13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 510

SEISMIC BEHAVIOR OF LOW-RISE WALLS CONSTRUCTED WITH STRAIN-HARDENING FIBER REINFORCED CEMENT COMPOSITES

Gustavo J. PARRA-MONTESINOS[1], and Kwang Y. KIM[2]

SUMMARY

The use of strain-hardening or high-performance fiber reinforced cement composites (HPFRCCs) in low-rise structural walls is evaluated through the testing of two wall specimens with a height-to-length ratio of 1.5 under displacement reversals. Two HPFRCC materials were used in this investigation, which contained either a 1.5% volume fraction of ultrahigh molecular weight polyethylene fibers or hooked steel fibers in a 2.0% volume fraction. Web reinforcement in ratios lower than the minimum values specified in the 2002 ACI Code was provided and the wall specimens were designed such that a diagonal tension failure would occur. Experimental results indicate that HPFRCC walls possess excellent strength, displacement capacity and damage tolerance. Both specimens sustained a peak shear stress of 3.6 MPa up to the cycles to 2.5% drift, during which fiber pullout occurred. Based on their post-cracking strength, the contribution of the HPFRCC material to shear strength was estimated as 70% of the total wall strength. A comparison between wall shear distortions and observed damage indicated that shear distortion limits of 0.5% and 1.5% could be associated with minor cracking and the initiation of severe wall damage, respectively.

INTRODUCTION

Structural walls are often used in low-rise buildings located in regions of high seismicity to provide the necessary strength and stiffness to sustain the lateral loads induced by ground motions. Because of their low height-to-length ratio, low-rise or squat walls tend to exhibit a shear-dominant response with limited drift capacity. During the past fourty years, several research programs have been conducted to evaluate the behavior of reinforced concrete (RC) low-rise walls subjected to displacement reversals. Cardenas [1] showed that the shear strength of low-rise walls can be satisfactorily predicted by assuming "concrete" and "steel" contributions as typically done for slender RC beams. Based on experimental results, it was concluded that well detailed RC low-rise walls could sustain average shear stresses as high as $0.83\sqrt{f'_c}$ (MPa), where f'_c is the compressive strength of the concrete. More recently, Wood [2] recommended an average shear stress of $0.5\sqrt{f'_c}$ (MPa) as a lower bound for the shear strength of squat walls with minimum web reinforcement, based on experimental results from the tests of 143 low-rise walls.

[1] Assist. Prof., Dept. of Civil and Environ. Eng., Univ. of Michigan, Ann Arbor, MI 48109-2125, USA
[2] Post-Doctoral Research Associate, School of Civil Eng., Purdue Univ., West Lafayette, IN 47907, USA

Although RC squat walls have been shown to exhibit excellent shear strength, attention must also be paid to their drift capacity if inelastic deformations are expected during earthquake events. Results from several experimental investigations [3-5] have shown that significant damage and strength loss in low-rise walls can occur at drifts below 1.0%. In some instances, drift capacities as low as 0.5% have been observed in laboratory tests [4-5]. Given this limited displacement capacity, sufficient wall area should be provided in low-rise structures to ensure that drift demands are sufficiently low to avoid excessive wall damage during a seismic event. Alternatively, low-rise walls can be designed to remain elastic during ground motions. However, this approach is generally not attractive because of economical and space usage constraints.

During the last few years, significant advancements have been made in the field of fiber reinforced cement composites (FRCCs). Nowadays, it is possible to achieve a tensile strain-hardening response with fiber volume fractions ranging between 1.5-2.0% [6-7], as opposed to traditional FRCCs which exhibit a drop in tensile strength after first cracking (Fig. 1). Because of their substantially larger strain capacity and toughness, strain-hardening FRCCs are generally referred to as High-Performance Fiber Reinforced Cement Composites (HPFRCCs) [8]. HPFRCC materials possess tensile strain capacities of 100-300 times that of plain concrete. In addition, as HPFRCC materials strain-harden, a multiple crack formation process takes place, leading to a large array of hairline cracks at large strains, as opposed to a few wide cracks observed in regular FRCC or reinforced concrete members. In compression, HPFRCC materials behave like well confined concrete with strain capacities larger than 1.0%.

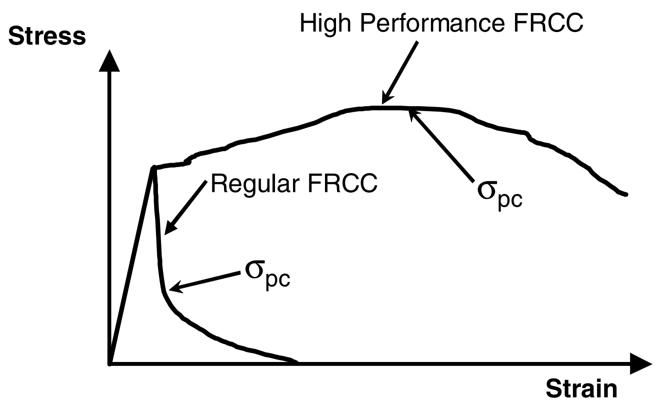


Figure 1 - Tensile stress-strain response of regular and high-performance FRCCs

The excellent tensile and compression behavior exhibited by HPFRCCs has led researchers to investigate their use in structures subjected to extreme loading conditions, such as those induced by earthquakes [9-11]. In this paper, the use of HPFRCC materials in seismic-resistant low-rise walls is evaluated. Particular emphasis is placed on the drift at which fiber pullout occurs, as well as in the contribution of the HPFRCC material to wall shear strength.

EXPERIMENTAL PROGRAM

Test Specimens

Two structural walls constructed with strain-hardening or HPFRCCs were tested under displacement reversals. The two specimens consisted of a cantilever wall specimen with a shear span-to-depth ratio of 1.5, a bottom RC block representing a stiff foundation, and an RC loading beam at the top of the wall. Fig. 2 shows a sketch of the wall specimens tested in this investigation. The structural walls had a 100 x 1000 mm cross section and were "identical" except for the fiber cementitious material used. Reinforcement in the wall edges consisted of 3-#19M bars, which represented a tension wall reinforcement ratio of approximately 1.3%. Horizontal and vertical web reinforcement consisted of #6M bars spaced at 250 and 150 mm, respectively. This distributed reinforcement translated into horizontal and vertical web reinforcement ratios of 0.13 and 0.21%, respectively, which are lower than the minimum of 0.25% specified in the ACI Building Code [12]. Fig. 2b shows the reinforcement layout for Specimens 1 and 2. Wall Specimen 1 was constructed with a HPFRCC material reinforced with ultrahigh molecular weight polyethylene (PE) fibers in a 1.5% volume fraction, while Specimen 2 contained hooked steel fibers in a 2.0% volume fraction.

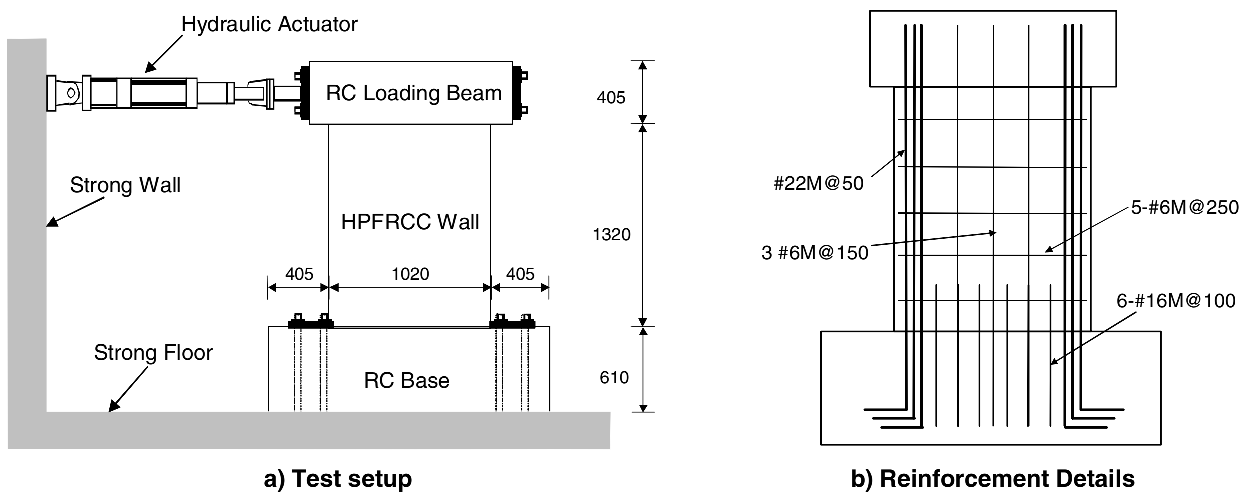


Figure 2 - Wall test setup and reinforcement details

The two wall specimens were designed to exhibit a diagonal tension failure with limited flexural yielding in order to better evaluate the shear strength and distortion capacity of HPFRCC low-rise walls. For estimation of the shear strength of HPFRCC walls, the "concrete" contribution to shear strength was determined assuming a critical crack oriented at 45 degrees and a material post-cracking strength, σ_{pc}, equal to 3.0 MPa. In addition, the effective shear depth was estimated to be 80% of the wall length. For design purposes, the compression strength of the HPFRCC material and the yield strength of the steel reinforcement were assumed to be 35 MPa and 450 MPa, respectively. Thus, any increase in shear strength due to arch action, dowel action or shear carried in the compression zone was ignored.

The typical construction sequence for structural walls involves separate concrete casting operations for the foundation and walls, leading to a construction joint at the wall base. In regular concrete construction, this construction joint would not pose major structural concerns. However, because of the superior tensile response exhibited by HPFRCC materials, the presence of a cold joint would lead to a weak plane at the wall base because no fibers would bridge such cold joint. In order to avoid large inelastic deformations concentrated at the wall base cold joint, six 200 mm long #16M dowel bars were provided at the wall base. Besides increasing flexural strength, these dowels would provide increased resistance against sliding shear at the wall base.

Displacement History, Test Setup and Instrumentation

The two wall specimens were subjected to lateral displacement cycles ranging from 0.125% up to 2.5% drift. Fig. 3 shows the lateral displacement history applied to the test specimens. Lateral displacements were applied through a 450 kN hydraulic actuator connected to the RC loading beam at one end and a strong reaction wall at the other end (Fig. 2a). Two cycles were applied to each drift level up to 2.0% drift in order to evaluate the loss of strength and stiffness during repeated cycles at the same drift level.

Applied lateral loads and displacements were monitored through a load cell and displacement transducer, respectively. Average shear distortions and rotations were measured through displacement transducers oriented vertically, horizontally and at a 45 degree angle. Strains in wall edge and web reinforcement were monitored through strain gages.

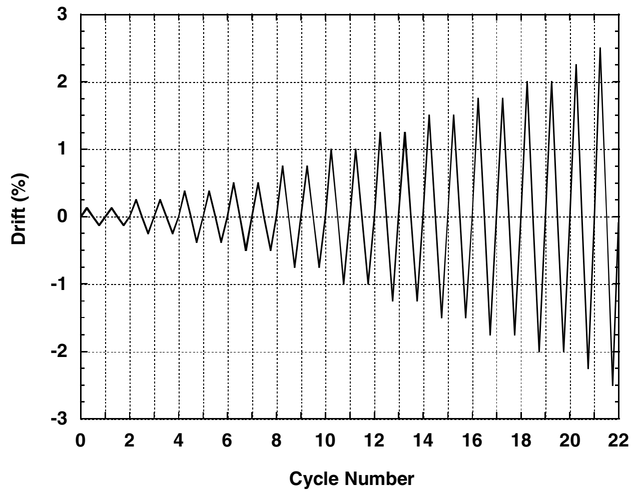


Figure 3 - Lateral displacement history

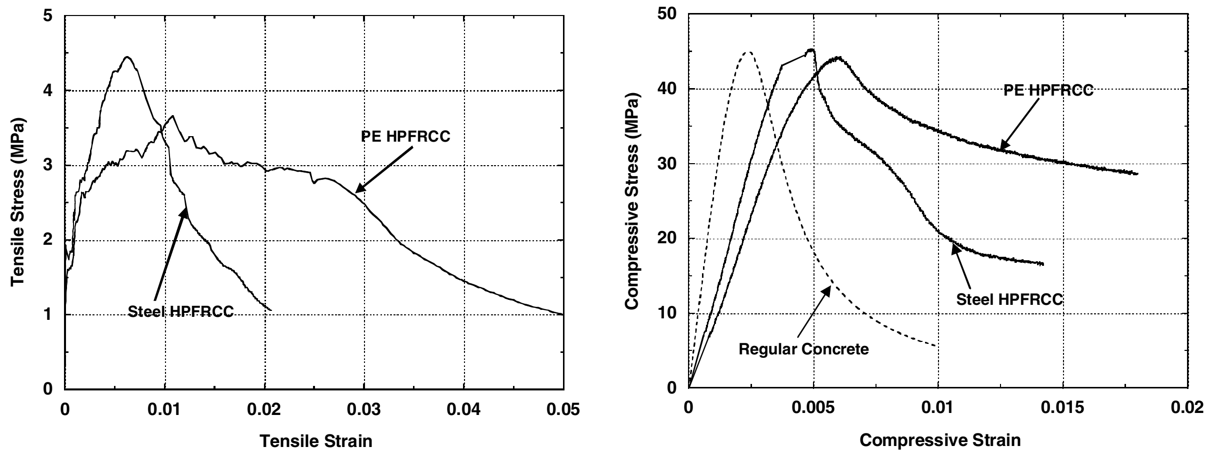
Material Properties

Two HPFRCC materials were used in this investigation: 1) HPFRCC with a 1.5% volume fraction of ultrahigh molecular weight polyethylene (PE) fibers, and 2) HPFRCC with 2.0% volume fraction of hooked steel fibers. Table 1 lists the main properties of the fibers used in this investigation. For the PE and steel HPFRCC materials, mortar matrices with a mix ratio by weight (cement: sand: water: fly ash) of 1:1:0.5:0.15 and 1:2:0.48:0.2 were used, respectively.

Table 1 - Properties of polyethylene (PE) and steel fibers

Fiber Type	Tensile Strength (MPa)	Elastic Modulus (GPa)	Diameter (mm)	Length (mm)
PE	2570	117	0.038	38
Steel	≈ 1200	200	0.5	30

In order to evaluate the tensile behavior of the HPFRCC materials, 25x25 mm dog-bone specimens and 100x100 mm beams were tested under direct tension and flexure, respectively. Figs. 4a and 4b show typical stress-strain responses obtained from direct tension tests for both HPFRCC materials. As can be seen in Fig. 4a, the PE HPFRCC material exhibited a more ductile response compared to the steel HPFRCC. Even though a gradual decay in tension strength occurred at strains larger than 1.0%, the PE HPFRCC material showed excellent post-peak behavior up to 3.0% tensile strain, the deformation at which significant fiber pullout occurred with the associated decrease in tension capacity. Comparing the post-cracking response of steel and PE HPFRCCs, it is clear that the steel HPFRCC material possessed substantially larger stiffness up to peak strength. However, damage localization under direct tension occurred at a tensile strain of about 0.7%. From direct tension tests, average post-cracking strengths of 3.5 and 4.5 MPa were obtained for the PE and steel HPFRCC materials, respectively. However, slightly lower strengths were obtained from bending tests, with average post-cracking strengths of 3.1 MPa and 3.2 MPa for the PE and steel fiber cement composites, respectively.



a) Tensile Behavior b) Compressive Behavior

Figure 4 - Stress-strain behavior of PE and steel HPFRCCs

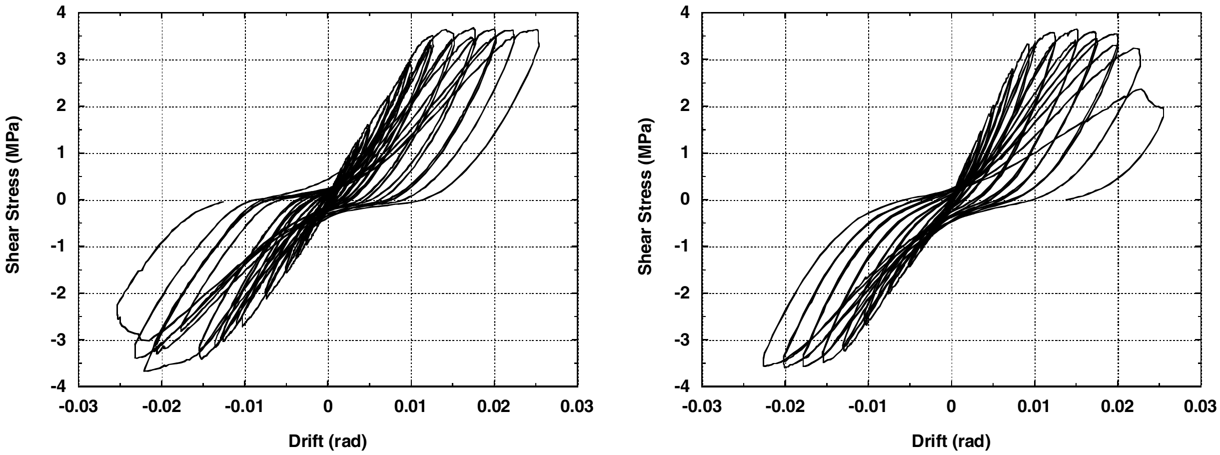
Cylinder tests were also conducted in order to evaluate the stress-strain response of PE and steel HPFRCC materials. Average compressive strengths of 44.1 and 46.8 MPa were obtained for the PE and steel HPFRCC materials, respectively. Fig. 4b shows typical stress-strain curves for the two HPFRCCs used in this investigation. For comparison purposes, an idealized stress-strain response obtained by using the constitutive model proposed by Ahmad [13] is also shown in the figure. As can be observed, both HPFRCC materials are clearly more flexible than regular concrete. However, it is also clear that unconfined HPFRCC materials exhibit a relatively flat post-peak response, which is typical of well confined concrete. Another aspect that must be noticed is the compression strain capacity of HPFRCC materials. For the particular case of the PE and steel HPFRCCs used in this research, strain capacities larger than 1.0% were obtained. At 1.0%, the PE HPFRCC had retained about 75% of its peak strength, while the stress in the steel HPFRCC corresponded to approximately 45% of the maximum compressive strength.

Grade 420M deformed steel bars were used for the wall boundary longitudinal reinforcement and dowel bars in both test specimens. The yield strength obtained from tensile tests was 450 MPa and 475 MPa for the #16M and #22M bars, respectively. The yield strength of the smooth #6M bars used as distributed web reinforcement was not available. However, a yield strength of 275 MPa was estimated based on strain gage readings obtained during the tests.

EXPERIMENTAL RESULTS

Overall Behavior

Figs. 5a and 5b show the average shear stress versus drift response for Specimens 1 and 2, respectively. As can be observed, both walls sustained displacement demands up to 2.5% drift, the displacement at which fiber pullout occurred. Given the low drift demands imposed in low-rise wall systems, a 2.5% drift capacity should be well above reasonable drift demands expected during a major earthquake. It is worth mentioning that the wall specimens were designed to fail in diagonal tension with limited flexural yielding. Thus, it is reasonable to think that even larger drift capacities could have been obtained if the walls were proportioned such as to undergo substantial flexural yielding. In terms of shear strength, the wall specimens sustained a peak shear stress demand of 3.6 MPa, even though web reinforcement ratios lower than 0.25% were provided.



a) Specimen 1 b) Specimen 2

Figure 5 - Average shear stress versus drift response

The “pinched” hysteretic behavior shown in Fig. 5 was associated with the shear-dominated response exhibited by the two wall specimens. During lateral loading in a given direction, stresses across cracks were primarily resisted by the fibers in the composite. However, during the unloading-reversed loading process, the fibers were ineffective to transfer compression stresses. Thus, full closing of diagonal cracks was required in order for the wall to regain stiffness.

In terms of damage progress in the test specimens, wall diagonal cracking was first observed at about 0.25% drift. As the test progressed, a dense array of hairline diagonal cracks formed, with a maximum crack width of about 0.7 mm at 1.0% drift. At this displacement level, only minor damage was observed in the two wall specimens (Fig. 6). For larger drifts, an increase in diagonal crack width was observed, with a maximum crack width at 1.5% drift of 1.5 and 2 mm for Specimens 1 and 2, respectively. Damage localization (single crack opening) began at 1.5% drift for Specimen 2 and at 2.0% drift for Specimen 1. The larger drift at localization of damage for the PE HPFRCC material was expected given the superior tensile response observed during the direct tension tests (Fig. 4a). At 2.25% drift, both wall specimens were sustaining significant damage (Fig. 7) with wide diagonal cracks that ultimately led to a diagonal tension failure due to fiber pullout during the 2.5% drift cycle. At the end of the test, only minor damage in the extreme fibers at the wall base was observed, even though peak average compressive strains of 1.5 and 0.54% were measured in Specimens 1 and 2, respectively. Based on the damage observed in the two wall specimens, it is clear that PE fibers in a 1.5% volume fraction were more effective than hooked steel fibers in a 2.0% volume fraction in terms of reducing crack spacing and width, and thus in increasing damage tolerance in the wall specimens. However, there was no significant difference in the overall hysteretic response of the wall specimens, as shown in Fig. 5.

a) Specimen 1 b) Specimen 2

Figure 6 - Cracking pattern at 1.0% drift

a) Specimen 1 b) Specimen 2

Figure 7 - Cracking pattern at 2.25% drift

Shear Distortions

Because the two wall specimens had a shear span-to-depth ratio of 1.5 and were designed such that a diagonal tension failure would occur, shear distortion measurements were used to establish a correlation with observed damage and to recommend deformation limits associated with various performance states. Figs. 8a and 8b show the average shear stress versus shear distortion response for Specimens 1 and 2, respectively. As can be observed, the two specimens exhibited good strength retention capacity up to a shear distortion of about 1.75%, the deformation at which fiber pullout with a sudden shear strength drop occurred. As expected, a “pinched” hysteretic response was obtained due to the fact that PE and steel fibers are not effective in transferring compressive stresses through cracks. By comparing the measured shear distortions with the observed damage, a shear distortion limit of 0.5% could be associated with minor damage or an Immediate Occupancy performance state. For a shear distortion of 1.5% (2.0% drift), damage was considered moderate to severe with crack widths of about 4 mm. This damage intensity could be linked to a Collapse Prevention performance level. For larger distortion demands, a rapid growth of diagonal cracks occurred, which led to a sudden failure during the cycle to 2.5% drift.

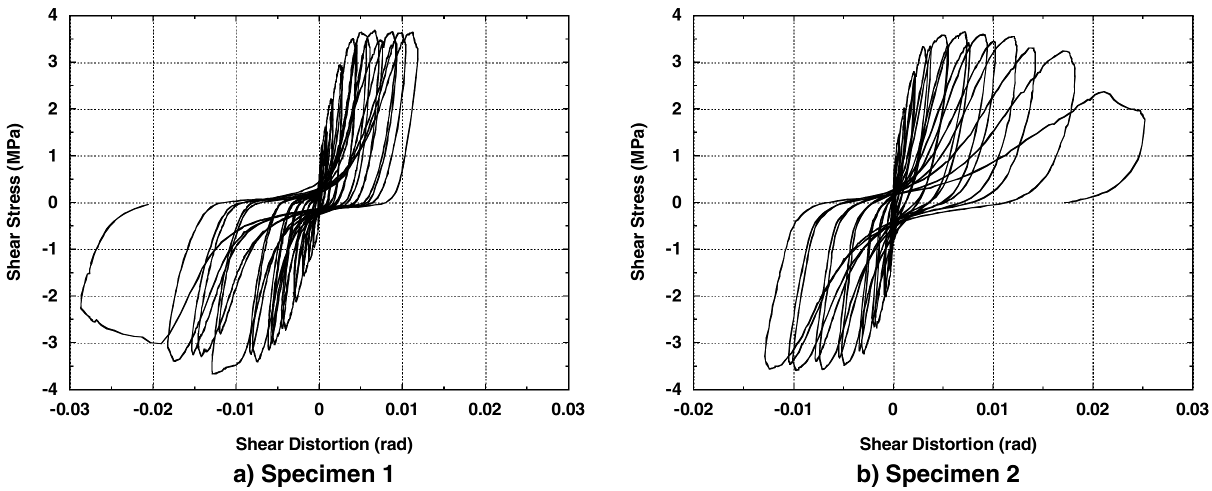


Figure 8 - Average shear stress versus shear distortion response

From the average strain measurements obtained from linear potentiometers oriented vertically, horizontally, and at 45 degrees, the state of strain in the web panel was determined at different stages during the tests. At large drift levels, the ratio between principal tensile and compression strains was in the order of 6 with a nearly 45 degree principal angle. This suggests that the shear deformation capacity of low-rise walls could be conservatively estimated as the HPFRCC material tensile strain capacity at damage localization. Specimens 1 and 2 showed a good shear strength retention capacity up to a shear distortion of about 1.75% and a principal tensile strain of 1.5%. Comparing the principal tensile strains at wall failure with the strain capacity obtained from material tests, it can be noted that the tensile strain capacity of Specimen 1 with PE fibers exceeded that at peak post-cracking strength from dog-bone tests. However, this material exhibited a nearly constant stress region up to a strain of about 2.0% (Fig. 4a). Specimen 2, with hooked steel fibers, also exhibited a good strength retention capacity at tensile strains exceeding the average strain capacity obtained from several dog-bone specimens. Although the results indicate that the shear distortion capacity of HPFRCC low-rise walls can be conservatively estimated as the strain at peak strength from direct tensile tests, it is clear that research is needed in order to accurately estimate the strain capacity of HPFRCC materials in large-scale structural members.

Based on the above, the shear distortion capacity of HPFRCC low-rise walls with a height-to-length ratio equal to or greater than 1.0 could be conservatively estimated as the tensile strain capacity of the fiber cementitious material. For more squat walls, the shear distortion capacity can be estimated as (material tensile strain capacity) x (sin 2θ), where the principal compression angle θ can be assumed to be equal to atan (wall height-to-length ratio).

Strains in Steel Reinforcement

The vertical and horizontal web reinforcement ratios in the wall specimens were 0.21 and 0.13%, respectively, and no lateral confinement was used in the boundary regions. Readings from linear strain gauges attached to the distributed vertical reinforcement in both walls revealed that most of the reinforcement yielded during the cycles ranging from 0.75% to 1.25% drift. In both specimens, yielding of horizontal reinforcement was observed during the cycles to 1.0% and 1.5% drift (shear distortions between 0.3% and 0.7%). However, because the PE and hooked steel fibers were effective in bridging the cracks at large average tensile strains, the two test specimens could sustain substantially larger drifts with good strength and stiffness retention up to fiber pullout at 2.5% drift. With regard to the dowel reinforcement at the wall base, maximum tensile strains of 0.1 and 0.12% were measured in Specimens 1 and 2, respectively. Yielding in the main vertical reinforcement at the wall edges was first observed at drifts ranging between 0.75 and 1.0% for both test specimens. Towards the end of the tests, flexural deformations at the wall base accounted for approximately 40% of the applied drift.

Shear Strength of HPFRCC Low-Rise Walls

The ACI Code [12] provides guidelines for estimating the shear strength of seismic-resistant structural walls. The approach taken in the ACI Code is similar to that used in slender flexural members, where member shear strength is assigned to a "concrete" and a "steel" contribution. In order to account for arch action in walls with low height-to-length ratios, the concrete contribution is increased from the typical shear stress value of $(1/6)\sqrt{f'_c}$ (MPa) for walls with aspect ratios larger than 2.0 to $(1/4)\sqrt{f'_c}$ for wall height-to-length ratios equal to or smaller than 1.5.

Because of the strain-hardening response of HPFRCC materials, the concrete contribution to shear strength of HPFRCC walls is proposed to be determined assuming a 45 degree crack with a fiber bridging stress equal to the post-cracking material strength. Thus, the shear strength of HPFRCC walls can be determined as,

$$V_n = (\sigma_{pc} + \rho_n f_y)(0.8 A_{cv}) \qquad (1)$$

where σ_{pc} is the fiber cementitious material peak post-cracking strength, ρ_n and f_y are the web horizontal reinforcement ratio and yield strength, respectively, and A_{cv} is the wall cross-sectional area (wall length x thickness). In Eq. (1), the wall effective shear depth has been approximated as 80% of the wall length. In general, for wall aspect ratios lower than 0.8, $(0.8A_{cv})$ should be replaced by the product of the wall height and wall thickness. Due to the low aspect ratio of the walls tested in this investigation, it is likely that a larger concrete contribution would occur due to arch action. However, this contribution is neglected in the proposed strength equation. It should also be noted that no shear strength contribution from the member compression zone and dowel action is taken into account. Thus, the shear strength of HPFRCC walls estimated using Eq. (1) should represent a lower bound of the actual wall shear strength.

Evaluating Eq. (1) with actual material properties, the predicted shear strength of Specimens 1 and 2 was 285 kN and 300 kN, respectively, and approximately 90% of the predicted strength (approximately 260 kN and 2.5 MPa of average shear stress) corresponded to the contribution of the fiber cementitious material. It should be mentioned that the values of post-cracking strength obtained from beam tests were used because they represent the behavior of larger scale specimens. Comparing the predicted strength with the measured peak load of 370 kN, a conservative estimation of the wall shear strength was obtained, as expected, the predicted shear strength representing approximately 80% of the actual wall strength.

SUMMARY AND CONCLUSIONS

This paper presents results from the tests of two low-rise walls constructed with strain-hardening or high-performance fiber reinforced cement composites (HPFRCCs) under displacement reversals. The materials used in this investigation contained either ultrahigh molecular weight polyethylene fibers or hooked steel fibers in volume fractions of 1.5% and 2.0%, respectively. Wall web reinforcement in ratios lower than the minimum specified in the 2002 ACI Code were provided. In addition, both specimens were designed to fail in diagonal tension in order to better evaluate the contribution of HPFRCC materials to wall shear strength and distortion capacity.

Experimental results indicate that the use of HPFRCC materials in lightly reinforced low-rise walls represents a viable alternative to ensure adequate behavior during seismic events. The two specimens tested in this investigation failed in diagonal tension due to fiber pullout during the cycles to 2.5% drift, indicating excellent displacement capacity given the member low aspect ratio and limited flexural yielding intended in the specimen design. Based on the strain-hardening response of HPFRCC materials, the contribution of the fiber cementitious material to member shear strength was estimated as approximately 70% of the total wall strength. In addition, only minor to moderate damage was observed up to 1.5% drift, displacement that could be considered a large drift for a low-rise wall structure. Polyethylene fibers in a 1.5% volume fraction were more effective than hooked steel fibers in a 2.0% volume fraction in terms of reducing crack spacing and width, and thus in increasing damage tolerance in the wall specimens. However, there was no significant difference in the overall hysteretic response of the wall specimens.

REFERENCES

1. Cardenas, A. E., Hanson, J. M., Corley, W. G., and Hognestad, E. (1973). "Design provisions for shear walls," *ACI Journal*, Vol. 70, No. 3, pp. 221-230.
2. Wood, S. L. (1990). "Shear strength of low-rise reinforced concrete walls," *ACI Structural Journal*, Vol. 87, No. 1, pp. 99-107.
3. Barda, F., Hanson, J. M., and Corley, W. G. (1977). "Shear strength of low-rise walls with boundary elements," *Reinforced Concrete Structures in Seismic Zones*, SP-53, American Concrete Institute, Detroit, pp. 149-202.
4. Paulay, T., Priestley, M. J. N., and Synge, A. J. (1982). "Ductility in earthquake resisting squat shear walls," *ACI Journal*, Vol. 79, No. 4, pp. 257-269.
5. Hidalgo, P. A., Ledezma, C. A., and Jordan, R. M. (2002). "Seismic behavior of squat reinforced concrete shear walls," *Earthquake Spectra*, Vol. 18, No. 2, pp. 287-308.
6. Li, V.C. (1993). "From micromechanics to structural engineering – the design of cementitious composites for civil engineering applications," *JSCE Journal of Structural Mechanics and Earthquake Engineering*, Vol. 10, No. 2, pp. 37-48.
7. Naaman, A. E. (1999). "Fibers with slip hardening bond," *High Performance Fiber Reinforced Cement Composites (HPFRCC 3)*, *Proceedings of the Third International RILEM Workshop*, Mainz, Germany, May 1999, Ed. H. W. Reinhardt and A. E. Naaman, RILEM Publications S.A.R.L., Cachan Cedex, France, pp. 371-385.
8. Naaman, A. E., and Reinhardt, H. W. (1996). "Characterization of high performance fiber reinforced cement composites – HPFRCC," *High Performance Fiber Reinforced Cement Composites 2 (HPFRCC 2)*, *Proceedings of the Second International RILEM Workshop*, Ann Arbor, USA, June 1995, Ed. A.E. Naaman and H.W. Reinhardt, E & FN Spon, London, UK, pp. 1-24.
9. Parra-Montesinos, G., and Wight, J.K. (2000). "Seismic response of exterior RC column-to-steel beam connections," *ASCE Journal of Structural Engineering*, Vol. 126, No. 10, pp. 1113-1121.
10. Parra-Montesinos, G. (2003). "HPFRCC in earthquake resistant structures: current knowledge and future trends," *High Performance Fiber Reinforced Cement Composites 4 (HPFRCC 4), Proceedings of the Fourth International RILEM Workshop*, Ann Arbor, USA, June 2003, Ed. A.E. Naaman and H.W. Reinhardt, RILEM Publications S.A.R.L., Cachan Cedex, France, pp. 453-472.
11. Wight, James K., Parra-Montesinos, Gustavo J., and Canbolat, B. Afsin. (2003). "Seismic behavior of fiber reinforced cement composite coupling beams," *International Symposium Honoring Shunsuke Otani*, Performance-Based Engineering for Earthquake Resistant Reinforced Concrete Structures, Japan, September 2003, pp. 357-371.
12. ACI Committee 318. (2002). *Building code requirements for structural concrete (318-02) and commentary (318R-02)*, American Concrete Institute, Farmington Hills, Michigan.

13. Ahmad, S.H. (1981). “Properties of confined concrete under static and dynamic loads,” *Ph.D. thesis* submitted to the University of Illinois at Chicago Circle, Department of Materials Engineering, Chicago, Illinois.