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SHEAR STRENGTH AND DRIFT CAPACITY OF HIGH-PERFORMANCE FIBER REINFORCED CONCRETE LOW-RISE WALLS SUBJECTED TO DISPLACEMENT REVERSALS

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ABSTRACT

The use of tensile strain-hardening, High-Performance Fiber Reinforced Concrete (HPFRC) in low-rise structural walls as a means to simplify reinforcement detailing without compromising wall seismic behavior was experimentally evaluated. Nine cantilever low-rise wall specimens with shear span-to-wall length ratio of either 1.2 or 1.5 and shear stress demands ranging from $0.38\sqrt{f_c'}$ to $0.79\sqrt{f_c'}$ (MPa) were tested under large displacement reversals. A total of five HPFRC walls and four regular reinforced concrete (RC) specimens were tested. The RC walls were detailed according to the seismic provisions of the 2005 ACI Building Code. The HPFRC walls, on the other hand, had little or no confinement reinforcement in the boundary regions and a reduced web distributed reinforcement compared with that in the RC walls. The RC specimens exhibited drift capacities ranging between 1.5% and 2.3%, whereas the HPFRC walls sustained drifts of up to 3%. No indication of early concrete crushing or buckling of the longitudinal reinforcement was observed in the boundary regions of the HPFRC walls, even when no confinement reinforcement was used. A dense array of narrow cracks was observed in the HPFRC specimens in contrast with a few wide cracks that developed in the regular concrete walls.

Introduction

Reinforced concrete (RC) structural walls are widely used in regions of high seismicity as the primary lateral load resisting mechanism because of their documented efficiency to provide lateral strength, stiffness and drift control (Wood, 1991). Low-rise walls, usually defined as walls with a height-to-length ratio less than two, exhibit a behavior highly influenced by shear and find applications in residential buildings, parking structures, industrial buildings, and nuclear power plants, among others.

The reinforcement detailing in RC structural walls required by Chapter 21 of the 2008 ACI Code (318 ACI Committee, 2008) typically consists of main vertical reinforcement at the wall edges, and distributed horizontal and vertical web reinforcement to provide cracking control and avoid a diagonal tension failure. When large inelastic deformations are expected, special

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transverse reinforcement is provided at the wall edges to avoid early concrete crushing and to restrain (or delay) buckling of the longitudinal reinforcement. The ACI Code requires spacing for this reinforcement to not exceed one-third of the minimum member dimension.

The nominal shear strength of structural walls according to the ACI Code seismic provisions is based on the modified truss analogy. An increased "concrete" contribution to shear strength equal to $0.25\sqrt{f_c'}$ (MPa) is attributed to low-rise walls because of the beneficial effect of arch action. In order to prevent web crushing failures, an upper limit of $0.83\sqrt{f_c'}$ (MPa) is imposed on the wall shear stress, based on the recommendations by Cardenas et al. (1973). In addition, Wood (1990) proposed a lower bound of $0.5\sqrt{f_c'}$ (MPa) for the shear strength of low-rise walls, based on test data on low-rise walls subjected to lateral loads.

The ACI Code required seismic detailing for low-rise structural walls generally ensures an acceptable level of performance in terms of shear resistance, but can result in severe reinforcement congestion and construction difficulties. In addition, some experimental studies have revealed inadequate deformation capacity for low-rise walls. For example, Barda et al. (1977) reported drifts at failures less than 1.0% for walls with shear span-to-length ratios ranging from 0.25 to 1.0.

An alternative design for low-rise walls is presented in this paper, which consists of the use of High-Performance Fiber Reinforced Concrete (HPFRC) in lieu of regular concrete combined with a relaxation in wall reinforcement detailing. HPFRC is a relatively new type of fiber reinforced concrete that exhibits a strain hardening behavior under direct tension with a compressive behavior similar to that of well confined concrete. This material has been used in shear-critical elements with very promising results in terms of shear resistance, deformation capacity and damage tolerance (Parra-Montesinos, 2005). The potential for a relaxation in the required wall web and boundary region confinement reinforcement, without a compromise in wall seismic performance, was the focus of this investigation.

Experimental Program

Nine cantilever low-rise walls, constructed with either conventional concrete or an HPFRC material, were tested under displacement reversals in the Structural Engineering Laboratory at the University of Michigan. Each specimen consisted of a wall fixed connected to a reinforced concrete base block anchored to the laboratory strong floor and a reinforced concrete top block used for loading purposes. The load was applied through a hydraulic actuator connected to the mid-depth of the top block at one end and to the laboratory reaction wall at the other end. The basic experimental parameters were the wall shear span-to-length ratio and peak shear stress demand. The walls had a rectangular cross section, 1000 mm long and 100 mm thick. The wall shear span, a, was equal to either 1200 mm or 1500 mm, which translated into a shear span-to-length ratio, a/l_w, of 1.2 and 1.5, respectively. The geometry of the specimens is shown in Fig. 1. The specimens were subjected to quasi-static reversed cyclic displacements, with peak displacement at a given cycle ranging from 0.125% drift to 3.0% drift or up to failure.

The main features of the specimens are summarized in Table 1. Specimens S6 and S7 were designed and tested by Kim and Parra-Montesinos (2003). Specimens S1, S3, S5 and S8 were constructed with regular reinforced concrete and designed following the seismic provisions in Chapter 21 of the 2005 ACI Code. The main longitudinal wall reinforcement was concentrated in the wall edges and selected such that the target shear stress level, v_{target}, (see Table 1) could be attained. Because large inelastic deflections were expected to be imposed on the test walls, special boundary elements were provided in all the RC specimens as required by the ACI Building Code. This provision led to single hoops at the tight spacing of 25 mm at the wall edges. As an example, the reinforcement layout in RC Specimen S3 is shown in Fig. 2a (elevation) and 2c (cross section).

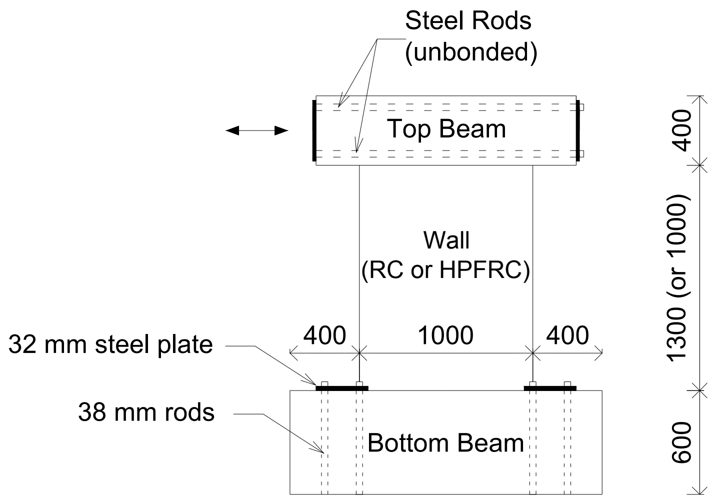


Figure 1. Geometry of test specimens (Dimensions: mm)

Table 1. Main features of test specimens

Specimen	a/l_w	v_{target} (in $\sqrt{f_c'}$ [MPa])	Fiber type	V_f (%)	Fiber length/ diameter (mm)	Fiber strength (MPa)	Compressive strength (MPa)
S1	1.2	0.40-0.50	-	-	-	-	45.8
S2	1.2	0.40-0.50	Regular-strength hooked steel fibers	2.0	30/0.55	1120	48.2
S3	1.2	0.65-0.75	-	-	-	-	45.8
S4	1.2	0.65-0.75	High-strength hooked steel fibers	1.5	30/0.38	2310	39.5
S5	1.5	0.55-0.65	-	-	-	-	46.5
S6	1.5	0.55-0.65	Regular-strength hooked steel fibers	2.0	30/0.55	1120	46.7
S7	1.5	0.55-0.65	Spectra fibers	1.5	38/0.038	2585	43.7
S8	1.5	0.70-0.80	-	-	-	-	42.4
S9	1.5	0.70-0.80	High-strength hooked steel fibers	1.5	30/0.38	2310	37.9

The configuration of the reinforcement for the HPFRC specimens (Specimens S2, S4, S6, S7 and S9) followed the same basic layout as their companion reinforced concrete walls. The ACI Code provisions for the web distributed reinforcement and the confining reinforcement in the boundary elements, however, were not followed, as a major goal in this study was the evaluation of a possible relaxation in the reinforcement configuration. The confining reinforcement in the boundary region of Specimens S2, S6 and S7 was completely eliminated.

The boundary regions in Specimens S4 and S9, designed for a higher shear stress, were confined with hoops at a 100 mm spacing, which corresponded only to one-fourth of the confinement reinforcement provided in the companion RC Specimens S3 and S8, respectively. Fig. 2b presents the reinforcing detailing for Specimen S4.

Table 2. Reinforcing details of wall in the test specimens

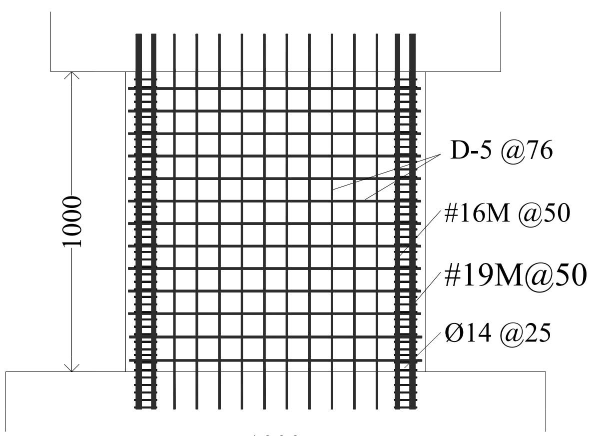
Specimen	Vertical wall boundary reinforcement	Wall web region		Confinement reinforcement (boundary region)	Dowel reinforcement (bonded/unbonded length)
		Horizontal	Vertical		
S1	4 #13M	#6M @90	#6M @90	Ø4 @25	-
S2	4 #13M	#6M @200	#6M @200	-	-
S3	2 #19M+2 #16M	D-5 @75	D-5 @75	Ø4 @25	-
S4	4 #16M	D-5 @100	D-5 @100	Ø4 @100	4 #13M (200mm/150mm)
S5	2 #19M+2 #16M	#6M @75	#6M @90	Ø4 @25	-
S6	3 #22M	#6M @255	#6M @150	-	2 #16M (200mm/0mm)
S7	3 #22M	#6M @255	#6M @150	-	2 #16M (200mm/0mm)
S8	2 #22M+2 #19M	D-4 @75	D-4 @75	Ø4 @25	-
S9	4 #19M	D-4 @100	D-4 @100	Ø4 @100	4 #13M (200mm/250mm)

Because of the construction sequence, which was meant to simulate that in real construction, a cold joint existed at the base block-wall interface. This cold joint is more critical in HPFRC walls compared to RC walls because no fibers bridge the base-wall interface, which results in a significantly weaker joint section than the adjacent wall sections above the foundation. Furthermore, because of the excellent bond between the reinforcement and HPFRC materials (Parra-Montesinos et al.; 2005; Chao et al., 2007), the longitudinal reinforcement would be expected to yield only in the close neighborhood of the cold joint. This concentration of yielding at the base of the wall could ultimately result into a sliding shear failure or even fracture of the reinforcement with detrimental effects on the displacement capacity of the wall. While the cold joint was left intact in HPFRC Specimen S2, dowel bars were added in the other HPFRC walls to strengthen the wall-foundation interface and prevent concentration of inelastic rotations at the cold joint. Further, in order to prevent the occurrence of a predominant flexural crack caused by the termination of the dowel bars within the plastic hinge region, these bars were extended, debonded, beyond the wall expected plastic hinge region in Specimens S4 and S9.

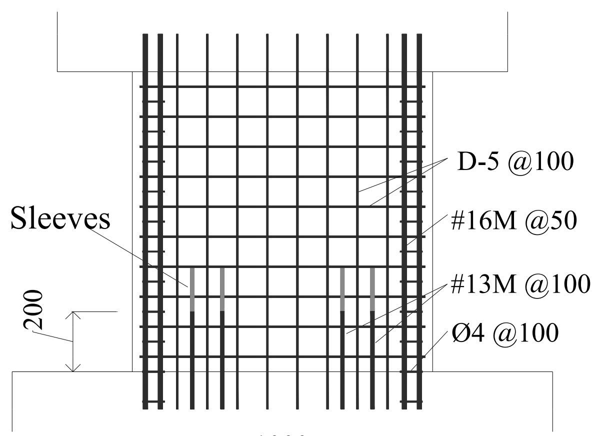
Material Properties

All the cement-based materials used for the wall of the specimens were prepared in the Structural Engineering Laboratory at the University of Michigan. For the RC specimens, the proportions of the regular concrete mix by weight (cement: sand: coarse aggregate: water) were 1:1.55:1.45:0.48. Type III cement (high early strength) and coarse aggregate with 13 mm maximum aggregate size were used. The fiber reinforced mortar mixture used in Specimens S2 and S6 had proportions by weight 1:2:0.48:0.20 for Type III cement, # 16 Silica Sand (Flint Silica #16 manufactured by U.S. Silica Company), water, and Class C fly ash. A 2% volume fraction of regular strength (1120 MPa) hooked steel fibers, 30 mm long and 0.55 mm diameter (Dramix ZP305), were used in this mixture. For the wall of Specimen S7, the mixture proportions by weight were 1:1:0.5:0.15 (Type III cement: #16 silica sand: water: Class C fly ash). Ultra high molecular weight polyethylene (Spectra) fibers with a diameter of 0.038 mm and a length of 38 mm were used in this mixture in a 1.5% volume fraction.

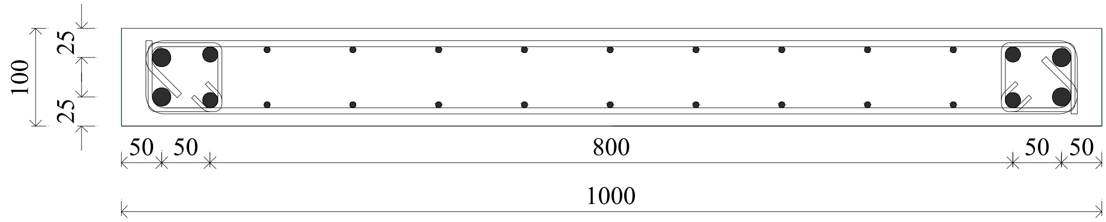
The mixture used for Specimens S4 and S9 was one of a series of self-consolidating HPFRC mixtures developed at the University of Michigan for seismic applications (Liao et al., 2007). This mixture was chosen to ease the casting of the wall specimens, given the small wall thickness (100 mm). The proportions by weight for this material were 1:2.2:1.2:0.8:0.89:0.005:0.038 for Type III cement, #16 Silica Sand, coarse aggregate (13 mm maximum size), water, Class C fly ash, superplasticizer, and a viscosity modifying admixture (VMA). For this mixture, high-strength (2310 MPa) hooked steel fibers, 30 mm long and 0.38 mm diameter (Dramix RC-80/30-BP), were used in a 1.5% volume fraction. Detailed information about the mechanical properties of the fiber reinforced concretes used in this investigation can be found elsewhere (Athanasopoulou and Parra-Montesinos, 2010).



a) Specimen S3



b) Specimen S4



c) Cross section details (Specimen S3)

Figure 2. Wall reinforcing details of selected specimens

In all the specimens, # 13M, 16M, 19M and 22M bars were deformed, made out of Grade 420 (metric) steel. The #6M bars used for the web reinforcement in Specimens S1, S2, S5, S6 and S7 were smooth bars. These bars exhibited a nearly elastic-plastic behavior with yield stress ranging from 630 MPa to 700 MPa. A deformed D-5 wire (area = 32.3 mm^2) was used as web reinforcement for Specimens S3 and S4, whereas a crimped D-4 wire (area = 25.8 mm^2) was used in Specimens S8 and S9. Both wires had a yield stress of approximately 630 MPa. A 4 mm diameter wire was used for the confinement reinforcement in the boundary regions, with yield and ultimate strength of 200 MPa and 330 MPa, respectively.

Experimental Results

Hysteretic response and cracking pattern

RC Specimens

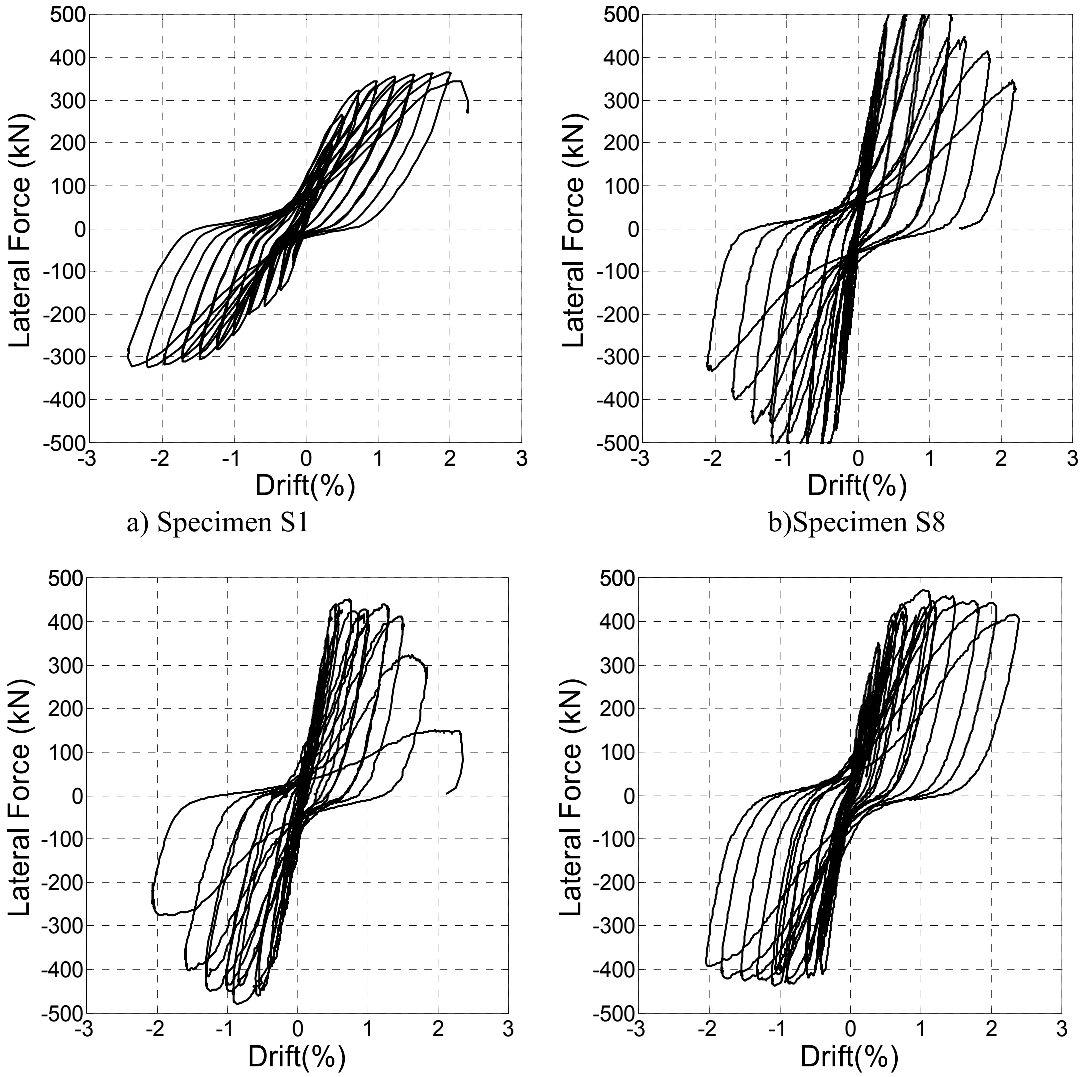
The main experimental results are summarized in Table 3. RC Specimens S1, S5 and S8 exhibited a similar behavior in terms of cracking pattern. At the end of the tests, major damage in these specimens concentrated in the corners at the wall base and diagonal crack widths in the wall web were approximately 2.5 mm. For drifts larger than 1.0%, sliding shear deformations at the wall base governed their behavior, with inelastic rotations concentrated at the wall base. Fig. 3a and Fig. 3b show the lateral load versus drift response for Specimens S1 and S8, respectively. Drift was defined as the lateral displacement at mid-depth of the top block in relation to the top of the base block divided by the shear span. The hysteretic behavior of Specimen S5 was similar to that of Specimen S1.

Table 3. Experimental Results

Specimen	F_{max} (kN)	$\frac{v_{max}}{\sqrt{f_c'}}$ (MPa)	$\frac{v_{max} - v_{s(ACI)}}{\sqrt{f_c'}}$ (MPa)	Drift at max load (%)	Drift capacity* (%)
S1	345	0.49	0.05	2.1	2.3**
S2	265	0.38	0.19	1.6	3.0
S3	465	0.67	0.15	0.75	1.5
S4	350	0.54	0.12	0.85	1.5
S5	350	0.56	0.12	1.5	2.1
S6	395	0.51	0.43	1.6	2.0
S7	365	0.54	0.46	1.8	2.2
S8	530	0.78	0.35	0.9	1.5
S9	455	0.71	0.37	1.1	2.2

*Drift capacity is defined as the maximum drift attained prior to a strength loss greater than 20%.
**Test was stopped due to excessive shear sliding at the wall base.

Specimen S1 sustained a peak average shear stress of $0.49\sqrt{f_c'}$ (MPa) at approximately 2% drift and the test was terminated at a drift of 2.4%, even though there was no clear indication of failure, because sliding at the wall base accounted for more than half of the applied displacement. However, shear-related damage near the end of the test was not significant. Specimen S5, on the other hand, sustained cycles of up to 2.1% drift with a peak average shear stress of $0.56\sqrt{f_c'}$ (MPa). In this specimen, concrete crushing in the compression edges at the wall base was evident at a drift of 1.9%, with splitting along the vertical reinforcement. Because of the increased shear stress demand, Specimen S8 was subjected to a peak shear stress of $0.78\sqrt{f_c'}$ (MPa) at a drift of 0.9%, with substantial loss of strength for larger drifts. At 1.8% drift, severe concrete spalling in the boundary elements was evident and the test was terminated at 2.1% drift when the wall had lost more than 40% of its strength. Fig. 4a shows the state of damage in Specimen S8 at the end of the test. Similar cracking pattern and damage localization was observed in Specimens S1 and S5.



a) Specimen S1

b)Specimen S8

c) Specimen S3

d) Specimen S9

Figure 3. Lateral load versus drift response

As opposed to the other RC specimens, Specimen S3 experienced significant shear-related damage at drifts larger than 1.3% (peak shear stress of $0.67\sqrt{f_c'}$ (MPa)). Damage was characterized by web concrete crushing in the lower part of the wall, followed by a rapid strength loss. The wall had lost about 30% of its strength at a drift of 1.7% (Fig. 3c) and the measured shear distortions in the lower part of the wall exceeded 0.01 rad. Fig. 4b shows the damage in Specimen S3 near the end of the test.

HPFRC Specimens

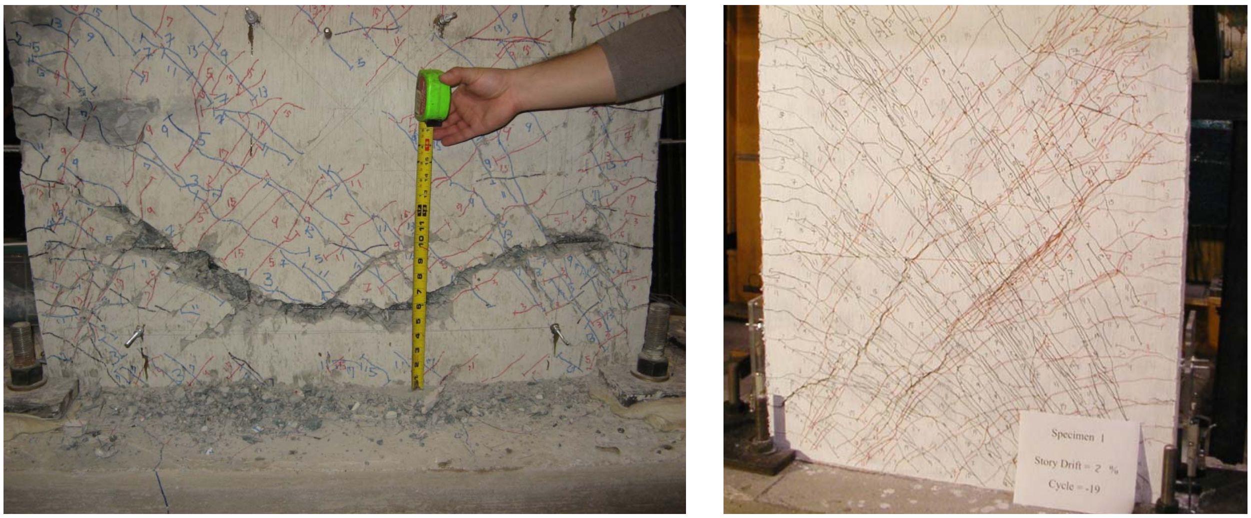
No dowel reinforcement was used in HPFRC Specimen S2 since it was argued that, given the low wall aspect ratio, shear deformations in the wall web would play a major role in wall behavior and thus, concentrated flexural deformations at the wall base would likely not be large enough to result in a sliding shear failure. However, significant sliding shear deformations at the cold joint characterized the wall behavior for drifts larger than 0.8%. Shear-related damage in the wall was minor until the end of the test at 3.0% drift, and the need for dowel reinforcement to strengthen the wall-foundation interface in HPFRC walls was evident. Shear sliding was predominant in the positive loading direction, resulting in a lower lateral load capacity compared to the negative loading direction. Despite the complete elimination of confinement reinforcement in the boundary regions, no appreciable damage was observed in the extreme compression fibers at the wall base for drifts of up to 1.5%.

a) Specimen S8 at 2.1% drift b) Specimen S3 at 1.9% drift

Figure 4. Damage in RC specimens

The addition of dowel bars in HPFRC Specimens S4 and S9 resulted in damage concentration in a horizontal crack at the section where the bonded region of the dowel bars ended. These two specimens sustained a peak shear stress of $0.54\sqrt{f_c'}$ (MPa) and $0.71\sqrt{f_c'}$ (MPa), respectively. Fig. 3d shows the load versus drift response for Specimen S9. Specimen S4 exhibited some minor concrete spalling at the wall web at 1.7% drift and failed at approximately 2% drift due to substantial rotations concentrated at the horizontal crack where the dowels terminated. On the other hand, damage concentration was evident in Specimen S9 only for drifts larger than 1.9%. The specimen could sustain more than 90% of the peak load at a drift of 2.4%. No indication of instability was observed in the wall boundary regions despite the significant relaxation in the confinement reinforcement (Fig. 5a). Compared to RC Specimens S3 and S8, the use of HPFRC led to a much denser array of cracks with smaller widths, improving the damage tolerance of the walls.

Damage in Specimens S6 and S7, tested by Kim and Parra-Montesinos (2003), was characterized by a heavily dense array of diagonal cracks for drifts of up to 1.5%. At larger drifts, damage localized in a few diagonal cracks, which ultimately led to a diagonal tension failure (Fig. 5b) at approximately 2.3% drift and 2% average shear distortion. It should be mentioned that these two specimens were designed to fail in diagonal tension with limited flexural yielding and had only one layer of reinforcement. Fiber type (hook steel fibers versus Spectra fibers) did not seem to affect the shear strength of these two walls (peak shear stress of $0.51\sqrt{f_c'}$ (MPa) and $0.54\sqrt{f_c'}$ (MPa) for Specimens S6 and S7, respectively). A more dense array of cracks, however, formed in the specimen reinforced with Spectra fibers.



a) Specimen S9 at 2.1% drift b) Specimen S8 at 2.3% drift

Figure 5. Damage in HPFRC specimens with dowel bars

Conclusions

From the reversed cyclic displacement tests of low-rise RC and HPFRC walls described in this paper, the following conclusions can be drawn:

1. The RC walls, designed according to the seismic provisions of the ACI Code and tested under moderate ($0.49\ to\ 0.56\sqrt{f_c'}$, MPa) and high ($0.6\ to\ 0.78\sqrt{f_c'}$, MPa) shear stress reversals, exhibited stable hysteretic response for drifts up to approximately 2.4% and 1.3%, respectively.

2. HPFRC low-rise walls subjected to moderate shear stress reversals exhibited a stable hysteretic response despite the complete elimination of confinement reinforcement in the wall boundary regions. The HPFRC walls also had relaxed web reinforcement ratio compared to the RC companion specimens which resulted, overall, in a simplified reinforcement detailing.

3. The use of self-consolidating HPFRC material with a 1.5% volume fraction of high-strength hooked steel fibers allowed a significant relaxation in the confinement reinforcement in walls tested under high shear stress reversals (1/4 of that in companion RC specimens) without compromising wall seismic behavior.

4. All HPFRC walls exhibited superior damage tolerance, characterized by a much denser array of cracks, compared to the RC companion specimens.

5. The wall-foundation interface is particularly critical in HPFRC low-rise walls because no fibers bridge that section, making it susceptible to damage localization and potential sliding shear failure. Thus, dowel reinforcement should be used to strengthen the cold joint and force inelastic deformations to occur within the HPFRC wall.

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