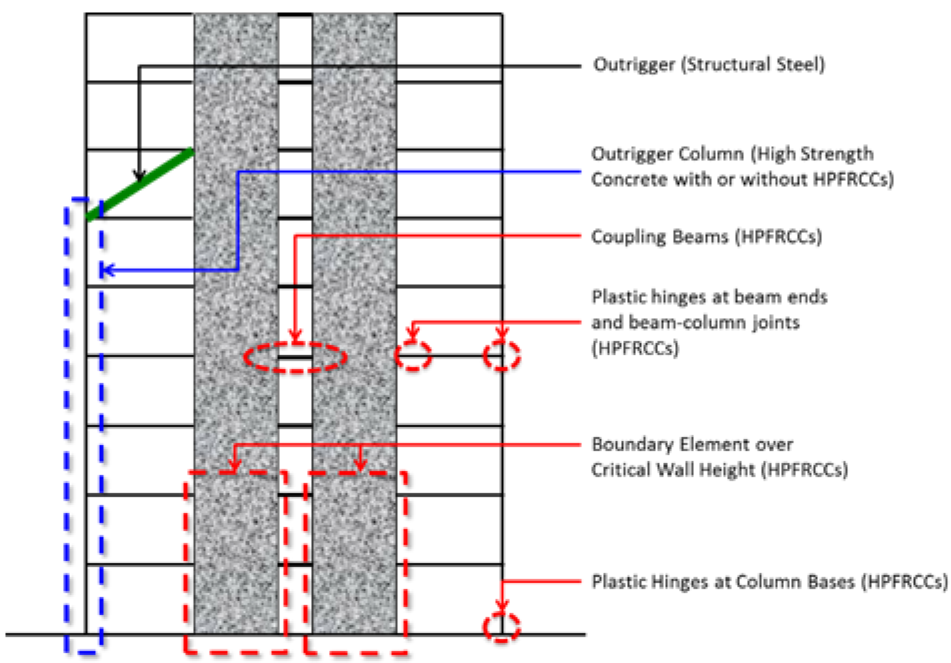


Fig. 4 – Potential Applications of High-performance fiber-reinforced cementitious composites (HPFRCCs) and high-strength concrete

Table 1 – Potential Benefits of Using HPFRCC

Structural Element Type	**Potential Benefits of Using HPFRCC**
Coupling Beams	Improved constructability Reduced crack widths
Boundary Element over Critical Wall Height*	Relaxation of confinement reinforcement
Outrigger Column	Relaxation of confinement reinforcement

In the following sections, potential means to improve the structural and nonstructural seismic performance is discussed.

4.1 Shear Walls

IDA study considered using ultra high strength concrete up to 19 ksi over the critical wall height. Based on the results and as shown on Figure 3, it could be argued that there is no tangible performance improvement in terms of limiting structural and nonstructural expected loss.

ACI 318-11 [6] Section 11.1.2 limits the maximum value of $10\sqrt{f_c'}$ to 100 psi (8.3 MPa). Due to this limit, increasing the specified compressive strength of concrete beyond 10 ksi does not provide additional shear capacity to the wall.

PACT software utilizes the fragility functions given on Figure 5 for slender reinforced concrete wall elements. Drift is used as the engineering demand parameter for the fragility functions for the slender reinforced concrete walls.

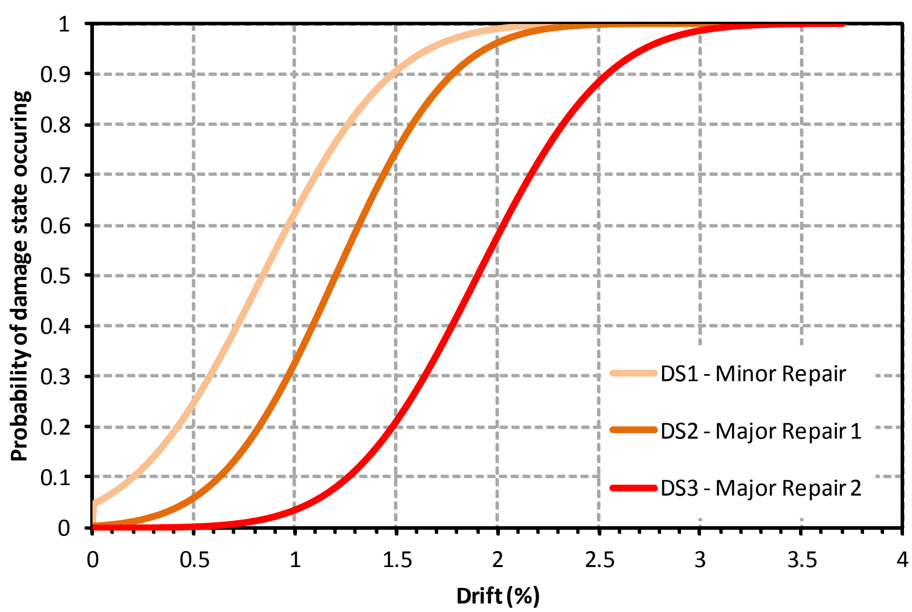


Fig. 5 –Fragility Curves for Shear Wall Elements

It should be noted that, although the interstory drift results shown on Figure 6 point to higher interstory drift results on the upper levels of the case study buildings, these drifts are due to the curvature of the reinforced concrete core at the base of the tower. Therefore, it is worthwhile to indicate that high interstory drift results obtained from nonlinear analysis do not necessarily mean damage to the core walls over the building height. According to the IDA results, the damage to shear wall elements was not a significant contributor to the expected loss.

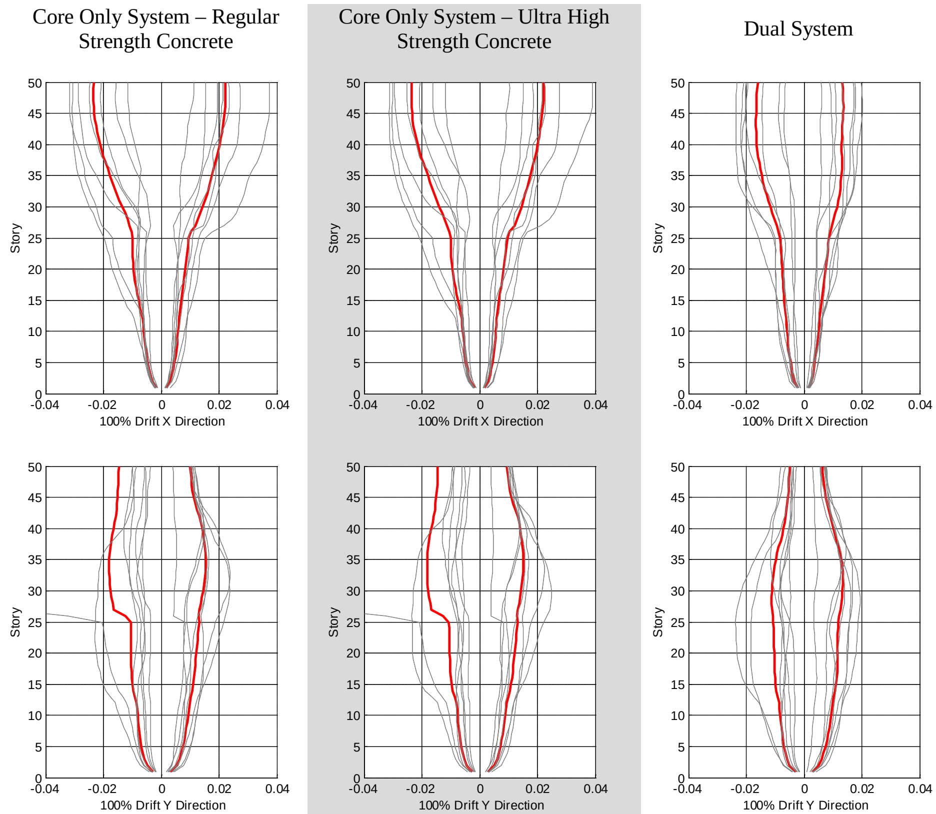


Fig. 6 –Interstory Drift Results of Case Study Buildings

One area where High-Performance Fiber-Reinforced Cementitious Composites (HPFRCCs) could be used to improve the performance and constructability of the shear walls is the boundary elements along the critical tower wall height. Figure 7 shows the boundary confinement reinforcement required by ACI 318 for different concrete and reinforcement grades. As shown on the figure as the concrete strength increases so does the required boundary confinement ratio. A study by Montesinos et al. [6] investigated the use of fiber-reinforced cement composites (FRCCs) in lieu of the boundary reinforcement over the critical wall height. In their experimental study, FRCC wall specimen with 2.0% volume fraction of steel hooked fibers and no boundary confinement was able to achieve a drift capacity of 3.0%. Another specimen with regular ready-mix concrete reinforced with steel hooked fibers in a 1.5% volume fraction had a drift capacity of 2.5%. For all three specimens with steel hooked fibers and reduced or no boundary reinforcement exhibited denser arrays of smaller cracks compared to the regular wall specimen. Following this study, Lequesne et al. [7] suggested that using a maximum spacing of wall boundary element confinement reinforcement of half the wall web thickness is adequate in HPFRC walls. The conclusions of these studies could be used to reduce the amount of boundary element confinement reinforcement improving not only the constructability and cost of the shear wall elements but also their cost and time to repair by reducing the crack widths.

Lequesne et al. [8] also suggested that steel hooked fibers could contribute to the shear stress capacity of the wall up to $4\sqrt{f_c'}$ (psi) ($0.33\sqrt{f_c'}$ (MPa)). This could be used to further reduce the amount of horizontal shear reinforcement over the critical wall height.

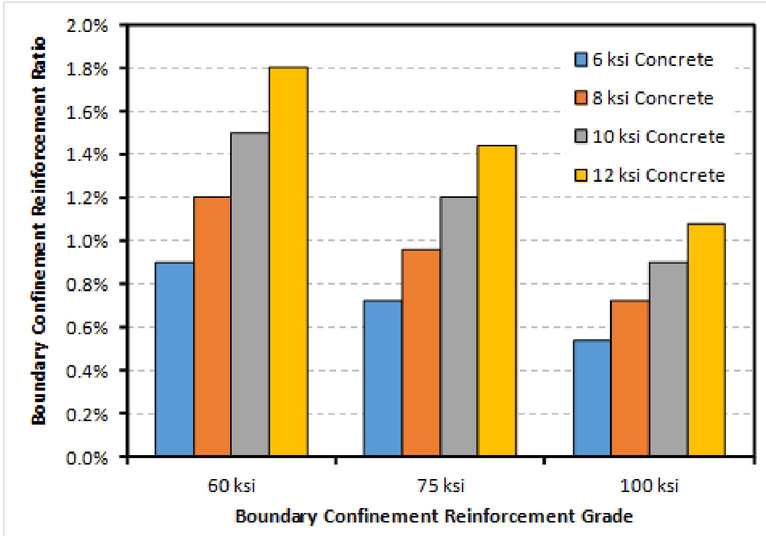


Fig. 7 –Boundary confinement reinforcement for given concrete strength and reinforcement grade

4.2 Coupling Beams

Coupling beams connecting shear wall segments to each other have a very distinct role within the lateral load resisting system of a tall building. Unlike the shear wall elements that are expected to undergo inelastic response over the critical wall height, the coupling beams undergo substantial plastic rotations over the entire building height. For this reason, coupling beams act as the main energy dissipating mechanism of the core only systems. For other systems including the dual system with backing moment frame and systems with outriggers, energy dissipated by the coupling beams constitute a major portion of the total dissipated energy.

It is common to have coupling beam dimensions and length governed by architectural requirements. For most commonly used coupling beam span to depth ratios, diagonally placed steel reinforcement bars provide shear (and flexural) strength. Test results by Naish et al. [9] indicate that diagonally reinforced coupling beams can undergo as much as 6.0% rotation without having significant strength reduction. Figure 8 and Table 1 present the fragility functions developed by Naish et al. [9].

It is common to design high-rise buildings in seismic regions to satisfy collapse prevention target performance when subjected to maximum considered eqarthquake (with a return period of 2,475 years) hazard level. ASCE 41-13 (2014) sets the plastic chord rotation limit for reinforced concrete coupling beams with diagonal reinforcement at 5.0%. As shown on Figure 8, this target performance aims for a damage state between DS2 – Major Repair 1 and DS3 – Major Repair 2. Noting that the coupling beams over the building height would be in similar damage states, repair procedures for the coupling beams could cost substantial time and money.

Table 2 –Damage States for HPFRCC Coupling Beams (Lequesne et al. [8])

Damage State	**Definition of Damage**	**Repair Procedures**
Yield	Substantial change in stiffness of load-deformation plot	None
DS1 – Minor Repair	Residual cracks greater than 1/16 in.	Epoxy injection of cracks
DS2 – Major Repair 1	Residual cracks greater than 1/8 in.; minor spalling of concrete	Epoxy injection of cracks in beam and slab; replacement of spalled concrete
DS3 – Major Repair 2	Significant strength degradation (<0.8 V_n); buckling/fracture of	Chip away damaged concrete; attach mechanical couplers to remaining bars; replace damaged/fractured reinforcement; replace

	reinforcement; crushing of concrete	damaged concrete.

ACI 318-11 equation 21-9 gives nominal shear strength of a diagonally reinforced coupling beam as:

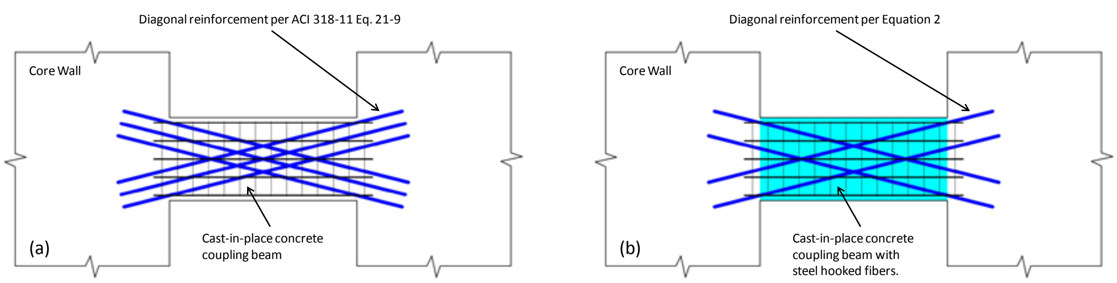
$$V_n = 2 \cdot A_{vd} \cdot f_y \cdot \sin\alpha \leq 10 \cdot \sqrt{f'_c} \cdot A_{cw} \text{ (psi) (ACI 318-11 Eq. 21-9)} \quad (1)$$

$$V_n = 2 \cdot A_{vd} \cdot f_y \cdot \sin\alpha \leq 0.83 \cdot \sqrt{f'_c} \cdot A_{cw} \text{ (MPa) (ACI 318-11 Eq. 21-9)} \quad (2)$$

where A_{vd} is the area of diagonal reinforcement, f_y is the yield strength of diagonal reinforcement, α is the reinforcement angle and A_{cw} is the cross sectional area of the coupling beam. Based on Equation 1, the diagonal reinforcement area to resist shear forces is dependent on the reinforcement angle. Shallow reinforcement angles could result in unconstructable designs. Harries and Shahrooz [9] argue that steel placement is often impractical for coupling beams having a span-to-depth ratio greater than 1.5. According to the same study, it would be difficult to construct a diagonally reinforced coupling beam that has a design shear strength of $10\sqrt{f_c}'A_{cw}$ psi ($0.83\sqrt{f_c}'A_{cw}$ (MPa)). For this reasons Harries and Shahrooz recommended $6\sqrt{f_c}'A_{cw}$ psi ($0.5\sqrt{f_c}'A_{cw}$ (MPa)) as the practical upper limit for the design shear capacity of diagonally reinforced coupling beams. Given the architectural constraints and deformation and shear demands, it would not be possible to avoid using diagonal reinforcement for span-to-depth ratios greater than 1.5 and/or limit the design shear strength to a lower value than the ACI 318 limit of$10\sqrt{f_c}'A_{cw}$ psi ($0.83\sqrt{f_c}'A_{cw}$ (MPa)) all the time. HPFRCCs could provide a solution to this problem.

Studies by Lequesne et al. [8] and Canbolat et al. [11] focused on ways to reduce the amount of diagonal reinforcement by utilizing High-Performance Fiber-Reinforced Cementitious Composites (HPFRCCs). Figure 8 presents the concepts where Figure 8(a) is the ordinary diagonally reinforced cast-in-place concrete coupling beam. Figures 8(b) and 8(c) shows coupling beams with steel hooked fibers with cast-in-place and precast concrete respectively.

Although the use of diagonal reinforcement improves the seismic performance of coupling beams, it brings constructability problems. The diagonal reinforcement bars need to be embedded into wall boundary zones very commonly very densely and heavily reinforced. Installation of the diagonal reinforcement could lead to construction delays. The precast or cast-in-place layout shown on Figure 8(c), with the help of the reduction in reinforcement by using HPFRCCs has the potential to alleviate this kind of constructability problems.



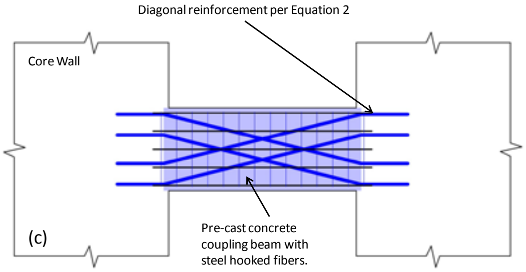


Fig. 8 –Coupling Beams (a) cast-in-place concrete (b) cast-in-place concrete with steel hooked fibers; and (c) pre-cast concrete with steel hooked fibers

According to the tests results by Canbolat et al. [11], HPFRCC coupling beams with a span-to-depth ratio of 1 were able to reach 4% coupling beam drift deformation while maintaining 80% of their shear capacity. Lequesne et al. [8] performed coupled wall system tests using HPFRCC coupling beams with a span-to-depth ratio of 1.75. The coupled wall system was able to retain 80% of the peak capacity of the system up to 2.5% drift. HPFRCC coupling beams were damage tolerant up to 2.5% system drift. The hysteretic response of the HPFRCC coupling beams are comparable to the cast-in-place reinforced concrete coupling beams with diagonal reinforcement. In addition, despite reduced reinforcement, steel hooked fibers prevent wide cracks. Damage states for HPFRCC coupling beams are presented on Table 3.

Table 3 – Damage States for HPFRCC Coupling Beams (Lequesne et al. (2013))

Damage State	**Definition of Damage**	**Drift**
DS1 – minor damage	Diagonal and vertical cracks less than 1 mm	1%
DS2 – major damage (I)	2.5 mm flexural cracks and <1 mm diagonal cracks	3.5%
DS3 – major damage (II)	Shear sliding observed. Cracks > 3.5 mm.	5.5%

Lequesne et al. (2013) recommended using $5\sqrt{f_c'}$ (psi) ($0.41\sqrt{f_c'}$ (MPa)) contribution of HPFRCC to the shear capacity of coupling beams. ACI 318-11 Eq. 21-9 could be modified to take into account this recommendation:

$$V_n = 2 \cdot A_{vd} \cdot f_y \cdot \sin\alpha + 5 \cdot \sqrt{f_c'} \cdot A_{cw} \leq 10 \cdot \sqrt{f_c'} \cdot A_{cw} \text{ (psi)} \tag{2}$$

$$V_n = 2 \cdot A_{vd} \cdot f_y \cdot \sin\alpha + 0.41 \cdot \sqrt{f_c'} \cdot A_{cw} \leq 0.83 \cdot \sqrt{f_c'} \cdot A_{cw} \text{ (MPa)} \tag{3}$$

Using Equation 3 to calculate the required diagonal reinforcement area would lead to a diagonally reinforced coupling beam with substantially less reinforcement than a regular diagonally reinforced coupling beam.

Fragility functions and nonlinear modeling parameters for diagonally reinforced coupling beams with HPFRCCs have not been developed yet. Considering the comparable hysteretic response between diagonally reinforced coupling beams with and without HPFRCCs, ASCE 41-13 modeling parameters are used to model the HPFRCC coupling beams.

4.3 Columns

Coupling beams and boundary elements of shear walls elements could immensely benefit from the use of HPFRCCs as presented in the previous sections. However, there were no tangible benefits of using high strength concrete for these elements. Shear and deformation capacity controlled the seismic performance of these elements. On the other hand depending on the use of the column within the structural system, utilizing high strength concrete could improve the seismic performance of the system. For dual systems where moment frames work in conjunction with shear walls, the columns are expected to experience high rotation demands. These columns are expected to have a ductile response. Bechtoula et al. [12] tested the seismic performance of reinforced concrete columns with concrete strengths ranging from 80 MPa to 180 MPa under cyclic loading. When moment - drift response were compared it was seen that columns with Grade 180 concrete exhibited strength loss around 2% drift. This would be undesirable considering the drift target set by the tall building design guidelines (3% average). Columns with 80 MPa concrete strength showed the most desirable cyclic response.

Another use of columns within the lateral load resisting system of a tall building would be with outriggers. For the tall buildings where the reinforced concrete core cannot provide adequate flexural stiffness on its own alone, outriggers can be used to utilize the axial stiffness of perimeter columns. The outrigger system does not lessen the shear demands on the core walls, but it can alleviate the coupling beam rotations and drifts substantially without using closely spaced columns and deep beams along the tower perimeter. Since the outrigger column response is governed by axial (not moment/shear) response, increasing the concrete strength (and consequently the modulus of elastisticity) would have a substantial impact on the efficiency of the system.

5. Conclusion

High force and deformation demands controlling the core wall thicknesses and coupling beam design led us to investigate the feasibility of using High-Performance Fiber-Reinforced Cementitious Composites (HPFRCCs) as an alternative to the normal to high strength concrete that is currently the common construction practice for high rise buildings in seismic zones. As part of the study, four case study buildings were analyzed. Case study lateral load resisting systems consisted of a) core only, (b) dual system (core + backing moment frame), (c) core with outriggers at mid-height, and (d) core with outriggers at mid-height and roof. Construction cost and seismic performance of each structural system was compared.

The use of HPFRCC's enabled the relaxation of confinement reinforcement in columns and boundary elements of shear walls. In addition, reinforcement quantities in coupling beams were reduced using HPFRCC's. Savings in reinforcement quantities by using fiber reinforced concrete is calculated to be considerably less than the additional cost of using HPFRCC's.

ATC 58 methodology and PACT program was used to assess the possible loss due to the seismic hazard levels. It should be noted that PACT's current capabilities are very limited in terms of assessing the damage and possible loss of thick concrete core wall elements and coupling beams. The damage sustained by coupling beams is not considered by PACT and the fragility functions used for the wall elements are for rectangular wall elements with behaviors expected to be substantially different than the behavior of core wall elements up to 1000 mm thick. For this purpose as the next phase of the study more realistic repair and loss data is planned to be used for the loss estimation calculations.

Reinforcement weights for systems with normal weight concrete and HPFRCC systems were compared. Substantial saving in steel quantities, especially in core walls and coupling beams were observed. Savings in reinforcement quantities by using fiber reinforced concrete is calculated to be close to $6,000,000.00. The cost of using fiber reinforced concrete is expected to be considerably less than this value. Considering the improved performance of the structural members, results of this study suggest that use of HPFRCC elements should be seriously considered for the design of tall buildings in seismic regions.

6. Copyrights

7. References

[1] Council on Tall Buildings and Urban Habitat (CTBUH) (2014): 2013: A Tall Building Review, *CTBUH Journal*, 2014 Issue I.

[2] Melek M. (2014): High Performance Wall System Study for Tall Buildings Located in Seismic Regions, Report.

[3] Applied Technology Council (2012): Seismic Performance Assessment of Buildings Volume 1, FEMA P-58.

[4] PACT Software Version 2.0, Applied Technology Council.

[5] ASCE/SEI 7-10 (2010): Minimum Design Loads for Buildings and Other Structures.

[6] ACI 318-11 (2011): Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI 48331.

[7] Parra-Montesinos G J, Canbolat B A, and Jeyaraman G (2006): Relaxation of Confinement Reinforcement Requirements in Structural Walls Through the Use of Fiber Reinforced Cement Composites, 8th National Conference on Earthquake Engineering, San Francisco, CA, April 2006.

[8] Lequesne R D, Parra-Montesinos G J and Wight J K (2013): Seismic Behavior and Detailing of High-Performance Fiber-Reinforced Concrete Coupling Beams and Coupled Wall Systems, *Journal of Structural Engineering* 139, SPECIAL ISSUE: NEES 2: Advances in Earthquake Engineering, 1362–1370.

[9] Naish D, Fry J A, Klemencic, R, and Wallace, J W (2009): Reinforced Concrete Link Beams: Alternative Details for Improved Constructability, Report to Charles Pankow Foundation, School of Engineering and Applied Science University of California, Los Angeles.

[10] Harries K A and Shahrooz B M (2005): Hybrid Coupled Wall Systems State-of-the-Art, *Concrete International*.

[11] 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.

[12] Bechtoula H, Mehani Y, Kibboua A, Naili, Kono S, and Watanabe F: Seismic Behavior of High Strength Reinforced Concrete Columns Under Reversed Cyclic Loading" 15th World Conference on Earthquake Engineering Proceedings, Lisbon.