Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No

PERFORMANCE OF AN 18-STORY COUPLED WALL SYSTEM WITH HIGH PERFORMANCE FIBER REINFORCED CEMENTITIOUS COMPOSITE (HPFRCC) COUPLING BEAMS

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ABSTRACT

Coupled reinforced concrete (RC) shear walls are often used in zones of high seismic risk. The RC coupling beams that connect individual shear walls must transfer adequate force between adjacent walls and, at the same time, are expected to contribute significantly to energy dissipation through ductile deformation during strong seismic shaking. These stringent requirements usually result in a dense configuration of reinforcement, which complicates erection of RC coupled wall systems. High performance fiber reinforced cementitious composite (HPFRCC) materials have a unique strain hardening behavior in tension that translates into enhanced shear and bending response at the structural level. Recent research has shown that the use of HPFRCC in coupling beams can result in a relaxation in reinforcing details while maintaining good strength characteristics and deformation capacity. This paper discusses the seismic performance of an 18-story coupled wall system in which HPFRCC coupling beams are used instead of traditional RC beams. System performance is evaluated through various parameters including inter-story drift and rotation of critical structural parts. The ability of HPFRCC coupling beams to provide acceptable performance is discussed in light of the simulation results.

Introduction

High performance fiber reinforced cementitious composites (HPFRCCs) are a class of materials that have properties that are attractive to structural engineers. These materials are characterized by pseudo-ductile tensile strain hardening behavior after first cracking accompanied by multiple cracks and large energy absorption prior to crack localization (Li 2003 and Naaman 2006). After crack localization, which typically occurs at strains ranging between 0.5% and 4% (about 2 orders of magnitude greater than traditional concrete), the material strain softens gradually and further deformation demand is accommodated within a single growing crack or band of cracks (Kim 2009). In compression, HPFRCC materials behave like confined

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concrete, i.e. the material has greater strength and ductility compared to regular concrete due to the confining effect introduced by the fibers. These properties imply that HPFRCCs have the potential to serve as highly damage tolerant and energy absorbing materials under severe loading conditions, especially seismic loading.

Reinforced concrete (RC) coupled walls are often used in mid- to high-rise structural systems in zones of high seismic risk to provide lateral stiffness. The coupling beams that connect adjacent walls are designed as the main component in the wall system for energy dissipation during strong seismic events. However, traditional RC coupling beams require a dense array of reinforcement in order to satisfy code requirements, which complicates erection of RC coupled wall systems. Experimental studies on individual coupling beams have shown that using HPFRCC to replace traditional concrete material can allow relaxation in reinforcing details while still providing satisfactory performance, i.e. strength and deformation capacity (Parra-Montesinos 2006). To investigate this premise at the system level, this paper numerically studies and compares the performance of a traditional RC 18-story coupled wall systems with another similar system in which the coupling beams and plastic hinge region are HPFRCC instead of RC.

System Design

The theme system considered herein consists of a 94 ft by 78.5 ft (in plan) steel moment resisting frame surrounding a 47 ft by 31.5 ft (in plan) core wall system. The plan view of the prototype structure is given in Fig 1. The total height of the structure is 267 ft. The first floor's height is 15 ft and typical floors are 12 ft high. The thickness of the wall is 20 in and the total structure's weight is 20,831 kips. Two different prototype wall systems are designed. The first is a traditional RC system. The second prototype has HPFRCC coupling beams. In addition, HPFRCC is used in the first four floors of the wall piers.

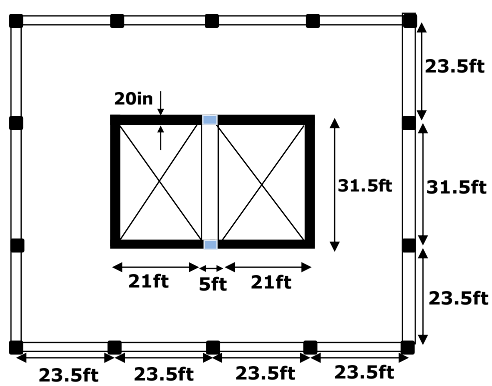


Figure 1. The plan view of the structural system.

The prototype structure is assumed to be an office structure in Los Angeles, and is categorized as Seismic Use Group I. The spectral response accelerations for short periods and at 1 second, Ss and S1, are then decided, respectively. Assuming site class D, the map accelerations are adjusted to the design accelerations, S_{DS} and S_{D1}. The building was classified as Seismic Design Category E. The structure was defined as a dual system due to the structure's frame-wall interaction, and the design coefficients and factors are chosen from Table 4.3-1 in (FEMA-450

2003) accordingly.

Seismic design loads are calculated in accordance with the equivalent lateral force analysis in (FEMA-450 2003). Due to the symmetric nature of the theme system, the numerical analysis is simplified by modeling only half of the structure. To account for cracking and loss of stiffness due to cyclic behavior of concrete walls, effective flexural and axial stiffness of wall piers are taken as 0.70EIg and 1.00EAg (ACI 2008) for the compression wall, and 0.35EIg and 0.35EAg for the tension wall, respectively.

A target coupling ratio of 45% is chosen for the system. The coupling ratio is the proportion of the moment generated by the coupling action to the system overturning moment. To simplify the coupling beam design, the generally reasonable assumption that the theme system does not have significant higher mode effects is made. In addition, it is assumed that the plastic deformation mechanism of the wall system is that plastic hinges form at the base of the shear walls and that all coupling beams yield. Analysis results presented in (Hassan 2005) suggest that these assumptions are fair for the set of 18-story buildings that they considered. The total coupling beam forces are uniformly distributed to each beam, and all coupling beams are therefore designed to have the same cross section. Following (Hassan 2005), the beam shear is

$$V_{beam} = \frac{CR \cdot OTM}{N \cdot S} \tag{1}$$

where CR is the coupling ratio; OTM is the overturning moment; N is the number of stories; S is the distance between wall centroids. An elastic frame model is then numerically constructed to determine wall forces and drifts. In the model, beam-column elements, which are located at the gross section centroid of each wall, are used to represent walls. Coupling beams are represented using rigid body elements with released restraints at both ends and applied coupling beam shear force, V_{beam}, and end moment, M_{beam} at the wall piers. The applied end moment is calculated using

$$M_{beam} = \frac{L_{beam} V_{beam}}{2} \tag{2}$$

where L_{beam} is the length of the coupling beam.

RC coupling beams with diagonal reinforcement cages are designed to be compliant with (ACI 318 2008). The diagonal reinforcement cage for the RC coupling beam consists of 4 #7 bars with a dense array of transverse reinforcement. The concrete strength and steel yield strength are 7 ksi and 60 ksi, respectively. The resulting coupling beam has a thickness of 20 in., depth of 28 in., and aspect ratio of 2.1. Reinforcement details are shown in Fig. 2.

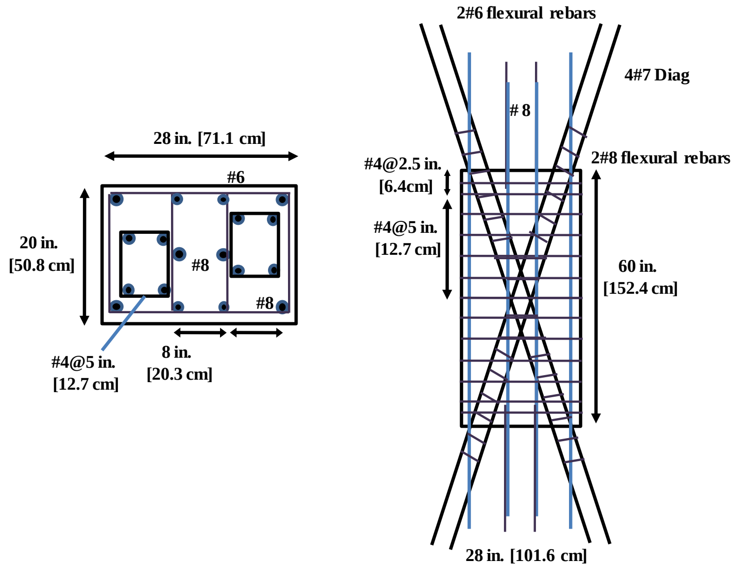


Figure 2. Reinforced concrete coupling beam design.

It has been shown that using HPFRCC can effectively reduce the required reinforcement detailing (Parra-Montesinos 2005), significantly alleviating reinforcement congestion. The simplified reinforcement details for HPFRCC coupling beam design, suggested by (Canbolat, Parra-Montesinos, and Wight 2005), are used here. The resulting HPFRCC coupling beam herein consists of 2 #10 diagonal bars with no confining reinforcement in each direction. The dimension and the amount of vertical reinforcement for the HPFRCC beams is the same as that for the RC beams, while the horizontal reinforcement is reduced for the HPFRCC coupling beams. Details of the HPFRCC coupling beam design are shown in Fig 3.

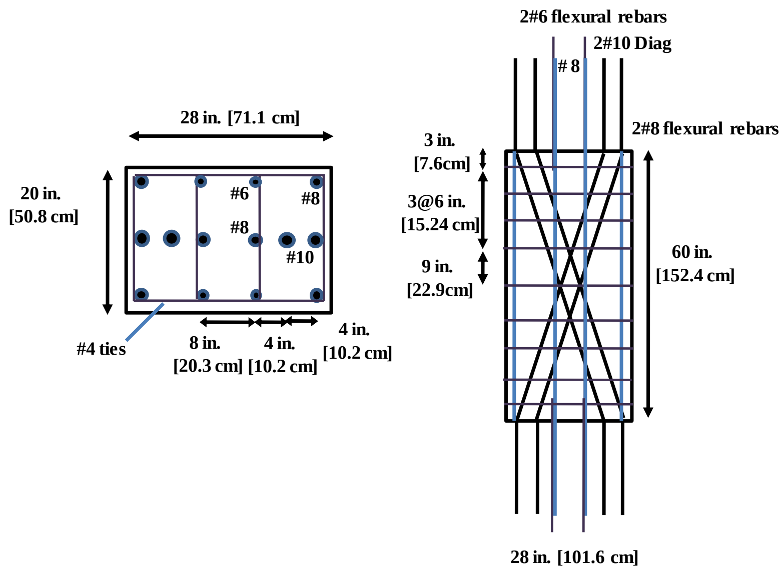


Figure 3. HPFRCC coupling beam design.

The RC wall design details are also compliant with (ACI 318 2008) and are shown in Fig 4 and Table 1; where t_w is the thickness of the wall, and L_{be} is the length of boundary zone. The flexural and horizontal reinforcement of the HPFRCC walls is reduced from that of the RC walls to take into account the additional tensile and shear capacity that HPFRCC can provide.

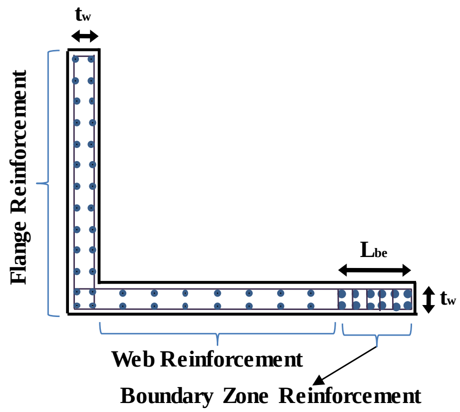


Figure 4. Shear wall cross section detail.

Table 1. Reinforcement details of HPFRCC and RC walls.

System	Floor	t_w(in)	L_{be} (in)	Flange Horiz.	Flange Vert.	Web Horiz.	Web Vert.	Boundary Zone Horiz.	Boundary Zone Vert.
HPFRCC Walls	**17-18**	20	0	#4@12”	#4@12”	#4@12”	#5@8”	-	-
	15-16	20	24	#4@12”	#4@12”	#4@12”	#5@8”	#4@12”	12#7
	13-14	20	32	#4@8”	#4@12”	#4@8”	#5@8”	#4@8”	16#9
	11-12	20	32	#4@8”	#5@12”	#4@8”	#5@8”	#4@8”	16#9
	9-10	20	40	#5@8”	#5@8”	#5@8”	#5@8”	#4@8”	20#9
	7-8	20	40	#5@8”	#6@8”	#5@8”	#5@8”	#5@8”	20#9
	5-6	20	48	#5@8”	#7@8”	#5@8”	#6@8”	#5@8”	24#10
	3-4	20	48	#6@8”	#6@8”	#6@8”	#7@8”	#5@8”	24#11
	1-2	20	48	#6@8”	#8@8”	#6@8”	#7@8”	#5@8”	24#11
RC Walls	**17-18**	20	0	#4@12”	#4@12”	#4@12”	#5@8”	-	-
	15-16	20	24	#4@12”	#4@12”	#4@12”	#5@8”	#4@12”	12#7
	13-14	20	32	#4@8”	#4@12”	#4@8”	#5@8”	#4@8”	16#9
	11-12	20	32	#4@8”	#5@12”	#4@8”	#5@8”	#4@8”	16#9
	9-10	20	40	#5@8”	#5@8”	#5@8”	#5@8”	#4@8”	20#10
	7-8	20	40	#5@8”	#6@8”	#5@8”	#5@8”	#5@8”	20#10
	5-6	20	48	#5@8”	#7@8”	#5@8”	#6@8”	#5@8”	24#11
	3-4	20	48	#6@8”	#8@8”	#6@8”	#7@8”	#5@4”	24#14
	1-2	20	48	#6@8”	#9@8”	#6@8”	#7@8”	#5@4”	24#14

Finite Element Modeling

A hybrid rotating/fixed crack model suitable for plane stress elements for simulating HPFRCC and reinforced concrete materials, developed by (Hung and El-Tawil 2009), is used to model the prototype structures. The developed material models were shown to be accurate and robust through extensive comparisons between experimental and computational results of hysteresis behavior and crack patterns from several types of structural components, including a coupling beam, shear wall, and double cantilever beam (Hung and El-Tawil 2009). Gravity loads are first applied to the prototype structures before seismic excitation is imposed. In addition, a 5% damping ratio was introduced in the analysis. The theme structure models were simulated using LS-DYNA, an explicit finite element software package.

Analysis Results

Performance of the systems is evaluated using nonlinear dynamic time history analysis. A simulated time history ground motion record, LA34, representing a seismic hazard level with probability of exceedance of 2% in 50 years (2/50) is used herein. The ground motion LA34, developed by the SAC (Acronym for *Structural Engineers Association of California* 'SEAOC', *Applied Technology Council* 'ATC' and *California University for Researc*h in Earthquake Engineering 'CUREe') steel project, has a PGA of 0.68g. To reduce the computational effort, only the strong motion duration of the ground motion record is adopted for seismic excitation. The strong motion duration is defined as the time segment in which 90 percent of the seismic energy is delivered (Hassan 2005). The resulting strong motion duration for LA34 is 12.5 sec.

The desired performance levels of the systems are evaluated in accordance with (FEMA-356 2000), which specifies that a building structural system must be able to deliver Collapse Prevention performance for a 2/50 event. The prototype structures are expected not to collapse in spite of significant damage from severe and infrequent earthquake events, i.e. 2/50 events.

The roof displacement histories from the two systems are plotted in Fig. 5. It can be seen that both system experience similar peak roof displacements despite key differences in the reinforcing steel quantities and detailing. Details of component responses, drawn at the time corresponding to the occurrence of maximum system response (at time step 10.2 sec) during the earthquake, are plotted in Figs. 6 and 7. Fig. 6(a) shows the story drifts of both systems. It shows that the story drifts of the two systems are approximately the same along the building height. The drift limits in (ASCE 7 2002) imply that a maximum interstory drift of 2% shall not be exceeded at the Lift Safety performance level, while this criterion does not need to be observed at Collapse Prevention level. Nevertheless, the maximum interstory drifts of both prototype structures are below the 2% drift limit. These results suggest that the HPFRCC system is capable of delivering similar performance to the traditional RC system in spite of the reduced reinforcement quantity and significantly relaxed reinforcement detailing.

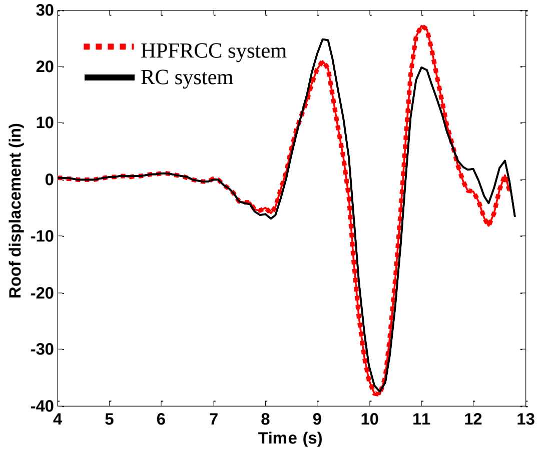
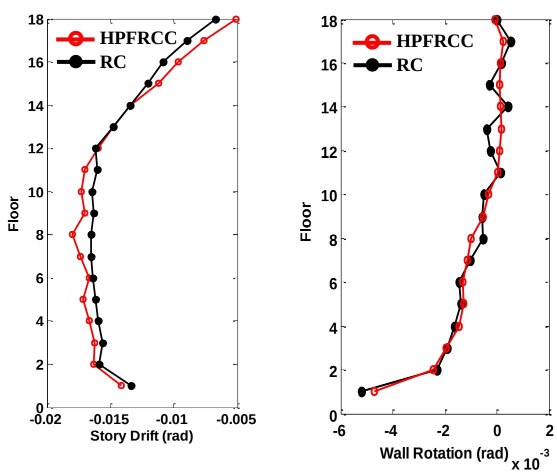


Figure 5. The time history response of RC and HPFRCC composite coupled wall systems.

Wall rotations are plotted in Fig. 6(b). As can be seen from the figure, both systems yield similar wall rotations along the height of the building. It is also clear that plastic hinge forms at the base of both systems when peak displacement is reached. Rotation of the coupling beams from both systems is plotted in Figs. 7. It is concluded from the figure that despite the greatly reduced reinforcement amount used for HPFRCC coupling beams, replacing RC coupling beams with HPFRCC coupling beams is able to maintain the rotation demand.



(a) story drift (b) wall rotation

Figure 6. Interstory drift and wall rotation levels at peak drift.

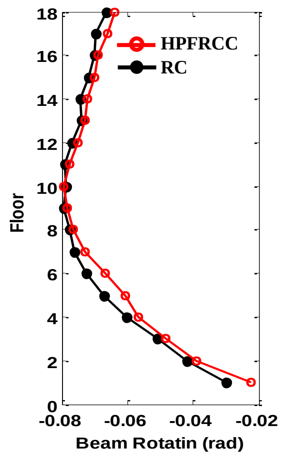
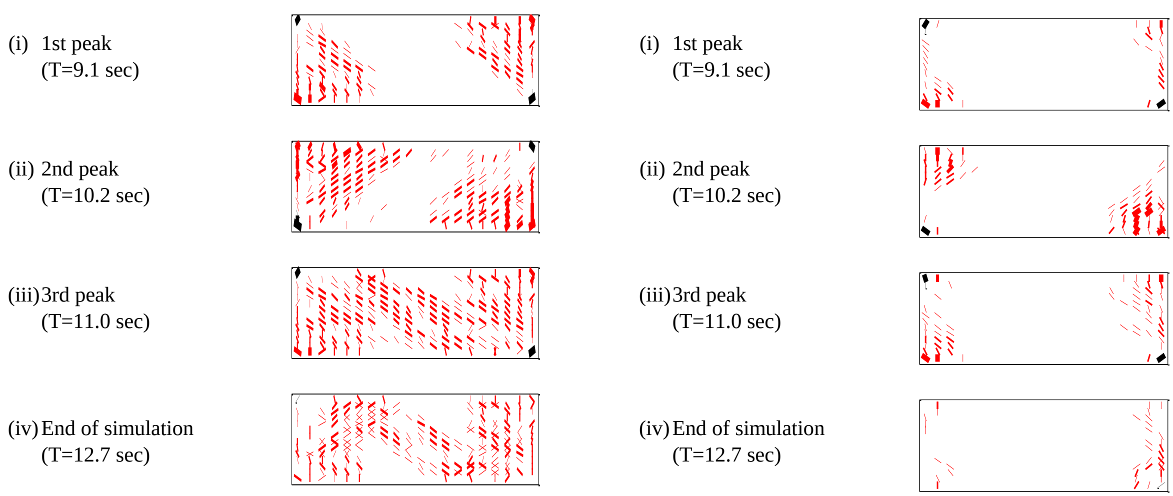


Figure 7. Coupling beam rotations.

Fig. 7 implies that the coupling beams between 8th floor to 12th floors are likely subjected to more damage compared to other coupling beams. The crack patterns of the coupling beams at the 12th floor from various stages (corresponding to different peak values of the roof displacement) during the simulated earthquake are plotted in Fig. 8, where the thickness of the lines indicating cracks is proportional to crack width, while black dots represent concrete crushing. It can be seen that the cracking damage in the RC coupling beams are more severe and widespread than that in HPFRCC coupling beams. Moreover, at the end of the ground excitation, the residual cracks in HPFRCC coupling beams are much less. It is therefore suggested that HPFRCC coupling beams should require less repair than RC coupling beams after a strong seismic event.



(a) RC coupling beams (b) HPFRCC coupling beam

Figure 8. Crack patterns of RC and HPFRCC coupling beams at the 12th floor at various stages of the simulation.

Conclusions

The effectiveness of using HPFRCC material in the critical portions of an 18-story coupled wall system is evaluated. Two coupled wall systems are considered: the first is a traditional RC design, while the second is a modified design in which HPFRCC is used in the plastic hinge regions of the wall piers and in the coupling beams. The latter system is designed with less steel and relaxed detailing than the former in recognition of the beneficial effects of HPFRCC. The response of both systems subjected to strong seismic event with a 2% probability of exceedance in 50 years is then compared. Comparisons of seismic response are made using story drift, wall rotation, and rotation of the coupling beams. The performance parameters studied suggest that both systems deliver similar performance in spite of the reduced reinforcement quantity and relaxed detailing in the HPFRCC system compared to the RC system. The advantage of using HPFRCC for the coupled walls is also evident in the coupling beam crack patterns of both systems which suggest that at the end of the seismic event, the HPFRCC coupling beams remain significantly more intact than the RC coupling beams, likely requiring less repair. This study was conducted with only one seismic event. The authors are currently expanding the study to investigate system responses in a stochastic manner, using a battery of seismic inputs representing various hazard levels.

Acknowledgments

The research described herein was sponsored in part by the National Science Foundation under Grant No. CMS 0530383, the University of Michigan, and the Studying Abroad grant from the Taiwanese Government. The opinions, findings, and conclusions expressed in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

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