ASSESSMENT OF SFRC FLAT SLAB PUNCHING BEHAVIOUR - PART II: REVERSED HORIZONTAL CYCLIC LOADING

Nuno D. Gouveia

PhD student, Department of Civil Engineering, Faculdade de Ciências e Tecnologia, Universidade NOVA de Lisboa, CERIS, 2829-516 Caparica, Portugal, nunodinarte@gmail.com

Duarte M. V. Faria

Structural Engineer, Lausanne, Switzerland, duamvf@gmail.com

António Pinho Ramos

Professor, Department of Civil Engineering, Faculdade de Ciências e Tecnologia, Universidade NOVA de Lisboa, CERIS, 2829-516 Caparica, Portugal, ampr@fct.unl.pt

Corresponding author:

Nuno D. Gouveia

Address: Universidade NOVA de Lisboa, Faculdade de Ciências e Tecnologia, Departamento de Engenharia Civil, Campus da Caparica, 2829-516 Caparica, Portugal.

Tel: +351 212 948 580 Fax: +351 212 948 398 email: nunodinarte@gmail.com

Abstract

This paper presents an experimental study of four flat slab specimens subjected to a combined vertical and horizontal cyclic loading. The Steel Fibre Reinforced Concrete (SFRC) was used only in the local region of the slab-column connection, whilst the rest of the slabs were cast using normal concrete (NC). The specimens measured 4.15x1.85x0.15 m and were connected to two steel half columns by 0.25x0.25 m rigid steel plates, prestressed against the slab using steel bolts, to ensure monolithic behaviour. The specimens were tested using an innovative test setup system that accounted for important factors such as the ability of bending moment redistribution, line of inflection mobility and assured equal vertical displacements at the opposite slab borders and symmetrical shear forces.

Results show that the presence of SFRC in the slab-column connection region is effective in increasing the deformation capacity of slab-column connections, allowing the increase of horizontal *drift* ratios.

Highlights:

- Improvement in punching shear and deformation capacity of SFRC slabs;
- Benefits obtained in terms of ductility are extremely important in flat-slabs to avoid the brittle failure mechanisms;
- The presence of SFRC in the connection zone increases the moment and deformation capacity of slab-column connections.

Keywords: Punching; Fibre-reinforced concrete; Slabs & plates; Testing, structural elements.

Notation:

Δ	displacement
Δ_r	residual deformation
ε_y	yield extension of longitudinal reinforcement
ξ_{eq}	equivalent viscous damping ratio
ρ_f	fibre volume content
ρ	longitudinal reinforcement ratio
d	effective depth
f_{ccm}	mean value of concrete compression strength on cubes (150x150x150mm)
f_{cm}	mean value of concrete compression strength on cylinders (ϕ150x300 mm)
$f_{cm,MLS}$	mean value of concrete compressive strength on cylinders of the MLS slab specimen
$f_{ctm,sp}$	mean value of concrete split tensile strength on cylinders (300x150 mm)
f_y	average steel yield strength (top longitudinal reinforcement)
E	east
E_c	modulus of elasticity of the concrete
E_d	energy dissipation
E_s	modulus of elasticity of the longitudinal reinforcement
E_S	elastic strain energy
F	force
M_{max}	maximum unbalanced moment
M_{fail}	unbalanced moment at failure
N	north
S	south
V_{app}	total vertical load applied
$V_{MLS,exp}$	experimental punching capacity of the MLS slab specimen

V_{norm}	expected extrapolated punching load capacity
$V_{norm,NC}$	expected extrapolated punching load capacity considering the f_{cm} of normal concrete
W	west
Dramix 3D	hooked-end steel fibres Dramix® 3D 65/35 BN
Dramix 4D	hooked-end steel fibres Dramix® 4D 65/60 BG
NC	normal concrete
SFRC	steel fibre reinforced concrete

1. INTRODUCTION

The design of flat slab-column connections is frequently influenced by their punching capacity, especially when the structures are in regions of moderate to high seismicity (such as, among others, southern Europe, the west coast of North and South America and Japan). This is due to concentration of stresses and deformations in this zone, caused by the simultaneous action of gravity and seismic loads. If nothing is done to prevent this phenomenon, the punching failure can be sudden and without ductility (brittle). Furthermore, an improper design of a flat slab can lead to a partial or total progressive collapse of the structure, with the possibility of injuries of inhabitants and heavy material losses.

In flat slab structures located in areas of moderate to high seismic activity, the slab-column connections must be able to transmit gravity loads if the structure is subjected to earthquake-induced displacements. These actions, besides introducing eccentricities in loading, also lead to large inelastic rotations of the connection thus decreasing the punching load capacity. There are a variety of methods to improve the behaviour and punching capacity of slab-column connections (for example Wey and Durrani, 1991; Robertson et al, 2002 and Stark et al, 2004), and the use of SFRC is one of the most promising.

The seismic behaviour of such structures is still not fully understood by the scientific/technical communities. Experimental studies of interior slab-column connections subjected to a combined vertical and horizontal actions are still rare, especially of connections in slabs with steel fibre reinforced concrete (SFRC). According to the investigation carried out so far, very few studies focused on the effectiveness of SFRC under seismic actions in slab-column connections (Diaz, 1991; Tegos and Tsonos, 1996 and Cheng and Parra-Montesinos, 2010), demonstrating the necessity of more research in this promising area. These preliminary studies revealed that SFRC flat slab-column connections presented a better seismic performance when compared with conventional solutions.

The principle objective of the present study was to investigate the behaviour and load capacity of SFRC flat slab-column connections subjected to combined vertical and horizontal cyclic loading, simulating actions induced by earthquakes. As SFRC is more expensive than NC, this material was used only around the slab-column connection, whilst the rest of the slabs were cast using NC. The use of SFRC only around the slab-column connection under monotonic vertical loading has already been presented in Part I (Gouveia et al, 2018), and the presence of steel fibres within the concrete resulted in an increase in the load and deformation capacities of the slabs.

2. EXPERIMENTAL PROGRAM

2.1. Properties of the Specimens

The four models used in this study were designed to represent an interior flat slab-column connection of a current office building. The dimensions of the specimen were 4.15x1.85x0.15 m and were designed to be truncated at mid-span in the longitudinal North-South direction, (N-S), and up to the zero-moment line in the transversal East-West direction, (E-W). The steel central column had a 0.25 m square cross section and a total height of 2 m, with 1 m above and below the specimen. The column was pinned at the base to the strong floor and at the top was connected to the mechanical actuator.

The slab specimens were named F1.0_3D, F0.5_4D, F0.75_4D and F1.0_4D where the number after the letter F denotes the fibre content in volume and 3D and 4D denotes the type of fibre used. A fifth specimen (C-50) tested by Almeida et al (2016) is also included, to be used as reference. Specimen C-50 is geometrically similar, but it was cast entirely in normal concrete (NC) without fibres.

The four SFRC slab specimens were cast using SFRC on the central perimeter of the slab-column connection zone, whilst the rest of the specimen was cast using a NC without fibres. In specimens using 4D fibres, the central perimeter of the slab-column connection zone with SFRC was the same as that used in part I of this paper (Gouveia et al, 2018), in this case up to 3*d* of the column face. In specimen F1.0_3D the central perimeter of the slab-column connection zone made with SFRC was up to 2*d* of the column face. Figure 1 presents the geometry of the different concrete regions within the specimens.

Figure 2 shows the hooked-end steel fibres used on the FRC mixtures, one with Dramix® 3D and three with Dramix® 4D. Both steel fibres used are hooked-end, the Dramix® 3D 65/35 BN with a total length of 35 mm, a diameter of 0.55 mm and a nominal tensile strength of 1150 MPa and the Dramix® 4D 65/60 BG with a total length of 60 mm, a diameter of 0.90 mm and a nominal tensile strength of 1500 MPa.

The F1.0_3D specimen with 1.0% of volume content of Dramix® 3D steel fibres used the same materials and mix F1.0_R0.75 as Gouveia et al (2017). The other three specimens

(F0.5_4D, F0.75_4D and F1.0_4D) in this paper have the same materials and mixes as in Part I (Gouveia et al, 2018).

For each slab specimen; cubes (150x150x150 mm) and cylinders (ϕ150x300 mm) were cast and tested to determine the mechanical properties of the concrete. Tests were carried out approximately 60 days after specimens were cast, on the same days of the slab specimen tests. The average concrete compressive strength, splitting strengths and modulus of elasticity of the concrete cubes and cylinders are summarized in Table 1.

Table 1: Concrete compressive, splitting and modulus of elasticity test results.

Mistura		f_{ccm} (1)		f_{cm} (2)		$f_{ctm,sp}$ (3)		E_c (4)	
		MPa	COV	MPa	COV	MPa	COV	GPa	COV
C-50	NC	48.6	-	52.4	-	2.9	-	-	-
F0.5_4D	SFRC*	58.0	0.06	56.6	0.02	7.6	0.03	40.6	0.03
	NC	43.5	0.03	44.3	0.06	6.0	-	36.1	0.03
F0.75_4D	SFRC*	70.0	0.03	65.0	0.01	8.3	0.03	41.8	0.01
	NC	42.4	0.08	39.2	0.07	5.5	0.08	34.1	0.05
F1.0_4D	SFRC*	64.4	0.03	57.9	0.01	8.8	0.10	38.3	0.01
	NC	52.1	0.03	52.3	0.02	8.0	-	39.1	0.08
F1.0_3D	SFRC*	53.7	0.03	52.3	0.01	5.3	0.05	35.7	0.07
	NC	41.5	0.06	42.4	0.08	3.4	0.07	40.7	0.02

* - only in the central region of slab specimen;
f_{ccm} - mean value of concrete compression strength on 150 mm sided cubes (150x150x150 mm);
f_{cm} - mean value of concrete compression strength on cylinders (ϕ150x300 mm);
$f_{ctm,sp}$ - mean value of concrete split tensile strength on cylinders (ϕ150x300 mm);
E_c - mean value of concrete modulus of elasticity.

The top and bottom longitudinal reinforcement details are shown in Figure 3. At the column region the top longitudinal reinforcement ratio was approximately 1.0%. The higher effective depth for bottom and top reinforcement is oriented in the N-S direction. The nominal cover of both top and bottom longitudinal reinforcement was 20 mm. Table 2 presents the average effective depth (for both reinforcement directions), measured before casting and confirmed after testing in the saw cuts of the slabs, and the average values of the steel yield strength.

Table 2: Average effective depth and yield strength of longitudinal reinforcement of slab specimens.

Specimens	ρ_f (%)	d (mm)	f_y (MPa)		ε_y (%)		E_s (GPa)	
			$\phi10$	$\phi12$	$\phi10$	$\phi12$	$\phi10$	$\phi12$
C-50	0	118.4	534.9	525.8	0.27	0.26	198	200
F0.5_4D	0.5	118.3						
F0.75_4D	0.75	118.0						
F1.0_4D	1.0	117.5						
F1.0_3D	1.0	118.0						

ρ_f – fibre reinforcement ratio;
d – average effective depth;
$\mathbf{f_y}$ – average steel yield strength (top longitudinal reinforcement);
$\boldsymbol{\varepsilon_y}$ - yield extension of longitudinal reinforcement;
$\mathbf{E_s}$ - mean value of concrete modulus of elasticity.

2.2. Test Setup

The specimens presented in this study were tested using the test setup described in detail in Almeida et al (2016). Figure 4 presents a scheme and a photo of the test setup used in this study.

Most of the experimental tests of flat slabs under combined vertical and cyclic horizontal loading found in the literature review used simplified boundary conditions, such as borders that are vertically fixed and free of bending moments. This results in a static position of the zero-moment line making bending moment redistribution impossible whilst supported borders can absorb vertical load and reduce slab degradation. To overcome some of these limitations and simplifications, a new test setup was designed.

This test setup designed and used by Almeida et al (2016) made possible: vertical displacements at the opposite N-S slab borders; equal magnitude shear forces, bending moments and rotations at the N-S slab edges; mobility of the line of inflection location along the longitudinal direction and high vertical load ratios. This vertical load is completely driven through the column. Thus, for vertical loads, the shear forces and rotations at the N–S borders are zero and the vertical displacements, as well the bending moments are the same at the opposite longitudinal borders. For the horizontal action, the vertical displacements and shear forces are equal in magnitude, but with opposite signs.

The vertical load was applied through steel plates to the top surface of the slab in eight points to make a uniformly distributed load possible. After reaching the target vertical load, that vertical load was kept constant during the duration of the test. It was used a closed structure using a system of spreader beams and steel tendons as shown in Figure 5

a) and b) to apply the vertical load. That structure follows the slab horizontal deformations. The bottom beams were supported by the column corbels, driven all the vertical load through the column. This way, applied vertical loads were directed to the inferior column, avoiding exterior conflicts.

To fulfil the equality of the border's shear forces and vertical displacements, a passive mechanical see-saw-like system was designed as shown in Figure 5 c) and d), allowing the vertical displacements that depend exclusively on slab´s stiffness and force balance. Regarding shear forces at the opposite N-S borders, and for vertical loads, the balance beams move freely, without introducing shear forces on the slab edges, as expected at the slab's midspan. When horizontal displacements take place, shear forces are applied to prevent the slab's rigid body rotation. The balance system ensures that those forces are equal in magnitude and opposite in direction, again, as theoretically expected. As the slab stiffness degrades, deformation due to the constant vertical load increases. This ensures that the vertical load is driven to the column only, also simulating the effect of the whole slab.

In order to have positive bending moments at slab´s midspan (N-S edges on the slab specimens) and equal rotations at the opposite borders, it was used a system consisting in a double pinned steel frame suspended on slab's border by two vertical fixed columns, as shown in Figure 5 e) and f). The frame has variable length by means of a hydraulic jack and a load cell to measure the frame's axial load. For the vertical load, a positive moment of equal magnitude is introduced at both borders. For the horizontal action, the slab rotations are the same at the opposite borders, and the moments are equal, but with different sign.

The concrete slab was connected to the two steel columns of the test apparatus with four M24 bolts and rigid 250X250 mm square steel plates, with each bolt prestressed with a force of 240 kN.

2.3. Instrumentation

The applied load, the deformation of the slab specimens and the strains in some of the top longitudinal reinforcement bars were monitored during testing. This was similar to the experimental tests carried out by Almeida et al (2016).

To quantify the vertical load applied to the specimens, four load cells were installed, located over the four hydraulic jacks used to apply the vertical loading. The horizontal load was measured by a load cell positioned near the mechanical actuator (Figure 4). The vertical displacements of specimens were measured by eighteen displacement transducers positioned along the upper mid-line of the specimens, with fourteen in the longitudinal direction (N-S) and four perpendicular. The positioning of the vertical load and displacement points in specimens is depicted in Figure 6. Two displacement transducers were also used to measure the horizontal displacements of the slabs, one at the top of column and the other one at slab level. To monitor the upper longitudinal reinforcement bar strains, eight pairs of strain gauges, glued to bars near to the column connection, were used. The positioning of the strain gauges is shown in Figure 3 (a).

2.4. Test protocol

The expected punching capacity (V_{norm}) of the tested specimens was extrapolated using the experimental result of the MLS specimen tested by Almeida et al (2016) following Equation (1). The MLS specimen was tested under a vertical monotonic increasing load until punching failure, which occurred at a vertical load 323.4 kN. The f_{cm} from slab MLS was 31.6 MPa. The parameter 0.41 was proposed by Mamede et al (2013) based on a potential regression analysis, they concluded that the punching capacity depends on the concrete strength to an average power of 0.41 ($f_{cm}^{0.41}$). This value lies between the one presented by EC2 (2004) (1/3) and the one recommended by *fib* Model Code (2013) (1/2).

$$V_{norm} = V_{MLS,exp} \cdot \left(\frac{f_{cm}}{f_{cm,MLS}}\right)^{0.41} \tag{1}$$

where:

- V_{norm} is the expected extrapolated punching load capacity for each slab specimen;
- $V_{MLS,exp}$ is the experimental punching capacity of the MLS slab specimen from Almeida et al (2016);

- f_{cm} is the normal concrete mean compressive strength of cylinders of each slab specimen;
- $f_{cm,MLS}$ is the concrete mean compressive strength of cylinders of the MLS slab specimen from Almeida et al (2016).

The applied vertical load (V_{app}) took into consideration the slab specimen and the weight of test system apparatus (approximately 39.4 kN). Table 3 presents the total vertical load applied to the slab specimens.

Table 3: Total vertical load applied to each slab specimen.

Specimens	V_{app} *(kN)*	$V_{norm,NC}$ *(kN)*	V_{app} / $V_{norm,NC}$
C-50	203	398	0.51
F0.5_4D	196	371	0.53
F0.75_4D	194	353	0.55
F1.0_4D	208	398	0.52
F1.0_3D	192	365	0.53

V_{app} – total vertical load applied;
$V_{norm,NC}$ – expected extrapolated punching load capacity considering the f_{cm} of normal concrete without fibres;

The vertical load was initially applied at 30 kN/min rate until the target vertical load for each slab specimen was reached. It was then maintained constant by a hydraulic pump until the end of the test. After reaching the target vertical load, cyclic horizontal displacements were imposed on the column top, in a north-south (N-S) direction, at 8 mm/min speed for horizontal *drifts* of up to 3% and 16 mm/min speed for horizontal *drifts* above 3%.

Figure 7 shows the cyclic displacement protocol used in the experimental tests, consisting of three consecutive cycles for each horizontal *drift* up to 3.5% with two consecutive cycles for 4% *drift* and one cycle until the final *drifts* of up to 6%. The displacements in the S direction were arbitrated as positive.

3. EXPERIMENTAL RESULTS ANALYSIS

3.1. Vertical Displacement

It was possible to analyse the evolution of the deformation of the slabs during the tests from the displacement transducers positioned along the slab specimens in a N-S direction, as shown in Figure 6. The shape of the deformed slab specimens is shown in Figure 8, in the N-S direction, for the four slab specimens in this study and also for the C-50 slab tested by Almeida et al (2016). The first stage consisted of applying the vertical load until it reached the target value for each specimen and the second stage consisted in eccentric loading by imposing cyclic horizontal displacements to the top of the column.

Two events occurred during testing that had repercussions for the data displacement output. The first event occurred in specimen F0.5_4D due the fact that displacement transducers 1 malfunctioned from the beginning of the test. The second event occurred on specimen F1_3D because the displacement transducers D1, D2, D3, D12, D13 and D14 placed in this specimen did not have sufficient measurement range for such high deformations. This problem was solved in the remaining specimens by using displacement transducers with a larger measurement range.

In Figure 8 it possible to observe that the specimen behaviour during the vertical load imposition phase were, as expected, very similar. Higher horizontal *drifts* resulted in higher vertical deformations under a constant vertical load. The deformed profiles also show a discontinuity point in the deformed shape of the slab next to the column, at the side with a greater negative bending moment. The ultimate deformation capacity of the SFRC specimens are relatively higher, when compared to the reference slab without fibres.

The inflection point position at each moment of the test was also computed through the approximate equation of the deformed configuration of the specimens along the N-S axis for each side of the column. Figure 9 shows the variation of the inflection points during the test for each specimen. For the F0.5_4D specimen, only the north inflection point variation is shown because on the south side the data of displacement transducer D1 is missing. In the F1.0_3D specimen the inflection point variation is only shown until 2.0% of *drift* because for bigger *drifts* the measurement range of displacement transducers D1, D2, D3, D12, D13 and D14 was exceeded.

In Figure 9 it can be seen that there are two inflection points at around 24-25% of the slab span, when the vertical load was applied and before the start of the horizontal displacement cycles, which is close the theoretical inflection point position (22%). As the *drift* increases, the amplitude of the change of the position of the inflection points increases, because the moment due to the vertical load loses magnitude when compared to the eccentrically induced bending moment. For higher horizontal *drift* ratios, a change in the sign of the bending moment near the column can be observed. This means that the moment induced by the horizontal action is bigger that the hogging moment due to the vertical loading. This leads to a negative moment on one side and a positive moment on the opposite side of the column.

3.2. Strain results

Figure 10 shows the strains for the first cycle of each drift step of the top-northern reinforcement bars. The localization of the strain-gauges is indicated in Figure 3. The considered yield strain is also highlighted by a dotted horizontal black line. The strain results from specimen F0.75_4D are not shown because the strain gauges were damaged during the curing period.

From the results presented in Figure 10, it can be seen that the vertical load target (0.0% *drift* in Figure 10) for each specimen was reached without any of the monitored reinforcement bars yielding at that stage. There was however some cracking present in the slab-column connection zone due to the hogging moment, mainly parallel to the East-West direction (specimen´s smaller span).

The cracking opening in the SFRC zone was smaller than the cracking opening in the normal concrete without fibres even though the moments were higher in the SFRC zone. This is because the fibres help control the crack width for the initial *drifts* (Figure 11).

According to the test protocol presented in Figure 7, when the horizontal displacement is applied in the N-S direction, the hogging moment is increased on the north side as the *drift* increases, resulting in an increase of strains on that side. There is a corresponding decrease in strain on the opposite side, as shown in Figure 10.

Due the horizontal action, the effect of the unbalance moments is visible in the strain variations, and is more pronounced in the vicinity of the column. The first reinforcement bars to reach the yielding strain (horizontal dashed line) are normally the ones nearest to the column. For the SFRC specimens, and for higher drift values, the remaining instrumented bars started to shown signs of yielding, presenting a full flexural negative yield line close to failure. In the C-50 slab only the bar closest to the column yielded. The moment redistribution capabilities of the slab specimens were also verified, as the bottom cracks due to sagging moments, started to become wider near the edges, as the test advanced.

3.3. Hysteretic behaviour

The hysteretic charts are presented in Figure 12, where the horizontal load and respective unbalanced moment are related to the horizontal displacement and respective *drift*.

From the results presented in Figure 12 it can be concluded that the SFRC specimens exhibited higher unbalanced moments and *drift* capacity. All specimens presented an approximate linear behaviour for loading and unloading up to a *drift* of 0.5%, whilst no major loss of stiffness was noted. Above 0.5% *drifts* the loss of stiffness becomes more apparent. As the horizontal *drift* increases, the unbalanced moment converges to an asymptotic value.

Table 4 presented details of the hysteretic behaviour of the specimens. The C-50 specimen in Almeida et al (2016) reached 1.1% *drift* before exhibiting a brittle punching failure. This specimen reached a maximum horizontal load of 37.4 kN for that *drift*. F0.5_4D specimen presented a punching failure at a *drift* of 3.0% and a maximum horizontal load of 59.5 kN. A maximum horizontal load of 72.7 kN was achieved by the F0.75_4D specimen and the punching failure took place at 5.5% *drift*. The specimen that had most resistance to horizontal *drift* was the F1.0_4D specimen which reached a maximum horizontal load of 66.7 kN and a 6.0% *drift,* by the end of the protocol, maintaining the vertical and horizontal loads capacities without an obvious failure. The F1.0_3D specimen presented a punching failure at 4.0% *drift* and achieved a maximum horizontal load of 58.8 kN.

When comparing the two specimens with 1.0% of fibre content, it was confirmed that the F1.0_4D specimen exhibited greater capacity for *drift* and unbalanced moments showing that the use of Dramix® 4D fibres were more effective for this type of test conditions.

Table 4: Maximum and failure unbalance moments and respective *drifts*.

Specimens	V_{app} (kN)	M_{max} (kN) [drift (%)]	M_{fail} (kN) [drift (%)]
C-50	203	74.8 [1.1]	74.8 [1.1]
F0.5_4D	196	119.0 [2.5]	-112.0 [3.0]
F0.75_4D	194	145.4 [3.5]	132.2 [5.5]
F1.0_4D	208	133.3 [3.5]	-*
F1.0_3D	192	117.5 [3.5]	98.8 [4.0]

V_{app} – total vertical load applied;
M_{max} – maximum unbalanced moment;
M_{fail} – unbalanced moment at failure;
* – this specimen reached the end of protocol (6.0% *drift*) without losing load capacity´s.

Figure 13 present the hysteretic envelope curves of the cycles. It shows that the initial stiffness of all the specimens are similar. All SFRC specimens reached higher *drifts* than the reference specimen without fibres. Comparing the three specimens with Dramix® 4D fibres, it can be concluded that the *drift* capacity of the specimens was higher with the increased fibre volume content.

3.4. Energy dissipation capacity

According to Hose and Seible (1999) an equivalent viscous damping ratio (ξ_{eq}) can be evaluated to quantify the energy dissipation capacity of the specimens. This parameter relates the dissipated cycle energy to the energy needed to linearly reach the peak of each cycle. Figure 14 shows the equivalent viscous damping for asymmetric hysteresis loops.

Equation (2) was defined by Hose and Seible (1999) to calculate the asymmetric equivalent viscous damping ratio:

$$\xi_{eq} = \frac{1}{4 \cdot \pi} \cdot \left(\frac{E_{d1}}{E_{S1}} + \frac{E_{d2}}{E_{S2}} \right) \tag{2}$$

where:

- E_{d1} – energy dissipation for positive displacement cycles;
- E_{d2} – energy dissipation for negative displacement cycles;
- E_{S1} – elastic strain energy for positive displacement cycles;
- E_{S2} – elastic strain energy for negative displacement cycles.

Viscous Damping Ratio values under 10% are indicative of a structural system with a non-linear elastic behaviour and therefore with low energy dissipation capacity (Hose and Seible, 1999). Figure 15 presents the viscous damping ratio for each specimen.

For low horizontal *drift* ratios, all the specimens presented narrow hysteretic graphs, where the load and unload branches are very close, indicating low energy dissipation capacity, which is verified by the Viscous Dampening Ratio, shown in Figure 15. As the horizontal *drift* increased, the Viscous Dampening Ratio values also increased. In each specimen, the Viscous Damping Ratio values decrease between different cycles of the same *drift* ratio, meaning less capacity for energy dissipation. The total dissipated energy is much higher in the SFRC specimens, mainly due to higher horizontal *drift* ratios and more cycles achieved.

3.5. Failure shape of specimens

Figure 16 shows the saw cuts from all specimens. The specimens C-50, F0.5_4D, F0.75_4D and F1.0_3D presented a punching shear failure, whereas the specimen F0.75_4D had large bending cracks in the connection zone. The specimen F1.0_4D maintained the load and deformation capacity until the end of the test protocol, but we can observe in Figure 16 d) that there were bending cracks in the connection zone and a punching shear crack had also already started developing, being mostly visible in the North side.

4. CONCLUSIONS

This paper presents the results of an experimental study of SFRC slab/column connections subjected to combined vertical and horizontal cyclic loading. Four specimens were compared with different slab-column connection specifications. Connection zones composed of SFRC were compared with normal concrete and two types of hooked-end steel fibres were used; Dramix® 3D 65/35 BN and Dramix® 4D 65/60 BG in a volume content of 0.5%, 0.75% and 1.0%. The gravity shear ratio applied to the specimens was approximately 50%. The four SFRC test results are also compared with the C-50 specimen without fibres tested by Almeida et al (2016).

An innovative test setup developed by Almeida et al (2016) was used to perform the tests. This new test setup allows for the mobility of the zero-moment line, bending moment redistribution, equal vertical displacements at the opposite borders, symmetrical shear forces and the application of high vertical load ratios, all driven through the column.

From the test results presented, the main conclusions are:

- The application of SFRC in the slab-column connection zone improves the load and deformation capacity of the slabs under combined vertical and horizontal cyclic loading. The SFRC specimens tested in this study showed a minimum *drift* capacity of 3% for a slab with 0.5% volume content of fibres and a maximum of 6% *drift* without losing the load capacity for a slab with 1.0% volume content of fibres, both with Dramix® 4D;
- The *drift* capacity in SFRC specimens was higher than the C-50 specimen without fibres, achieving a minimum *drift* of 3% with 0.5% of fibres whilst the C-50 specimens achieved a *drift* of only 1.1% This demonstrates the potential of SFRC applied in this type of structural connection even with a low fibre volume content;
- The deformation capacities of SFRC specimens are larger than the reference slab without fibres;
- All the tested SFRC specimens exhibited higher energy dissipation than the fibreless reference slab. These results show that SFRC specimens have the potential to have a better behavior under horizontal actions, even for high horizontal *drifts*;
- For the initial target vertical loads, none of the tested specimens exhibited longitudinal reinforcement bar yielding;
- The obtained viscous damping ratios show the tendency to increase slightly for higher *drifts*. In each specimen, the viscous damping ratio values decrease between different cycles of the same *drift* ratio, meaning less capacity for energy dissipation.

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CAPTIONS

Figure 1. Different concrete regions in different specimens.

Figure 2: Hooked-end steel fibres used in this study: a) Dramix® 3D 65/35 BN and b) Dramix® 4D 65/60 BG.

Figure 3. Reinforcement details and strain gauge position (dimensions in millimetres): (a) top longitudinal reinforcement and (b) bottom longitudinal reinforcement.

Figure 4. Scheme and photo of the test setup used.

Figure 5. Setup system: a) vertical load distribution (undeformed), b) vertical load distribution (deformed), c) compatibilization of shear forces and vertical displacements at the slab edges (undeformed), d) compatibilization of shear forces and vertical displacements at the slab edges (deformed), e) compatibilization of bending moments and rotations at the edges (undeformed) and f) compatibilization of bending moments and rotations at the edges (deformed).

Figure 6. Vertical load and displacement transducer points positioning.

Figure 7. Cyclic displacement protocol.

Figure 8. Vertical displacements for the first south cycle of each *drift* step for slab specimen.

Figure 9. Inflection point position throughout the test for each slab specimen.

Figure 10. Strains for the first cycle of each drift step.

Figure 11. Crack opening in SFRC and normal concrete without fibres for the intended vertical load.

Figure 12. Hysteretic behaviour of individual slab specimens.

Figure 13. Envelope curves for positive horizontal displacement.

Figure 14. Equivalent viscous damping for asymmetric hysteresis loops.

Figure 15. Viscous damping ratio of each slab specimen.

Figure 16: Saw cuts from all slab specimens.