STRAIN HARDENING FIBRE REINFORCED CEMENT COMPOSITES FOR THE FLEXURAL STRENGTHENING OF MASONRY ELEMENTS OF ANCIENT STRUCTURES

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Abstract:

To assess the strengthening ability of a strain hardening cementitious composite (SHCC), a layer of SHCC was applied to masonry beams subjected to bending. When compared to the strengthening performance of steel fibre reinforced self-compacting concrete (SFRSCC) layer for this type of brittle beams, the SHCC presented better workability in fresh state, and provided a higher load carrying capacity and deflection ductility even with a smaller layer thickness. By using the data derived from the experimental tests with the constituent materials of the strengthened masonry beams, the behaviour of the tested strengthened masonry beams was numerically simulated with good accuracy.

Keywords: strain hardening cementitious composite; steel fibre reinforced self compacting concrete; masonry; flexural strengthening; ECC; finite element model.

1. Introduction

Masonry construction technology has been largely used in the majority of countries. According to an inventory done in 1989, 70% of residential buildings in USA are masonry-based construction [1]. To increase the load and the deformational capacity of masonry structures, mainly for lateral loadings produced by seismic and wind events, several strengthening techniques are being developed and applied, such as: ferrocement, shotcrete, grout/epoxy injection, and attachment of external steel reinforcements [2-7]. However, these techniques are mostly time consuming, of intensive labour, prone to interfere with the normal functioning of the spaces, costly, and in general they increase undesirably the mass of the structure. A detailed analysis of the major drawbacks of these conventional strengthening techniques is provided by ElGawady et. al [8].

Numerous studies have been done recently on the application of fibre reinforced polymers (FRP), as a more advanced strengthening technique for masonry structures [9-11]. Although the advantages of FRP, such as high strength to weight ratio, corrosion resistance, and fast application that demonstrate the potentialities of these composites for the structural rehabilitation when compared to conventional materials, some disadvantages still exist. In fact, the properties of the organic resin used to bond FRP to the substrate are detrimentally affected by high temperatures. Furthermore, these resins do not work properly when applied on damp surfaces. The tensile strength and the fracture energy of the substrate materials are much lower than those of the resins, which favour the occurrence of precocious FRP-substrate debonding. Therefore, the maximum strain level that can be achieved in a FRP strengthening system is much lower than its ultimate strain capacity, mainly when applied to masonry structures.

To overcome the organic resin related deficiencies of FRP systems, a novel composite was designed, which is currently designated in the literature as textile reinforced mortar (TRM). The organic resin used for FRP was substituted with an inorganic binder such as cement. To achieve a strong matrix/polymer bond interface, FRP sheets were replaced by textiles. Better impregnation of the textile fibres by the cementitious mortars and the mechanical interlock provided by the trapped matrix inside the textile meshes improved significantly the bond characteristics of the fibrous phase to the matrix in TRM systems. TRM has the potential of forming multiple cracks diffusely distributed, being a composite capable of increasing the ductility and the energy dissipation capacity of structures reinforced with TRM systems, mainly those subjected to dynamic loading conditions. The research conducted on the application of TRM for seismic strengthening of masonry structures showed promising increase in terms of deformability. However, in terms of ultimate load carrying capacity, the strengthening effectiveness of the TRM technique was only 70% of the solution based on the equivalent FRP jacket (identical cross sectional area of fibres) [12, 13].

The main disadvantage of TRMs is attributed to the relatively large crack width that forms at serviceability limit state (SLS) conditions, which can have repercussions in terms of durability protection for the strengthened structures, and significant decrease of stiffness with the corresponding deflection increment at cracked phase for the SLS[14, 15]. Moreover, the results of some studies on the long term behaviour revealed the strength loss of TRM systems as a result of degradation of the fibres or continued densification of the matrix [16-18].

Another generation of advanced composites have been developed, demonstrating high potential for new constructions [19] and for the structural rehabilitation of existing structures [20, 21]. These composites incorporate short fibres mixed with a cementitious matrix, and are mainly categorized in two groups, regarding their post cracking tensile strain response: a) ordinary fibre reinforced concretes (FRC) with a strain softening response; and b) strain hardening cementitious composites (SHCC). The initial version of SHCCs was tailored by Li and co-workers [22, 23] and was designated as engineered cementitious composite (ECC). ECC is composed of 2% polyvinyl alcohol (PVA) or polyethylene (PE) fibres mixed into a specific matrix with maximum ingredients size limited to 250 µm. This composite exhibits a tensile strain hardening capacity up to 6%, which means that, after crack initiation the tensile strength continues increasing up to a tensile strain of 6%, where the micro-cracks degenerate in a macro-crack, followed by an ultimate softening response.

In a previous research program completed at the University of Minho, the application of a thin layer of steel fibre reinforced self-compacting concrete (SFRSCC) was found to be capable of increasing the flexural capacity and ductility of masonry beam elements [24]. Difficulties were found in filling the joints of the masonry with this material, and dispersion on the increase level of the load carrying and deflection capacities was quite significant. This was caused by the low volume percentage (V_f=0.57%) of relatively long hooked ends steel fibres (l_f=35 mm) adopted as the reinforcement, which favoured the occurrence of a quite different number of effective fibres bridging the failure surfaces of the strengthened masonry beams. In fact, the SFRSCC exhibited tensile strain softening, which has limited the effectiveness of this strengthening technique. Corrosion of steel fibres can also be a concern, mainly if the crack width of the SFRSCC layer overcomes a certain limit (0.25 mm according to available research [25]) and if the strengthened structure is subjected to aggressive environmental conditions.

On the other hand, due to the relatively small crack width that SHCCs can guarantee for serviceability limit state conditions (average values of about 60 µm), the durability of the structure to be strengthened can significantly increase, since self-healing character can be promoted [26, 27]. These unique characteristics along with its high ductility have motivated some researchers to explore the use of SHCCs for the strengthening of masonry structures [28-31].

Recently, Esmaeeli *et al.* [32] has started the development of a SHCC for the manufacture of thin panels in the context of structural rehabilitation. Although the developed SHCC has almost the same mix composition as the ECC, it was, however, tailored for larger grain size of silica sand available in Portugal, in order to produce a more sustainable composite material. Furthermore, the SHCC was tailored to get the properties that can avoid the difficulties presented by the SFRSCC layer for the flexural strengthening of masonry beams.

Three fibre mortar compositions were designed and tested with the aim of obtaining a composition with sufficient workability and mechanical properties required for this strengthening application. The strengthening effectiveness of the tailored SHCC material was assessed by applying a layer of the SHCC to masonry beams that were tested under a 4-point load bending configuration. Finally, the tests carried out with the strengthened masonry beams were simulated by a FEM-based computer program capable of modelling the nonlinear behaviour of the constituent materials of these beams.

2. Development and characterization of the SHCC

The development of a mix composition able to produce the required material characteristics was executed in two steps: i) Tailoring the rheological properties through experimental testing of three mortar mix compositions; ii) Characterizing the tensile behaviour by performing tensile tests with dog bone type specimens manufactured from these mixes. Tensile tests with notched SHCC specimens were also carried out to characterize the single crack opening response of the final composition, and to provide information for the fracture mode *I* parameters of the constitutive model adopted in the numerical simulations.

2.1. Tailoring the rheological properties

For the mortar matrix design, the ratios of the dry ingredients, the sand (S), cement (C) and fly ash (FA), were based on the knowledge acquired in previous development of SHCC [32], and they were kept constant throughout the mix design. Mortar mixes for three different weight ratio of water (W) to binder (cement and fly ash) were produced. The ratio of superplasticizer (SP) to binder was kept constant. The plastic viscosity of mortar was controlled in order to reduce the segregation of the particles. For this purpose a viscosity modification agent (VMA) was introduced to the mortar mix in an appropriate dosage. Table 1 includes the abbreviations and the density of the constituent materials of the compositions.

The dry ingredients were firstly mixed in an automatic mixer. The superplasticizer and half of the water were combined and added in the second step. Finally the rest of the water and the VMA were combined and introduced into the mix. The detailed mix procedure is shown in Table 2. For each mix composition mini slump diameter (D_{MS}) and V-funnel flow time (T_{VF}) were measured as an indication for its fluidity and deformability, respectively. Further information about the specificities of the test procedures can be found elsewhere [33]. In terms of the fluidity of the material, the objective was to obtain a v-funnel time, T_{VF}, in the range of 10 to 20 seconds. This was combined with the goal of a D_{MS} of about 300 to 350 mm, in order to provide enough deformability of the mortar. These objectives were set with the expectation that the addition of PVA fibres (2% by volume) would affect both fluidity and deformability. Hence, the mortar needs to have certain rheological characteristics prior the addition of fibres in order to minimize this adverse effect at the composite level. The corresponding rheological test results of these mortars are shown in Table 3.

To produce the SHCC, new mortar mixes were prepared and 2% in volume of PVA fibres were added to each one. Nominations of these SHCCs with respect to their matrix, and the results of the mini slump tests are indicated in Table 4. According to the supplier, these fibres, which original name is RECS, are 8 mm in length and 40 micrometers in diameter with a tensile strength and an elasticity modulus of 1600 MPa and 40 GPa, respectively. The effect of the addition of fibres on the deformability of the mortar compositions is represented in Figure 1. As expected, the addition of fibres significantly and consistently has reduced the mini-slump diameter of the mixes.

2.2. Tensile behaviour

To characterize the tensile behaviour of the fibre reinforced mortar mixes, direct tensile tests were performed on two dog bone type specimens that were cast from each of the three mix compositions. The specimens were de-moulded 24 hours after casting and placed in a moisture controlled room until 24 hours before the test. A servo-controlled direct tensile test machine with a 50 kN load cell was used for the tests. The test setup and specimen geometry is presented in Figure 2a and 2b, respectively. Two ribbed grips secured the specimen with both ends fixed. The fixing conditions of the grips were optimized to minimize both the in-plane and out-of-plane bending of the specimen. One LVDT with a measuring length of 130 mm (distance between support points of the LVDTs, L_{meas}) was installed on each lateral edges of the front face of the specimen to measure its axial deformation, Figure. 3. The tensile strain of the specimen was evaluated by dividing the average of the displacements registered in these LVDTs by L_{meas}. A third external LVDT was fixed to the testing frame (Figure 2a) and was used to control the test loading conditions by imposing a displacement rate of 5 μm/sec to the top grip of the equipment.

The reported values from each of the tensile tests were: stress at the first crack initiation, f_{cr}, and its corresponding strain, ε_{cr}; ultimate strength, f_{ctu}, and its corresponding strain, ε_{ctu}; and tensile strain ductility, $\varepsilon_{ctu}-\varepsilon_{cr}$. The stress at the first crack initiation was defined as the point in the stress-strain relationship where a significant decrease in the initial tensile stiffness of the specimen has occurred.

The average results from the direct tensile tests performed on the specimens at the age of 14 days are presented in Table 5 and Figure 4. The results indicate a direct correlation between the W/B ratio and the strain ductility: the higher is the W/B ratio the larger is the strain ductility. An inverse relationship is also observed between the W/B ratio and the stress at first crack initiation: the higher is the W/B ratio the lower is f_{cr}. No correlations can be established between the W/B ratio and f_{ctu}.

Considering both rheological (fluidity and deformability) and mechanical properties (ultimate tensile strength and tensile strain ductility), T42-SHCC was considered the most appropriate composition for the flexural strengthening of masonry beams. This mix composition showed enough fluidity and deformability in the fresh state, a strain ductility that is intermediate to the other compositions, and the highest tensile strength. Figure 5 shows the typical diffuse crack pattern formed in the tested specimens. A detailed description of these tests and all the results can be found elsewhere [33].

2.3. Fracture parameters

In the numerical simulations of the flexurally strengthened masonry beams, a smeared crack model will be used, since, as will be observed in the following chapter, a diffuse crack pattern is formed in the selected SHCC during the loading process of the beams. However, the accuracy of this model depends significantly on the values attributed to the parameters that define the fracture mode I process of the SHCC, namely, the fracture energy, G_f^I, which is the energy to create a crack of unit area. To obtain the G_f^I and the shape of the relationship between the stress and the crack width, $\sigma-w$, direct tensile tests were performed with notched SHCC specimens, capable of forming a single crack at the notched plane. For this purpose, another batch of the same mix was prepared, and a square panel with a thickness of 20 mm was cast. The panel was cured at the same conditions of the dog bone specimens. Four specimens with a length of 240 mm and a width of 70 mm were cut out from the panel. A notch was cut in each lateral side at mid-height of the specimen (120 mm from each extremity) with the geometry indicated in Figure 6a and 6b. The crack opening displacement was measured using four LVDTs (Figure 6c), allowing also to measure the in plane and out-of-plane rotation of the specimen. Similar to the test setup for dog -bone specimens, an external LVDT was used to control the test with the same displacement rate. The test setup is shown in Figure 6c. For the purposes of estimating the fracture energy of the selected SHCC, the average results from all four notched specimens were used. The fracture energy is calculated as the area under the $\sigma - w$ diagram, where σ is the applied force divided by the net area of the notched plane, and w is the average of the displacements measured in the four LVDTs. The envelope and the average $\sigma - w$ curve for the test results of these specimens are presented in Figure 7. The average value for the calculated fracture energy is 3.7 N/mm, which is about 100 times higher than the value of the homologous non-fibrous mortar.

3. Assessment of the effectiveness of the flexural strengthening technique

The effectiveness of the tailored SHCC material for the flexural strengthening of masonry elements subjected to bending loading configuration was assessed. For this purpose masonry beam elements were constructed, strengthened with a layer of the SHCC material, and then tested in four-point bending. The results of these tests are compared with those obtained in a previous experimental program where steel fibre reinforced self-compacting concrete (SFRSCC) was used for the flexural strengthening of this type of elements.

3.1. Masonry beams, strengthening procedures, test setup, and monitoring system

The geometry of the masonry beams strengthened with SHCC layer was based on the geometry of the beams tested in a previous experimental program, Figure 8. The beams were 805 mm in length and consisted of eleven clay bricks bonded with low strength mortar (LSM) joints of an average thickness of 20 mm. The average thickness, height and width of the bricks were 55, 105 and 205 mm, respectively. The width of the masonry beam is equal to the width of the brick. The bricks and the LSM used for the construction of the beams are similar to the ones used by Häβler, and their relevant properties can be found elsewhere [34].

A layer of SHCC material is applied to the surface of the masonry beam that will be submitted to tension, in order to increase the flexural capacity and the deformability performance of these beams. The SHCC layer was applied to the top of the masonry beams and then the beams were inverted for the bending tests. Two beams were strengthened with a SHCC layer thickness of 15 mm, and two with a SHCC layer thickness of 20 mm. A control beam was also built without a SHCC layer. The bricks were placed into the moulds and LSM was cast between the bricks. For the beams to be strengthened with SHCC layers, the joints were cleared of mortar to a depth of 20 mm in order to be filled with SHCC (Figure 8). After 24 hours, the LSM was cured for seven days under wet towels. The SHCC layer was cast on the top of the beams 14 days after the LSM was cast. The SHCC layer was screed to the level of the pre-sized moulds. Thus, in some places, due to the lack of uniformity of the dimensions of the ancient bricks used for the beams, the thickness of the SHCC layer was different from the planned thickness. The SHCC was externally vibrated to release entrapped air. After 24 hours, the beams were again cured for seven days using wet towels. De-moulding of the beams was done after 12 days, and the four-point bending tests took place after 13 days. Therefore, the LSM and the SHCC had 27 and 13 days when the tests were executed.

Figure 9 shows the setup of the four-point bending tests. The supports for the beams were placed at 27.5 mm from the extremities of the beams, conducting to a span length of 750 mm. Loading points were set at 150 mm from the mid-span of the beam, where the actuator with a 50 kN load cell was applied. Five LVDTs were used to measure the deflection of the beam. These LVDTs were supported in a metal bar fixed at mid-height of the beam in the alignment of the supports of the beam, in order to assure that the LVDTs only register the deflection of the beam. The LVDT at the mid-span was also used to control the test, at a displacement rate of 3 μm/sec. The tests were stopped when fracture occurred through the entire beam section, or when the vertical deflection of the beam reached 10 mm.

3.2. Flexural tests: results and analysis

The reference masonry beam has failed during its de-moulding process, by debonding at the LSM-brick interface in the centre joint, Figure 10, due to the very low bond strength between bricks and LSM. The quite low bond strength obtained in the bond tests between LSM and brick elements carried out by Häβler [34] justifies this behaviour. Due to the negligible flexural capacity of the reference masonry beam it can be concluded that any increase in terms of load carrying capacity and deformability in the strengthened masonry beam will be assured exclusively by the influence of the SHCC layer.

The two masonry beams strengthened with a SHCC layer of 15 mm thickness (B15-1 and B15-2) had a quite similar force-deflection (*F*-*d*) relationship (Figure 11), with almost linear-elastic behaviour up to a load level of approximately 3.3 kN and a deflection of 0.4 mm. This was followed by a pronounced nonlinear behaviour due to the formation of a diffuse crack pattern in the SHCC, mainly in the “pure bending zone”, Figure 12. These specimens failed by the formation of a macro-crack that, after has crossed the SHCC layer, has progressed, in general, through the interface between a brick and the cementitious materials in the joint. For the maximum load (F_{max}) and for the deflection at failure (d_{max}), an average value of 9.57kN ($\bar{F}_{\max}$) and 5.65 mm ($\bar{d}_{\max}$) was obtained, respectively. The masonry beams strengthened with a SHCC layer of 20 mm thickness (B20-1 and B20-2) had more heterogeneous behaviour in terms of *F-d* than B15-1 and B15-2 beams. This is caused by the larger irregularities on the SHCC layer thickness along the length of the B20-1 and B20-2 beams, in consequence of using bricks with more irregular height for these beams. Like in B15 series, the beams of B20 series presented a quite similar maximum load, with $\bar{F}_{\max}$ =12.89 kN. The deflection at failure was, however, more different, with $\bar{d}_{\max}$ = 8.2 mm, but both beams presented a deflection at failure larger than the one of beams of B15 series (d_{max}=9.7 mm and 6.6 mm for the B20-1 and B20-2, respectively). The relevant results obtained in the masonry beams flexurally strengthened with SHCC are included in Table 6.

Except B20-2 beam, all the other beams had the same failure mode, with a first phase where a diffuse crack pattern is formed in the SHCC layer, followed by the failure crack localization and its propagation through the brick/SHCC and brick/LSM interfaces. In the B20-2 beam, the flexural failure crack formed in the SHCC near to one of the loaded sections, has progressed through the brick unit with an inclined shear configuration, and finally has followed at the brick/LSM interface up to the collapse of the beam. The lower deflection capacity of B20-2 beam, when compared to B20-1 beam, can be justified by the higher brittleness of the shear crack propagation through the brick element.

For the purpose of comparing the force versus deflection curves (*F-d*) of the beams strengthened with SHCC layer of 15 and 20 mm thickness, and SFRSCC layer of 30 mm thickness, Figure 13 represents the *F-d* curves obtained in the beams strengthened with SFRSCC, where T0i (i=1 to 3) represents the beams where the LSM in the joint was not partially replaced by SFRSCC, and T3i (i=1 to 3) designates the beams where a thickness of 30 mm of LSM was replaced by SFRSCC [34]. The relatively high dispersion of results obtained in the beams flexurally strengthened with SFRSCC is quite evident in this figure, since it was verified that $\bar{F}_{\max}$ is almost directly dependent on the number of fibres bridging the fracture failure surface [24], and this number was relatively different between the strengthened beams. The average *F-d* curves of the T0 and T3 are compared with the *F-d* curves of B15 and B20 in Figure 14, where it is verified that the beams strengthened with a layer of 15 mm of SHCC presented a $\bar{F}_{\max}$

(=9.57kN) that is almost equal to the $\overline{F}_{\max}$ (=10.18kN) of the beams strengthened with a layer of 30 mm of SFRSCC and 30 mm LSM replaced by the SFRSCC (T3 series). Another remarkable result is the much larger $\overline{d}_{\max}$ (=5.65mm) of series B15 when compared with the $\overline{d}_{\max}$ of both T0 (=1.65mm) and T3 (=1.68mm) series. Figure 14 also shows that the beams flexurally strengthened with a layer of 20 mm thickness of SHCC presented a much larger $\overline{F}_{\max}$ and $\overline{d}_{\max}$ than those corresponding to the series of masonry beams strengthened with a SFRSCC layer. This high performance of the beams flexurally strengthened with SHCC is caused by the strain hardening character of the developed SHCC, with a strain at peak tensile load much larger than the strain at peak tensile load of the corresponding value for SFRSCC.

4. SIMULATIONS OF THE BEAM TESTS BY FINITE ELEMENT MATERIAL NONLINEAR ANALYSIS

4.1. Model

To simulate the behaviour of the SHCC flexurally strengthened masonry beams, a finite element method (FEM) software package, FEMIX 4.0, was used. Taking the advantage of the structural symmetry of the beam, only half of the beam was simulated. The adopted mesh, loading and support conditions are presented in Figure 15. In this figure the bricks are enumerated (1 to 6), and the borders between LSM and bricks, LSM and SHCC and SHCC and bricks are highlighted in order to easily distinguish the location of these elements in the mesh. Eight nodded plane stress elements with 2×2 Gauss Legendre integration scheme were used. In the case of the beam type reinforced with a SHCC layer thickness of 15 mm (FEM_B15), three rows of finite elements with a depth of 5 mm were adopted, while for the B20 four rows of finite elements were applied. The analyses were executed by using the arc length method available in FEMIX, by imposing a displacement rate in the loaded point. A multi-directional fixed smeared crack model was used to simulate the crack formation and propagation in the SHCC, LSM and brick units. This model is described in detailed elsewhere [35]. The tri-linear diagram that defines the fracture mode I initiation and propagation of the intervening materials is represented in Figure 16. In this diagram ε_n^{cr} and σ_n^{cr} is the strain and the corresponding stress normal to the crack, respectively, f_{ct} is the stress at crack initiation, G_f^I is the mode *I* fracture energy, l_b is the crack band width, α_i and ξ_i are the parameters that define the shape of this diagram, and the ultimate strain normal to the crack ($\varepsilon_{n,u}^{cr}$) is obtained by the following equation (more information regarding the definition of these parameters can be found in [35]):

$$\varepsilon_{n,u}^{cr} = \frac{2}{\xi_1 + \alpha_1\xi_2 - \alpha_2\xi_1 + \alpha_2} \frac{G_f^I}{f_{ct} l_b} \tag{1}$$

Above this strain value the crack is assumed completely open, without capacity to transfer normal and shear stresses. The values of the α_i and ξ_i parameters for the SHCC were determined based on the stress-crack width obtained from the notched tensile SHCC specimens, and by transforming the crack width into a normal crack strain by using the concept of crack band width, l_b. Table 7 includes the values for the model parameters adopted in the numerical simulations. By assuming the fracture energy as a material property that is used to define the σ_n^{cr} - ε_n^{cr} relationship, and adapting the area behind this diagram according to crack band width that is a characteristic length of the cracked integration point ($l_b = \sqrt{A_{IP}}$, where A_{IP} is the area of the integration point), the results of the simulations are independent of the mesh refinement[36].

In the preliminary material nonlinear FEM simulations, the values determined in the experimental tests with the constituent materials of the strengthened masonry beams were used to define the corresponding constitutive models. However, the simulated force-deflection responses during the first loading phase were much stiffer than the ones recorded experimentally. Modelling of the debonding of the LSM-brick, LSM-SHCC or/and SHCC-brick as a potential source for this divergence was then investigated by using interface finite elements. The constitutive laws of these interface finite elements were capable of simulating the shear sliding based on the principles of the Mohr-Coulomb theory, as well as the free separation of the contact surfaces in tension [37]. However, the results of this study showed that debonding has just marginal effect on the initial bending stiffness.

A parametric study was then executed to assess the influence of the parameters of each material model on the initial stiffness of the response of the simulated strengthened masonry beams, having been concluded that this stiffness is quite dependent on the Young's modulus attributed to the LSM (E_m) and the Brick (E_B), but just slight dependence of the Young's modulus of the SHCC *(E_S)*. Since the material property of the brick is independent of the casting conditions, therefore, E_m was taken as the only source for discrepancy in the initial slope of load-deflection response. Although, the value of E_m, extracted out by inverse analysis, is abnormally lower than the value recorded in the tests on the LSM material, considering the cast conditions of the LSM, this discrepancy on the E_m values maybe justified. In fact, the joints were filled by pouring the LSM without any vibration. A closer inspection of the appearance of the fracture surfaces progressed through the LSM and at the LSM-brick interface in the tested masonry beams revealed a microstructure with much higher porosity than observed in LSM specimens, which can be justified by unfavourable casting conditions. Furthermore, during the curing of the LSM, shrinkage would have also contributed for the development of cracks connecting existing internal pores and flaws, leading to a decrease in the stiffness of the LSM. Therefore, the value adopted for the E_m in the numerical simulations was the one resulting from the fitting process of the initial phase of the force-deflection response of B15 beams.

4.2. FEM results and discussion

The *F-d* relationship obtained numerically for the B15 (FEM_B15) and B20 (FEM_B20) beams is presented in Figure 17a and 17b, respectively. The deformational response of these beams in the post-cracking stage is decomposed in the A-B, B-C, C-D and D-E phases, and the representative crack patterns for these phases are represented in Figure 18. According to the Figure 16, the colours of the cracks indicate the following cracking status: red=>crack in opening; green=>crack in closing; cyan=>crack in reopening; Blue=> closed crack; Pink=>crack completely open. The crack patterns for these phases are similar in both types of beams. Phase A-B corresponds to the propagation of cracks in the SHCC layer, mainly in the zone between the point loads, with the consequent decrease of stiffness when compared to the stiffness of un-cracked phase. In the B-C phase, macro-cracks have started being formed at the brick-SHCC lateral interface of the left faces of the bricks number 5 and 6, which has conducted to a further loss of stiffness. This effect was more pronounced in the B15 beam due to the small thickness of the SHCC layer. In the C-D phase these macro-cracks have progressed through the LSM-Brick interface, and due to the relative small LSM-Brick bond strength, the loss of stiffness was much higher than in the previous phases. Due to the relative high bond strength between bricks and SHCC applied into the joints, in this phase cracks have also progressed through the bricks number 5 and 6. The last D-E phase corresponds to the localization of the failure crack at the left lateral face of the brick number 5, while in the other zones the cracks turn into closing and completely closed status, which reproduces closely the typical failure mode registered in the experimental tests.

In Figure 17 the *F-d* relationships determined numerically are compared to the average *F-d* curves of the B15 and B20 series of masonry beams. It can be concluded that the model is capable of simulating with high accuracy the response registered up to the failure load of the tested beams, provided that the influence of the real LSM placing conditions is taken into account to define the values of material constitutive model.

5. Conclusions

In this study it was shown that a strain hardening fibre reinforced cementitious composite material (SHCC) can be optimized to provide appropriate fluidity and deformability in the fresh mix state, and suitable strength, high ductility, and strain hardening behaviour in the hardened state, for the strengthening of ancient masonry elements subjected to flexural loading. This material was obtained by conducting an experimental mix design process, varying the quantities of the admixture and water constituents, and then characterizing the tensile behaviour of the composite through direct tensile tests with un-notched and notched specimens. The optimized mix used to strengthen masonry beam elements subjected to four-point bending has greatly increased the ultimate capacity of the beams, and added ductility to the members through the fibres ability to bridge initial cracks, forming multiple cracks and contributing to the overall deflection hardening behaviour of the beam. The FEM model was able of simulating with good accuracy the results of the experimental tests, as long as the values for the parameters of the constitutive models of the intervening materials are representative of the materials of the masonry beams by taking into account the casting conditions of these beams.

Finally, the SHCC strengthening layer can be seen as an improvement over strengthening with a steel fibre reinforced self-compacting concrete layer, since the SHCC provides comparable strength, but larger deflection capacity and more diffuse tight crack patterns with a thinner layer. This study provides a strong foundation for further development of SHCC for the flexural strengthening of ancient masonry structures.

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Table Captions:

Table 1: SHCC constituent materials

Table 2: Mortar/composite mix procedure

Table 3: Mortar mixes proportions for composite mix production

Table 4: SHCC mix compositions and mini-slump results

Table 5: Relevant results from direct tensile tests on SHCCs

Table 6: Main results from the four-point bending test with masonry beams strengthened with a layer of SHCC

Table 7: Values of the parameters of the constitutive model adopted for the material nonlinear of masonry beams flexurally strengthened with SHCC layer

Figure Captions:

Figure 1. Effect of mixing 2% of PVA fibres on mortar deformability

Figure 2. a) Direct tensile test setup; b) geometry of dog bone shape specimen (dimensions in “mm” and the specimen has a thickness of 20 mm).

Figure 3. Arrangement of the LVDTs in the dog bone direct tensile tests.

Figure 4. Average results of direct tensile test performed on dog bone specimens at 14-days (two specimens for each group were tested).

Figure 5. Diffuse crack pattern formed in T42-SHCC specimen #3.

Figure 6. Notched tensile SHCC specimen subjected to tensile load: a) notched section detail (dimensions in mm); b) close view of the single crack propagation in the notched plane; c) test setup.

Figure 7. The envelope and the average results for tensile stress versus crack opening displacement (COD) from testing four notched specimens.

Figure 8. Detail of strengthening layers for masonry beams. For B15: d1=15mm and d2=20mm; for B20: d1=20mm and d2=20mm; for T0i: d1=30mm and d2=0mm and for T3i: d1=30 and d2=30mm (i=1 to 3 and all dimensions are in mm).

Figure 9. Four-point beam bending test setup (all dimensions are in mm).

Figure 10. Failure mode of the reference masonry beam

Figure 11. Force versus mid-span deflection curves for the beams strengthened with SHCC.

Figure 12. Multiple cracks on the SHCC strengthening layer of masonry beams loaded in 4-point bending a) B15-1; b) B15-2; c) B20-1; d) B20-2.

Figure 13. Force versus mid-span deflection for beams strengthened with SFRSCC.

Figure 14. Comparison of the average force versus mid-span deflection curves of the masonry beams flexurally strengthened with SHCC (B15_avg and B20_avg) and SFRSCC layer (T3_avg and T0_avg).

Figure 15. Finite element mesh, load and support conditions for B20 (the bricks are enumerated).

Figure 16. Tri-linear stress-strain diagram to simulate the fracture mode I propagation

Figure 17. Experimental (avg.) and numerical (FEM) relationship between force and mid-span deflection for: a) B15; b) B20 beams.

Figure 18. Crack patterns representative of the phases indicated in figure 17 for the beams: a) B15; b) B20.