PERFORMANCE OF SHFRCC-RC CONCRETE MEMBERS UNDER CYCLIC DISPLACEMENT REVERSALS

Antroula GEORGIOU, Stavroula PANTAZOPOULOU

ABSTRACT

The new generation of ternary-cement based, fiber-reinforced materials demonstrate impressive strain hardening response in tension and extensive ductility marked by multiple crack formation and size-insensitivity (strain hardening fiber reinforced cementitious composites or SHFRCC). This advent in cementitious materials technology enables the prospect of development of a new generation of structural systems that benefit from the inherent ductility of concrete in tension in order to reduce the amounts of transverse reinforcement (stirrups) that would be normally required in design for confinement on the compression zone of concrete, shear strength, and tension-force development capacity of the main reinforcement. In this study a number of tests are conducted to explore the behavior of SHFRCC materials under cyclic loads, simulating seismic effects. The experimental responses of two half-scale interior beam column connections subjected to reversed cyclic loading are compared; one of the connections was constructed with a cementitious matrix without fibers, and was detailed according with the Eurocode provisions for ductility class M (moderate, \( \mu = 3.5 \)). The other connection was constructed with a SHFRCC mix; (2\% by volume of PVA fibers was used to reinforce the matrix and the minimum amount of shear reinforcement allowed by Eurocode 2 for non-seismic detailing was used in the specimens). No axial load was applied on the specimens during loading. The SHFRCC specimen with minimum detailing showed improved performance and enormous ductility suggesting new possibilities to the seismic design of structures.

Keywords: Fibers; Ductility; Strain Hardening

1. INTRODUCTION

Earthquake ground motion imposes reversed cyclic loading on structures. The analysis and design of structures in seismically active areas need to take these effects under consideration. The design philosophy of Codes for concrete members reduces seismic loads by a response coefficient (behavior factor) \( q \) in order to create more economical and more resilient structures. This decrease in the load carrying capacity of the structures affects the requirements for ductility \( \mu \) and energy dissipation of members. Especially in Frame Structures, where Capacity Design concepts are used, the “strong column-weak beam” requirement forces the creation of plastic hinge regions in the beam-ends and at the base of ground-floor columns. Therefore beam and column hinging regions must be detailed specifically in order to maintain their shear capacity as seismic loading imposes yielding in the longitudinal reinforcement (BS EN 1998:3, 2005) both in the tensile and the compressive region.

Ductility of beam members is influenced by the geometry details, the percentage of the reinforcement, the composite material’s stress-strain properties and the shear span ratio. In the case of conventional concrete, the concrete’s tensile strength was neglected in calculating moment capacity of members due to the brittle nature of concrete, while in the region of the beams under compression spalling of the concrete cover reduces the capacity of the zone under compression and usually leads to brittle failures. Additionally for the shear design of the beams that undergo reversed cyclic loading, causing diagonal shear cracking (X-shape cracks), the concrete’s contribution to shear is neglected and stirrups are

1Researcher/Instructor, University of Cyprus, Nicosia, Cyprus, ageorg44@ucy.ac.cy
2Professor, Dept. of Civil Engineering, York University, Toronto, ONT M3J1P3, Canada, pantazo@yorku.ca
required to transfer all shear loading. This leads to excessive amounts of detailing, denser placement of shear reinforcement that complicates construction and casting.

1.1 State of the Art

In light of these practical limitations the use of a new type of composite material reinforced with short discontinuous fibers that exhibits strain hardening properties (SHFRCC) under tension and improved performance in compression could be an alternative solution to the stringent detailing requirements. The improved performance in tension benefits the composite’s behavior in shear as seen in the short span beams subjected to cyclic loading in previous studies (Kanta, Watanabe, & Li, 1998). Fischer & Li (2007) who studied the combined behavior of steel reinforcement in SHFRCC matrix have shown that strain hardening behavior of both components past yielding prevents localization of deformations leading to uniformly distributed yielding of the reinforcement over the length of the specimens they tested, increasing energy dissipation. The flexural behavior of beams with SHFRCC and FRP bars were experimentally investigated by Yuan and Pan (F Yuan & Pan, 2013), that concluded that wide cracks opening in normal concrete, diffuse to wider areas with many micro cracks in SHFRCC, leading to higher deformability and compatibility of deformations of the FRP reinforcement with the surrounding matrix, reducing bond splitting. Additionally the shear rupture of the bars that appears usually at the crack locations due to stress concentrations was avoided and the SHFRCC showed no delamination. Limited experimental research concerning the reversed cyclic loading of steel reinforced SHFRCC members has been published todate ([Lequesne, Parra-Montesinos, & Wight, 2016], (Gregor Fischer & Li, 2003), (Fang Yuan, Pan, Dong, & Leung, 2014), (Parra-Montesinos & Chompreda, 2006]) to support generalized conclusions; however, it is evident from the available experimental results that – owing to its unique properties, SHFRCC can work effectively together with longitudinal steel reinforcement and improve ductility of reinforced concrete structural members without the requirement of significant additional transverse reinforcement. The present paper investigates the behavior of SHFRCC-RC Structural members under cyclic loads, simulating seismic effects. To this end, the experimental responses of two half-scale interior beam column connections subjected to reversed cyclic loading are compared; one of the connections was constructed with a cementitious matrix without fibers, and was detailed according with the Eurocode provisions for ductility class M (moderate, $\mu=3.5$). The other connection was constructed with a SHFRCC mix; (2% by volume of PVA fibers was used to reinforce the matrix and the minimum code-specified amount of transverse reinforcement for non-seismic detailing was used in the specimens). Specimens were tested under flexure shear with no simultaneous axial load. The SHFRCC specimen with minimum reinforcement showed improved performance and enormous ductility over the conventional concrete seismically detailed member, suggesting new possibilities to the seismic design of structures.

2. EXPERIMENTAL PROCEDURE

2.1 Members under cyclic loading - test setup

Specimen geometry is depicted in Figure 1. Each specimen consisted of two cantilever flexural members with a shear span ratio of 3.5 connected with a stiff node which is used for loading by the actuator in both directions. The beams were supported on rollers both on their bottom and top faces. The entire system behaves like a beam under three point loading where each shear span is subjected to shear equal to half the machine load, and a linearly moment from zero at the supports to $M=(P/2)L_c$ at the connection with the rigid node, where $L_c$ is the shear span length. One of the two connecting flexural members of each specimen was designed as per the research objective, whereas the other connecting segment was heavily reinforced (both in the longitudinal and transverse direction) in order to preclude any form of failure – in the remainder of the article this segment is referred to as “support beam”. 2
Specimens were constructed of SCC with a high content of fly ash (FA) without coarse aggregate; one was made with SHFRCC mix with PVA fibers of 2% by volume, while in the other the cementitious matrix was the same as that of the SHFRCC but without the fibers (Control). The Control Member was reinforced as per Eurocode 8 provisions for a DCM (i.e. q=3.5) (EN1998-1-2004, 2004). The SHFRCC beam had the same longitudinal reinforcement but the minimum required transverse reinforcement for non-seismic construction was used - required to prevent buckling of the longitudinal bars in compression. The purpose of the tests was to investigate the beneficial action of the fibers on the shear/flexural capacities of members under reversed cyclic loading.

![Figure 1. Stirrup detailing for (a) Member A: Control and (b) Member B: SHFRCC and (c) Test setup and measuring equipment](image)

2.2 Member Detailing

Member detailing follows the design for Ductility Class Medium according to Eurocode 8 (EN 1998-1 (EN1998-1-2004, 2004)). Given the dimensions and strength of the available testing frame it was decided to make the specimens at a scale of 1:2. Thus, all dimensions and reinforcement detailing of the test specimens was calculated for the full scale and then reduced to half. Eight bars of 20 mm diameter were chosen to be placed on the full scale specimen based on the detailing provision of the EC8. Therefore the 1:2 scaled specimens have the following characteristics considering that the material parameters such as the compressive strength of the cementitious material and the yielding strength of the reinforcement remain constant:

**1:2 Scaled Member Parameters:** $f_c=50$ MPa, Axial Load=$0$ kN, cross section $200x200$ mm, $L=1.4$ m, $L_f=700$ mm, $c=20$ mm, reinforced with 8 bars (10 mm diameter) distributed evenly on the perimeter of the column (three per side). Yield moment of the Control Member was calculated at
22.27 kNm whereas the flexural strength was 26.47 kNm. Thus, the shear force demand was $V_{d,e}=1.4\cdot26.47/0.7=41.6$ kN. Based on this value, 6 mm-diameter stirrups having 300 MPa yield stress spaced at 90mm were used. Stirrup spacing was reduced to 80 mm in the critical region, subject to the detailing specification (min{8d_{sl}=80 mm, b_{fj}=80 mm, 175 mm}). Therefore Φ6/80 stirrups with 3 legs in each direction were used. For the member with the SHFRCC matrix mass reinforced with 2% per volume of fibers the same geometry of the cross-section was applied as well as the same longitudinal reinforcement (8Φ10). In order to obtain the beneficial results of the use of fibers on the shear strength and ductility of the member the stirrups placed were not based on the shear demand. Only the provision for the DCL class of column members was followed in order to assure that no buckling of the reinforcement would be observed. Maximum stirrup spacing in this case is controlled by the requirement: min{120d_{sl}=1200, 0.6h_e=120, 0.6b_f, 240 mm}; thus Φ6/120 perimeter stirrups were used. Detailing is depicted in Figure 1.

Using stress-strain relationships for the materials as measured in the Laboratory in pertinent specimens sectional analysis was conducted: owing to the beneficial action of the fibers flexural strength of the SHFRCC specimen was calculated at 35 kNm. Therefore the supporting member was consistently overdesigned to be able to develop the strength of the test specimen (Eight bars of 16mm diameter for the longitudinal reinforcement and 8mm bars for stirrups spaced at 80 mm with three legged pattern in each direction were used as depicted in Figure 1). All the longitudinal bars were very well anchored within the rigid zone. The central rigid node connecting the test and the support segments of the specimen was overdesigned in order to be able to support moment transfer and to anchor the strength of the longitudinal reinforcement. A “cage” type reinforcement was chosen as depicted in Figure 1 with closed stirrups encasing the node. The reinforcement that was parallel to the longitudinal axis of the beams comprised 16 mm bars at 70 mm, while the vertical reinforcement that was parallel to the loading direction was 16mm diameter bar also spaced at 70mm.

### 2.3 Test setup and loading

Specimens were tested at the Concrete Technology and Structures Laboratory of the University of Cyprus. The test was conducted on a 250 kN MTS hydraulic actuator, digitally controlled and attached on a self-reacting steel frame (Figure 2 (a)). The specimen assembly was supported on rollers at the edges in order to allow rotation at the supports while the central rigid node was attached to the load actuator. Thus the node was bound between an upper and a lower steel plate; threaded rods connected the two plates to the actuator. The setup did not allow horizontal translation of the rigid node, while it allows the beams to expand away from the node without the creation of spurious axial loads. The supports were specifically designed for this purpose (Figure 2 (a)). The rollers were placed both on the top and bottom sides of the beam specimens and were attached to the loading frame in order to prevent upward movement during the reversed loading. Figure 2 (a) shows a photo with the details of the actual placement. The test specimen had a total length of 2.50 m, while the cross section of the beams was 200x200 mm. The shear span had a length of 700 mm, giving a shear span ratio a/h=3.5.

Instrumentation comprising 13 LVDTs and DTs with various travel-lengths were used as shown in Figure 1. LVDTs 1 to 5 had a travel distance of 100 mm; LVDTs 1 to 3 were placed under the bottom steel plate that was holding the middle rigid node and transferring loads from the actuator to the specimen, LVDTs 4 and 5 were placed on the bottom side of the two beams at a distance of 30 mm from the rigid node. DTs 1 to 8 had a travel distance of 40mm and were placed as follows: DTs 5 and 6 were placed so as to measure the vertical deflection of the neutral axis at the ends of the shear spans of the two beams (middle of height) and DTs 1 to 4 and DTs 7 and 8 were placed on the top and bottom sides (three on each side) of the Control and SHFRCC members close to the rigid node, at the zone where the plastic hinges were expected to form. All the measuring equipment was mounted on a system of metallic beams that were not in contact with the reacting frame.

A cyclic displacement history with full reversal of deflection was applied on the beam. Three cycles were imposed for each displacement level up to a displacement of 18 mm, two for the displacement of
22.5 mm and one for the higher of the displacement levels. The amplitudes of displacement and number of cycles are listed in Table 1. Twenty cycles were employed for the reversed loading as shown in Fig 2(b), while loading was stopped at each peak in order to record the new cracks that were formed. The test was performed under displacement control at a rate of 3 mm/min and was controlled by the actuator’s movement. The chosen displacement level, that was used to determine the amplitude, was the deflection of the beams at yielding of the flexural reinforcement $\Delta_y$.

Table 1. Number of cycles and amplitude

<table>
<thead>
<tr>
<th>No. of cycles</th>
<th>3</th>
<th>3</th>
<th>3</th>
<th>3</th>
<th>3</th>
<th>2</th>
<th>1</th>
<th>1</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amplitude $\Delta$ (mm)</td>
<td>2.25</td>
<td>6.00</td>
<td>9</td>
<td>13.5</td>
<td>18</td>
<td>22.5</td>
<td>27</td>
<td>31.5</td>
<td>36</td>
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</tbody>
</table>

Figure 2. (a) Placement of the specimen and (b) Cyclic load pattern used in testing

The expected yield curvature for the Control member was used in order to determine the target displacement levels shown in Table 1. Based on the Moment-curvature diagram of Figure 3 that was an estimation of the behavior of the control member the yield curvature was 0.000037 and therefore $\theta = \phi \cdot L / 3 = 0.000037 \cdot 700 / 3 = 0.00863$ and yield deflection $\delta_y = 0.00863 \cdot 700 = 6.04$ mm.

Figure 3. $M$-$\phi$ diagram of the 1:2 scaled members (a) Member A: Control and (b) Member B: SHFRCC

2.4 Properties of Materials

The cementitious matrix comprised 2% PVA fibers (SHFRCC mix) and mortar made of Portland Composite Cement EN 197-1 CEM II / A-M (L-S) 42.5 R produced using pure calcite limestone and is more impermeable and dense as compared to OPC, with a higher degree of workability and reduced plastic shrinkage. Silica sand used had a maximum grain size of 300 $\mu$m. The characteristic of this product is spherical crystal quartz particles showing a very narrow grain size distribution, a low content of side minerals and a high $\text{SiO}_2$ content as well as a very special light color. The percentage of silica oxide is in the order of 98.6%. The Mohs Hardness of this product is 7, while raw density is
2.65 gr/cm³ and bulk density is 1.35 gr/cm³. A significant portion of the binding agent was Type F Fly ash with pozzolanic and physical properties that enhance the performance of concrete (complies fully with ASTM C618 (2008) and EN 206 (2013)). This fly ash contains extremely fine (0.45 μm) latently reactive silicon dioxide. The concrete becomes extremely cohesive and the pumping properties are substantially improved. In the set concrete the latently reactive silicon dioxide forms a chemical bond with free lime. The dry bulk density is 1000 kg/m³. Class F FA has been used in other studies as cement replacement at fractions ranging from 20-30% of the mass of cementitious material but in this research the amount of FA used was 120% by weight of the amount of cement for the cases with the PVA fibers (that is, one part of hydraulic material in the final mixture comprised 45% cement and 55% fly-ash).

In the present investigation 12 mm long, 39 μm diameter PVA fibers were used. The nominal tensile strength was 1600 MPa, Young’s Modulus was 40 GPa, the fibers’ strain capacity was 6.5% and the density was 1300 kg/m³. This synthetic fiber called Kuralon K-II is based on polyvinyl alcohol (PVOH) resin. Detailed proportions of the two mixes are listed in Table 2.

Table 2. Mix proportions per weight

<table>
<thead>
<tr>
<th>Mix</th>
<th>C</th>
<th>FA</th>
<th>S</th>
<th>W</th>
<th>SP</th>
<th>ρf (%V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>HVFA</td>
<td>1</td>
<td>1.2</td>
<td>0.80</td>
<td>0.60</td>
<td>0.012</td>
<td>2.0</td>
</tr>
<tr>
<td>SHFRCC</td>
<td>1</td>
<td>1.2</td>
<td>0.80</td>
<td>0.58</td>
<td>0.012</td>
<td>-</td>
</tr>
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</table>

Compressive displacement control tests on cylinders of 100x200mm were performed on both mixes used for the construction of the members, while uniaxial tensile tests were performed on samples of the SHFRCC mix (Georgiou & Pantazopoulou, 2016). Additionally the steel bars used as reinforcement were tested under uniaxial tension. The longitudinal bars tensile yield strength was 500 MPa whereas stirrups were of mild steel (f_y=300 MPa) smooth bars (Figure 1).

3. EXPERIMENTAL RESULTS

3.2 Material Properties under tension and compression

The tensile tests were carried out on a floor standing testing machine with a load cell capacity of 300 kN (Lloyd LR300K), using displacement control of the longitudinal translation of the machine. The displacement rate was 0.0025 mm/s. Deformations were measured using LVDTs mounted on the two wider opposite sides of the coupons over a 100 mm gauge length in the narrowest region of the specimen where the cross section was 25x50 mm. The testing setup had spherical hinges on both sides to ensure self-correction of possible misalignment and that no spurious stresses other than axial loading would be developed within the measuring range. The gripping equipment was custom made (Figure 4). Direct tensile tests were carried out for three specimens at the age of 100 days. Uniaxial tensile stress-strain curves were extracted through this test. Stress was calculated as the Machine Load divided by the critical cross section area of 25x50 mm. Strains were obtained as the averaged deformation of the two opposite LVDTs divided by the measuring length of 100 mm.

Uniaxial compression tests were performed well into the post-peak range under displacement control using a closed-loop, servo hydraulic controlled testing machine at a loading rate of 1.50 μm/s. The specimens were cylinders, 100 mm in diameter and 200 mm in height. Axial deformations were measured using two linear variable differential transducers (LVDTs) mounted on opposite sides of two rings that were attached on the specimen over a gauge length of 140 mm. An additional LVDT measured the deformation over the entire length of the specimen, through the displacement of the platens. The displacement control of the machine was based on this LVDT. Circumferential elongation was measured using a circumferential extensometer (chain), placed at the middle of the height of the coupon. Testing setup and equipment are shown in Figure 4. Compressive axial stress - axial strain -
lateral strain curves extracted from the experimental measurements are reported in the diagrams of Figure 4. Stress is computed by dividing the Load by the area of the cross section while axial deformations are obtained as the average of the two opposite LVDTs divided by the 140 mm distance between the gripping equipment. Lateral strain is extracted by the deformation of the extensometer divided by the perimeter of the cylinder. Past work (Georgiou & Pantazopoulou, 2017) has shown that the extent of damage in concrete may be quantified by the lateral expansion of the cross section supporting the axial load; indeed, softening of the compressive strength is directly related to cracking parallel to the direction of loading and lateral expansion. As a corollary to that, the effect of confinement is reflected in the degree of restraint provided against lateral expansion.

![Graph](image1)

**Figure 4.** (a) tensile test and (b) compression test

The most important behavioral parameter of SHFRCC is the tensile strength and strain capacity. During the formation of multiple cracks, the composite matrix-fiber system appears to strain-harden. Due to stress, fibers bridging the cracks elongate but they also pull-out partially from the matrix at the crack locations. In the dog-bone specimens the formation of cracks saturated the full measuring length (narrow part of the specimen). Beyond a limiting strain, the value of which appears to depend on the competition of interfacial bond and tensile strength and stiffness of the fibers, new cracks cease to form and crack localization occurs. Plain mix specimens (HVFA) were fractured incipiently during their extraction from the molds (they were very brittle) and therefore it was not possible to test them in direct tension. Figure 4 (a) shows the average tensile stress-strain curves for the three SHCC specimens.

Figure 4 compares the uniaxial response of the plain and FRC-reinforced matrix. Specimens with no fibers cracked and collapsed suddenly. Cracks were vertical, parallel to the load and extended far inside the core of the specimen. After sudden collapse the specimens seemed unable to carry any remaining loads. In the case of the fiber reinforced composites multiple cracking is displayed all around the specimens. Cracking was visible close to 70% of the maximum load, with formation of multiple parallel cracks that increased in number up to the attainment of the peak load. Up till the end of the test no spalling of the specimen was observed. After peak load degradation was slower than in
the case of the HVFA and the specimens after the test seemed solid enough and able to withhold a remaining load. A major crack was formed in all cases of SHFRCCs at the end of the test that had a steep slope that extended from the bottom of the specimen to the side surface. Through these major cracks, the fibers seemed stretched, pulled or ruptured.

Figure 4 (b) plots the axial stress-axial strain (right) and axial stress-lateral strain (left) obtained from the compression tests of the SHFRCC materials and the results for the same matrix without fibers (mix HVFA). The compressive stress of the material without the use of fibers reached an ultimate stress value of 50 MPa (average of three tested specimens), but with the use of fibers, this stress value was slightly decreased. Both strength levels attained were deemed sufficiently high for the construction needs envisioned, which would be focused on the benefits that may be attained through strain-resilience in the tensile response. An increase by 0.5% in the axial deformation of SHFRCC at peak load was reported, a value that is comparable to those obtained for the plain mixes; however, a stable descending branch marked the improved performance of the SHFRCC mixes. Note that lateral expansion is responsible for the steepness in the post peak branch in the compressive stress-strain response of concrete (Zanganeh & Pantazopoulou, 2001). However, in the presence of fibers the behavior was improved and the post-peak decay was much milder as fibers were mobilized in the lateral direction bridging the cracks, transferring load and limiting lateral expansion of the cylinder under compression, as evidenced by the restricted growth of post-peak lateral strain as compared to the plain concrete specimens which undergo excessive, uncontrolled lateral expansion (Figure 4 (b)). The stress-strain curves suggest that past the peak load, concrete with fibers behaves as if passively confined. The intensity of the effective confinement owing to the role of fibers is directly related to the materials’ characteristic stress-strain behavior under tension.

The Modulus of Elasticity of the composites (determined from the compression stress-strain diagram at 45% of peak load) was low, in the range of 20 GPa due to the elimination of aggregates in the mix design. Poisson’s ratio, $\nu$, differed between the SHFRCC ($\approx 0.22$) and the plain HVFA matrix ($\approx 0.33$). In the absence of coarse aggregates the class of materials examined is generally more compliant than normal concrete, attaining their strength, $f_c$, at higher axial compression strain values ($\varepsilon_{co} \approx 0.0032$ for PM). Note that for concrete class C50 the code-prescribed values of strain at peak stress is 2.5‰; the ultimate strain in the post-peak region is taken as 3.4‰ (based on Table 5.1-8 of the Model Code (FIB Model Code, 2010)) while strain at peak stress increases with compressive strength ([Popovics, 1973], (EN1992-1-1, 2004)]. At failure, dislocations at discrete fracture zones occur, that are related to the specimen size and the capacity of the material for stress redistribution. In fiber composites the inclined shear band develops over the full height of the specimen and not over a limited length, whereas the specimen maintains integrity (i.e. fragments are not disconnected) presenting a non-negligible post-peak capacity up to large axial strains; this residual resistance is attributed to the confining effect of the fibers engaged in the lateral direction (restraining the dilation).

### 3.2 Members under reversed cyclic loading

Photos of the progressive failure of the Control and SHFRCC members are given in Figure 5. The results indicate that the SHFRCC member -even though it only had $\Phi 6/120$ perimeter stirrups as compared to the Control member who had $\Phi 6/80$ stirrups (3 legs parallel to the loading) - showed an improved behavior under reversed cyclic loading due to the presence of the fibers that contributed to the transfer of loads, the confinement of the regions under tension and compression, the increase of shear capacity of the member and the sustained load capacity up to large levels of deflections. Up to the point that the experiment with the SHFRCC was stopped the load-drift curve did not show any load drop leading to drift ductility in the order of $\mu_d=6/1=6$ (at the end of the test at which no load drop was observed) as compared with the Control beam that was designed as per the Eurocode, that attained a drift ductility of $\mu_d=4/1=4$ at a load drop of 20% of the maximum load.
Figure 5. Crack pattern at various cycle levels for (a) Control and (b) SHFRCC

The Shear Load-Deflection diagrams from the two members are depicted in Figure 6. The deflection recorded is based on the central LVDT1 that measures the deflection at the middle of the lower steel plate. Deflection is considered positive when the beam moves up and negative when moving down. The Load is positive when the top cross section is under tension. Shear in the spans is half the total Load $V=P/2$. In order to calculate the total deflection of the beams the average deflection at the supports $(DT5+DT6)/2$ is deducted from the total deflection of the central measuring LVDTs.

Figure 6. Shear Load-Vertical deflection of central rigid node for the (a) CONTROL and (b) SHFRCC

The diagrams of Figure 7 show the behavior of the two tested members under reversed cyclic loading. The Control member shows yielding of the reinforcement at a deflection of 6.8 mm and a shear load of 34.7 kN. Past the yield point the specimen showed some mild hardening with a peak shear load of 40.7 kN at a central rigid node deflection of 12.8 mm. After the peak load softening of the curve initiated with a remaining shear load of 25 kN in the last cycle of 36 mm deflection. During the second and third cycle of each target deflection the shear load capacity decreased, while the loading path of the following higher target displacement coincided with the lower target of the last excursion. As deflection increased the residual deformations of the specimen also increased, while the energy absorption of the consequent cycles decreased. The shear load-deflection curve of the SHFRCC member shows yielding at a deflection of 7.6 mm and 39 kN shear load. Past the yielding the load carrying capacity remained constant throughout the remaining cycles at a shear load of 41 kN even at a deflection level of 36 mm. The energy absorption capacity appeared to increase with each cycle and the members showed great ductility levels. The same type of behavior was observed if the right beams’ force-deflection diagram is considered as shown in Figure 7. As shown on the diagrams of Figure 7 the deflection values measured at the hinge zone of the beams had higher values than those of
the steel plate at the central rigid node. This was due to the accumulation of damage at the rigid node and the partial rotation of the node with the supporting beam.

Figure 7. Shear Load-Right Beam deflection for the (a) CONTROL and (b) SHFRCC

The Shear Load-Drift diagrams of the tested beams are depicted in Figure 8 (a). The drift $\theta$ is calculated as the deflection of the beam divided by the shear span length. This deflection though does not take into consideration the rotation of the rigid node with the control beam. To account for this the load-drift ratio diagrams were corrected: the adjusted drift is calculated by the rotation of the rigid zone $\beta$, where in our case is obtained from the differential deformations of LVDT2 and LVDT3 divided by their horizontal distance as per Parra-Montesinos & Chompreda (2006). The diagrams with the adjusted drift ratio values are depicted in Fig 8 (b) and show a tendency for increase in the drift values if compared to those without the adjustment. When the load-drift ratio diagrams of the CONTROL and SHFRCC beams are closely observed it is found that there is a reduction of “pinching” of the hysteresis loops when compared with the load-drift ratio diagrams when drift is adjusted taking into consideration the rigid node’s rotation. The decrease of load capacity in the diagrams is associated with the material degradation in the extreme layers. At the last level of loading large portions of the composite within the plastic region had spalled off the specimen and the reinforcement experienced excessive buckling.

Figure 8. a) Shear Load-Drift for the CONTROL (left) and SHFRCC (right) and b) Shear Load-Adjusted Drift for the CONTROL (left) and SHFRCC (right)
4. CONCLUSIONS

The results indicate that the SHFRCC specimen showed improved behavior under reversed cyclic loading due to the presence of the fibers that contributed to the transfer of loads, the confinement of the regions under tension and compression, the increase of shear capacity of the member and the sustained load capacity up to large levels of deflection (enhanced deformation capacity). After vigorous cyclic displacement reversals the test of the SHFRCC specimen was terminated, attaining the limit of stroke of the actuator. There was no evidence of strength loss in the load-resistance curve up to a ductility of six in terms of the relative drift ratio ($\mu_\theta=6$). In comparison, the Control specimen (reinforced as per the Eurocode for class M) attained a drift ductility of $\mu_\theta=4/1=4$ at a strength loss of 20% of peak load. The experimental findings suggest a possible future shift of construction towards High Performance Structures.

5. REFERENCES


