

# Shear Strengthening of Reinforced Concrete T Beams with Hybrid Composite Plate (HCP)

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**Abstract:** This paper aims to evaluate the effectiveness of Hybrid Composite Plates (HCPs) technique for the shear strengthening of the Reinforced Concrete (RC) T cross section beams. HCP consists of a thin plate of Strain Hardening Cementitious Composite (SHCC) reinforced with Carbon Fiber Reinforced Polymer (CFRP) laminates. Two HCPs with different CFRP laminates percentage ( $\rho_{fv} = 0.08\%$  and  $\rho_{fv} = 0.14\%$ ) were adopted for the shear strengthening of the beams. The HCPs were bonded to substrate in two different ways. In the first case the HCPs were bonded using epoxy adhesive, while in the second case they were bonded using epoxy adhesive and fixed by mechanical anchors. The effectiveness of this technique was limited by the tensile strength of the concrete cover of the strengthened beams. Therefore in the second case, mechanical anchors prevented a premature debonding of the HCPs and a certain concrete confinement was applied in the zone of the beam to be strengthened, resulting in favorable effects in terms of shear strengthening.

Advanced FEM-based numerical simulation was performed by using a constitutive model, whose predictive performance was demonstrated by simulating the experimental tests carried out. After demonstration the good predictive performance of the numerical model, a parametric study was carried out to study the influence of shear reinforcement ratio, as well as the influence of thickness of the HCPs on the beam's load carrying capacity.

**Keywords:** Hybrid Composite Plate; CFRP laminates; Shear strengthening; Mechanical anchors; Numerical simulation; Finite Element Method.

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26

27 **Introduction:**

28 The strengthening or retrofitting of existing reinforced concrete (RC) structures to resist higher or different loading  
29 conditions, to attend for construction deficiencies, or to increase the ductility of their behavior has been accomplished  
30 using different materials and techniques. Bonding of thin steel plates (Abdel-Jaber et al. 2003), shotcrete jacketing  
31 (Tsonos 2010; Soleimani and Banthia 2012), and application of fiber reinforced polymer (FRP) by using externally  
32 bonded reinforcement (EBR) (Khalifa et al. 1998) or near surface mounted (NSM) (De Lorenzis and Nanni 2001;  
33 Rezazadeh et al. 2014) are some techniques that have been used as common retrofitting/strengthening techniques.  
34 Among the aforementioned techniques, application of the FRP material has been extensively studied due to high  
35 strength and stiffness-to-weight ratios. However, surface preparation, premature debonding of the FRP sheets and lack  
36 of protection against vandalism and fire are some disadvantages of this technique. Based on the results reported in De  
37 Lorenzis and Nanni (2001), Rizzo and De Lorenzis (2009), and Dias and Barros (2010), debonding of the FRP in the  
38 EBR technique and debonding with concrete fracture in the NSM technique (Bianco et al. 2009) avoid the full  
39 exploitation of the strengthening potentialities of the FRP materials. This is due to relatively low tensile strength of  
40 concrete cover that limits the bonding force between concrete and FRP (Chaallal et al. 2011). It was also observed  
41 that the effectiveness of the EBR and NSM techniques has been decreased with the increase of the percentage of steel  
42 stirrups (Chaallal et al. 2011; Mofidi and Chaallal 2011).

43 The use of anchor systems can prevent or delay this type of failure mode (Khalifa and Nanni 2000; De Lorenzis and  
44 Nanni 2001; Napoli et al. 2010; Mofidi and Chaallal 2011; Sena-Cruz et al. 2012), however the application of these  
45 systems requires extra time and costs, and can be susceptible to vandalism acts and the detrimental effect of  
46 environmental agents, since in general they are made of metallic materials. Special CFRP laminates, capable of being  
47 fixed with anchor devices without premature local rupture in the anchor zones, have been proposed, but these  
48 composite system are more expensive than current CFRP systems (Coelho et al. 2012; Sena-Cruz et al. 2012).

49 Strain Hardening Cementitious Composite (SHCC) can be support material for the FRP reinforcements in order to  
50 constitute an effective strengthening solution that can be fixed to concrete structures with anchorage systems.  
51 Recently, Hybrid Composite Plates (HCPs) have been used to increase the load carrying capacity, energy dissipation,  
52 hysteretic response (Esmaeeli et al. 2015), and ductility behavior of the RC elements. HCP is a thin layer of SHCC  
53 that is reinforced by CFRP laminates (Baghi et al. 2015), sheets (Anwar et al. 2009; Esmaeeli et al. 2013a). Due to

54 the excellent bond conditions between SHCC plate and CFRP laminates/sheet, these reinforcements provide the  
55 necessary tensile strength capacity to the HCP. Moreover, the high post-cracking tensile deformability and resistance  
56 of SHCC avoid the occurrence of premature fracture failure of this cement composite in the stress transfer process  
57 between these two materials when the HCP is crossed by a shear crack (Baghi et al. 2015). The effectiveness of this  
58 technique is limited by tensile strength of the concrete substrate of the RC beams. Since, for a higher mobilization of  
59 the strengthening potentialities of this technique, the HCPs should be not only bonded to the substrate with an adhesive  
60 but also fixed with mechanical anchors (Esmaeeli et al. 2014). Besides the contribution of the SHCC for the  
61 strengthening efficiency of HCPs, the SHCC also assures some protection to the CFRP laminates and adhesive with  
62 respect to accidental actions, such as vandalism, aggressive environmental conditions, and fire.

63 In this work, the effectiveness of the HCPs with different shear percentage of the CFRP laminates ( $\rho_{fw}$ ) and  
64 mechanical anchors is assessed experimentally. For this purpose, seven T cross section beams were tested, composed  
65 of one control beam, and six strengthened beams in shear with three different techniques: i) NSM-CFRP laminates,  
66 ii) SHCC plates, and iii) HCPs. In the last series, HCPs had different shear percentage of CFRP laminates (  
67  $\rho_{fw} = 0.08\%$  and  $\rho_{fw} = 0.14\%$ ). The CFRP shear strengthening percentage,  $\rho_{fw}$ , is defined by the following equation  
68 (Eq. 1):

$$\rho_{fw} = \frac{2a_f b_f}{b_w s_f \sin \theta_f} \quad (1)$$

69 where  $b_w = 180$  mm is width of the beam's cross section,  $a_f = 1.4$  mm and  $b_f = 10$  mm are the dimensions of the  
70 NSM CFRP laminate cross section,  $s_f$  and  $\theta_f$  represent the spacing and inclination of these laminates, respectively.  
71 The HCPs were bonded to substrate in two different ways. In the first case, the HCPs were bonded using epoxy  
72 adhesive, while in the second case they were bonded using epoxy adhesive and fixed by mechanical anchors. The  
73 experimental program is detailed and the obtained results are presented and discussed.

74 Advanced numerical simulations were carried out by using a multi-directional fixed smeared crack model available  
75 in FEMIX computer program that includes a crack shear softening law to simulate the crack shear stress transfer  
76 degradation with the crack widening (Sena-Cruz et al. 2007). The values of the parameters that define this law were  
77 calibrated by simulating the tested beams and considering the properties obtained in the experimental programs for  
78 the characterization of the relevant properties of the used materials. By using this numerical model, a parametric study

79 was carried out to evaluate the effectiveness of the shear strengthening ratio, as well as the influence of the thickness  
80 of the HCPs on the load carrying capacity of the strengthened beams.

81

## 82 **Research Significance:**

83 An experimental program with seven T cross section RC beams was executed in order to demonstrate the effectiveness  
84 of the Hybrid Composite Plates for the shear strengthening. The new technique aims to overcoming shortcomings of  
85 the EBR and NSM techniques such as: debonding of the FRP, lack of protection (vandalism and fire), and stress  
86 concentration caused by anchorage devices when used for avoiding the premature debonding of the FRP. The  
87 application of the HCPs with mechanical anchors was also effective in terms of residual load carrying capacity after  
88 peak load due to the excellent bond conditions between SHCC and CFRP laminates.

89

## 90 **Experimental Program:**

### 91 *Test Program*

92 Figure 1 presents the T cross section of the tested beams. The reinforcement systems were designed to assure shear  
93 failure mode for all the tested beams. The design methodologies were the same adopted elsewhere (Dias 2008). As  
94 shown in Fig. 1, to localize the shear failure in only one of the shear spans, a three point load configuration of a distinct  
95 length of the beam shear spans was selected. The length of the monitored shear span ( $L_f$ ) was 2.5 times the effective  
96 depth of beam, which is the minimum recommendable value for assuring a negligible arch effect. To avoid shear  
97 failure in the other span a relatively high percentage of steel stirrups ( $\phi 6 @ 75$ ) was applied in this span ( $L_f$ ). The  
98 characteristics of the beams are presented in Table 1. The C-R was a reference beam without any type of shear  
99 reinforcement and strengthening throughout the  $L_f$  shear span. The NSM-3L45 was a beam without steel stirrups in  
100  $L_f$  span and was strengthened according to the NSM technique with 3 inclined CFRP laminates ( $\theta = 45^\circ$ ) in each  
101 lateral face of  $L_f$  span, with a space ( $s_f$ ) of 275 mm. The SP was a beam strengthened with SHCC plates to study  
102 the effectiveness of these plates for the shear strengthening. Each plate had overall dimension and weigh of  
103  $800 \times 300 \times 20$  mm<sup>3</sup> and 8.6 kg, respectively. The SHCC plates were bonded to concrete substrate by using epoxy  
104 adhesive (S&P 220 supplied by clever Reinforcement Company). By applying the SHCC plates to the lateral faces of

105 the beam the weight and width of beams' cross section became, respectively, 5% and 22%, larger than the  
106 corresponding one of the control beam (C-R).

107 As mentioned in the literature, the arrangement consisting of laminates at 45° is the most effective in terms of shear  
108 strengthening of the RC beams (De Lorenzis and Nanni 2001; Dias and Barros 2010). Therefore, this arrangement  
109 was selected in this experimental program. The SP-3L45 and SP-5L45 beams were strengthened by applying the HCPs  
110 in each lateral face of the  $L_f$  span by using epoxy adhesive. As shown in Fig. 2, the HCP is a 20 mm thick SHCC  
111 plate that was reinforced with three or five inclined CFRP laminates, spaced at 275 mm (SP-3L45 series) and 157 mm  
112 (SP-5L45 series).

113 In order to fully explore the shear strengthening potentialities of the HCPs, the HCPs in the SP-3L45-B and SP-5L45-  
114 B beams were not only bonded to lateral faces of their corresponding beams by using epoxy adhesive, but also fixed  
115 to the beam with 12 bolts of 10 mm diameter, according to the configuration presented in Fig. 3. The application of  
116 these mechanical anchors aims to prevent a premature debonding of the HCPs and to introduce a certain concrete  
117 confinement in the zone of the beam to be strengthened.

118

### 119 ***Material Properties***

120 The concrete compressive strength was evaluated at the age when the beams were tested (45 days) by executing direct  
121 compression tests with cylinders of 150 mm diameter and 300 mm height according to EN-206-1 (de Normalisation  
122 2000). The values of tensile properties of the steel bars were obtained from uniaxial tensile tests executed according  
123 to EN10002-1 recommendations (ISO 1990). The tensile property of the CFRP laminates was characterized by  
124 executing uniaxial tensile tests according to the recommendations of ISO 527-5 (European Standard 1997).

125 The SHCC is composed of a cementitious mortar reinforced with 2% in volume of short discrete polyvinyl alcohol  
126 (PVA) fibers of 40  $\mu\text{m}$  diameter and 8 mm length. The average tensile stress at crack initiation and average tensile  
127 strength of the SHCC were 2.5 MPa and 3.4 MPa, respectively. More details on the preparation and testing the SHCC  
128 can be found elsewhere (Esmaeeli et al. 2012; Esmaeeli et al. 2013b). Tables 2 and 3 include the average values  
129 obtained from the experimental programs for the assessment of the relevant properties of concrete, CFRP laminates,  
130 SHCC, and steel bars.

131

### 132 ***Strengthening Techniques***

133 Three different strengthening techniques were used in this study: i) NSM CFRP laminates, ii) SHCC plates, and iii)  
134 HCPs. The CFRP laminates were applied to the NSM-3L45 beam according to the NSM technique described in Dias  
135 and Barros (2010). The slits opened on the lateral faces of the RC beam for the installation of the CFRP laminates had  
136 a width and depth of about 5 mm and 15 mm, respectively.

137 For preparing the HCP, the CFRP laminates were applied into the SHCC plates following a procedure similar to the  
138 ones adopted for the RC beam. However, in this case the slits on the SHCC plates had a width and depth of about 4  
139 mm and 11 mm, respectively. The HCPs were applied to their corresponding beams 7 days after the application of the  
140 CFRP laminates in order to guarantee a proper curing of the adhesive.

141 The SHCC/HCPs were applied to the lateral faces of the concrete beams by the following procedures:

142 1) A 1-2 mm roughness with sandblast was executed in the concrete substrate to improve the bond conditions between  
143 SHCC/HCPs and concrete beams;

144 2) In SP-3L45-B and SP-5L45-B beams, twelve holes were drilled with a diameter of 12 mm (Fig. 3) based on the  
145 suggestion of Li (1998) and Esmaeeli et al. (2014);

146 3) An epoxy adhesive (S&P220) layer of a thickness of about 1 mm was homogeneously applied in the surfaces of the  
147 concrete beam and of the SHCC/HCPs that will be in contact;

148 4) In SP-3L45 and SP-5L45 beams mechanical clamps were used to maintain the SHCC/HCPs pressed against the  
149 lateral surfaces of the beam;

150 5) In the SP-3L45-B and SP-5L45-B beams the HCPs were fixed to the concrete substrate of these beams with 12  
151 bolts and nuts (Fig. 3), by applying a torque of 20 N.m in the nuts on both sides of the beams. More information about  
152 these strengthening techniques can be found in Baghi (2015).

153 The C-R beam exhibited an incipient shear failure at support cross section due to a deficient execution of the anchorage  
154 length of the longitudinal reinforcement. Hence, to improve the anchorage conditions of the longitudinal reinforcement  
155 of the other beams, and avoid concrete spalling at the beam support section, a strengthening system based on the use  
156 of longitudinal NSM CFRP laminates of  $1.4 \times 20 \text{ mm}^2$  cross section and with a total length of 400 mm was applied on  
157 the bottom face of the beams, as illustrated in Fig. 4a.

158

159

160 *Test Setup and Monitoring System*

161 The load was applied by using a servo closed loop control equipment, taking the signal read in the displacement  
 162 transducer (LVDT) of the servo-actuator to control the test at a deflection rate of 0.01 mm/s. The deflections of the  
 163 beams at loaded section and at mid-span were measured by two LVDTs that were supported on an aluminum bar fixed  
 164 at the alignments of the supports of the beams (Fig. 4a). With the purpose of obtaining the strain variation in the  
 165 laminates, strain gauges were bonded to the CFRP laminates according to the arrangement represented in Fig. 4b.

166

### 167 **Results of the tested beams**

168 The relationships between force and deflection of the tested beams are presented in Fig. 5a. The SP-3L45, SP-3L45-  
 169 B, and SP-5L45-B beams presented higher load carrying capacity compared to the other beams, which shows the  
 170 effectiveness of the HCPs for the shear strengthening. The loaded section deflection at peak load of the SP-5L45-B  
 171 beam was around 1.1 and 1.3 times higher than the corresponding deflection of the NSM-3L45 and SP beams,  
 172 respectively. From the obtained results the  $\Delta F_{\max} / F_{\max}^{SP} = (F_{\max} - F_{\max}^{SP}) / F_{\max}^{SP}$  ratio was evaluated, and the values are  
 173 indicated in Table 4, where  $F_{\max}^{SP}$  and  $F_{\max}$  are the maximum load capacity of the beam strengthened with SHCC plates  
 174 and of the other shear strengthened beams, respectively. For deflections higher than the one corresponding to the  
 175 formation of the first shear crack in the NSM-3L45 (1.8 mm) and SP (2.5 mm) beams, it was calculated the  
 176  $\Delta F^{NSM-3L45} / F^{NSM-3L45}$  and  $\Delta F^{SP} / F^{SP}$  ratios, respectively, where  $\Delta F^{NSM-3L45}$  and  $\Delta F^{SP}$  is the increase of the load  
 177 provided by HCPs ( $\Delta F^{NSM-3L45} = F - F^{NSM-3L45}$ ,  $\Delta F^{SP} = F - F^{SP}$ ), being  $F^{NSM-3L45}$  and  $F^{SP}$  the load capacity of the  
 178 beam strengthened with NSM CFRP laminates and SHCC plates, respectively, and  $F$  is the corresponding load  
 179 capacity (for the same deflection) of the other strengthened beams with HCPs. These ratios were calculated up to  
 180  $3\Delta_u^{SP}$  of the SP beam, where  $\Delta_u^{SP}$  is the deflection corresponding to the maximum load of the SP beam.

181 As shown in Figs. 5b and 5c, the load of the SP-5L45-B and SP-3L45-B beams at deflection of about 15 mm is around  
 182 125% and 100% higher than the load of the NSM-3L45 and SP beams, respectively. These results show the  
 183 effectiveness of the HCPs and also of the mechanical anchors in terms of post peak load carrying and deformability  
 184 capacity. In fact, apart SP-5L45 beam, the post-peak performance of the beams shear strengthened with HCPs was  
 185 much higher than the performance of the NSM-3L45 and SP beams.

186 By comparing the results of the strengthened beams with HCPs (Fig. 5b and Table 3) with those determined in the SP  
 187 beam ( $\Delta F_{\max} / F_{\max}^{SP}$ ) it is verified that the CFRP laminates contributed for the higher shear strengthening effectiveness

188 of HCPs, since the laminates avoided the degeneration of the micro-cracks in the SHCC plates on macro-cracks, which  
189 had also a positive effect in terms of the stiffness preservation of the beam. The following sections provide a detailed  
190 analysis of the results for individual tested beams.

191

#### 192 *C-R beam*

193 The C-R beam had no steel stirrups in the  $L_i$  span and any type of strengthening intervention. At a load of about 100  
194 kN, two cracks became visible. One crack initiated at the support section (splitting crack), and the other one formed  
195 at the center of the shear span (Fig. 6a). By increasing the load, the cracks widened and propagated up to load of 214  
196 kN and failure occurred at the support section of the beam (Fig. 6b). This failure mode was not expected and was  
197 caused by a deficient execution of the anchorage length of the longitudinal reinforcement. More information about  
198 this beam can be found in Baghi et al. (2015).

199

#### 200 *SP beam*

201 The SP beam was strengthened with two  $800 \times 300 \times 20$  mm<sup>3</sup> SHCC plates that were bonded to each lateral face in  $L_i$   
202 span by using epoxy adhesive (Fig. 4a). The first shear crack became visible by spraying oil (WD-40) on the surface  
203 of the SHCC plate (Fig. 6c). This shear crack formed at a load level of about 230 kN for a deflection of 2.6 mm. In  
204 this stage, the load was maintained almost constant up to deflection of around 3.5 mm, with the widening of this shear  
205 crack and the formation and propagation of some new cracks near the major shear crack. After that, the load started  
206 increasing due to the propagation of the shear failure crack through the flange of the beam towards the loaded area,  
207 and collapse occurred at a load level of about 255 kN and a deflection of 5.0 mm (Fig. 6d). This beam presented a  
208 brittle behavior, with an abrupt load decay at the post peak. As indicated in the previous section, the reinforcement  
209 mechanisms of fibers that were used in the SHCC (40  $\mu$ m diameter and 8 mm length) were not able to absorb in a  
210 stable way the huge amount of energy released in the formation process of critical shear cracks, which justifies the  
211 brittle behavior of this beam in the post peak stage.

212

213

#### 214 *NSM-3L45 beam*



215 The NSM-3L45 beam was strengthened with three inclined (45°) CFRP laminates in each lateral face of monitored  
216 shear span, spaced at 275 mm (Fig. 4). The CFRP shear strengthening percentage of this beam was  $\rho_{fw} = 0.08\%$ . The  
217 first shear crack became visible at around 350 mm from the support section (between laminates number 1 and 2, ellipse  
218 in Fig. 6e), at a load level of about 173 kN.

219 As reported in Bianco et al. (2011), the failure mode of a NSM CFRP laminate subjected to an imposed end slip can  
220 be categorized into four groups: a) debonding, b) tensile rupture of laminate, c) concrete semi-pyramid tensile fracture,  
221 and d) a mixed shallow semi-pyramid plus debonding failure mode (Fig. 7). These modes of failure are dependent on  
222 the relative mechanical and geometric properties of the materials involved. When principal tensile stresses transferred  
223 to the surrounding concrete attain its tensile strength, concrete fractures along a surface, envelope of the compression  
224 isostatics, whose shape can be assumed as a semi-cone (Bianco et al. 2010) or a semi-pyramid (Bianco et al. 2014).  
225 As shown in Fig. 6e by a circle, by increasing the load, some cracks formed around the laminate number 2, and the  
226 aforementioned mixed failure mode occurred in this laminate. Due to the quite short bond transfer length of the other  
227 two laminates, they did marginal contribution for the ultimate shear capacity of this beam. The laminate number 2  
228 failed at a load and a deflection of about 275 kN and 4.1 mm, respectively, and the load was decreased of about 7%.  
229 After that, the load started increasing due to propagation of the shear crack through the flange of the beam towards  
230 the load area. An ultimate load of 291 kN was achieved at a deflection of 5.2 mm. As shown in Fig 5a, after an abrupt  
231 load decay, the load was stabilized at a load level of about 100 kN (32% of maximum load), which almost corresponds  
232 to the shear resistance assured by the longitudinal bars due to dowel effect, obtained according to the CEB-FIP MC  
233 (2010).

234 Due to the crack pattern (Fig. 6f), the highest longitudinal strain in the CFRP laminates was recorded in the SG2 (Fig.  
235 4b) positioned almost coinciding with the shear failure crack, and was approximately 1.04%, which corresponds to  
236 63% of the ultimate strain of the CFRP laminate. Figure 8 represents the relationship between applied load and strain  
237 in the SGs where the maximum CFRP laminate strain was registered in the strengthened beams. Up to the formation  
238 of the shear crack, the maximum strain increased almost linearly with the applied load, but did not exceed the strain  
239 value of 0.01%, demonstrating that these CFRP laminates had marginal shear strengthening contribution during this  
240 stage, as expected. However, at the formation of the shear failure crack an abrupt increase of strain occurred, more  
241 pronouncedly in the NSM-3L45 beam. This strain value and all those herein reported are not necessarily the maximum  
242 values, since they are dependent on the relative position of the SGs with respect to the shear cracks.

243

244 *SP-3L45 beam*

245 The SP-3L45 beam was strengthened with HCPs bonded to each lateral face in the  $L_i$  span using epoxy adhesive. As  
246 shown in Fig. 2a, the HCPs formed by the SHCC plates reinforced with three inclined CFRP laminates, spaced at 275  
247 mm. The inclined lines in Fig. 8a show the position of the CFRP laminates.

248 The first shear crack formed at a load level of about 220 kN between laminates number 1 and 2, almost in the same  
249 location of the first shear crack in the NSM-3L45 beam. By increasing the load, several micro-cracks formed on the  
250 surface of the HCPs (Fig. 9a). The beam failed at a load of about 367 kN and a deflection of 5.5 mm. The failure of  
251 the beam was governed by the detachment of HCPs (Fig. 9b). As mentioned, the effectiveness of this technique was  
252 limited by the tensile strength of the concrete cover, as shown in Fig. 9b by an ellipse, at failure load, part of concrete  
253 cover was attached to the HCPs and local detachment occurred. After local detachment, the load was stabilized at a  
254 level of 42% of the maximum load (155 kN). This load level was higher than the load of the previous beam (NSM-  
255 3L45), since HCPs had connection in the other parts, Fig. 9c shows the local detachment of HCPs. The HCPs caused  
256 an increase in the load carrying capacity (44%) and its corresponding deflection (10%), when compared to the effect  
257 of the SHCC plates in the SP beam. The highest longitudinal strain in the CFRP laminates was recorded by the SG1  
258 (Fig. 4b), and was approximately 0.41%, which corresponds to 25% of the ultimate strain of the CFRP laminate. The  
259 detachment of the HCPs justifies the relatively low collaboration of the CFRP laminates for the shear strengthening,  
260 demonstrated by the relatively small maximum strain registered (Fig. 8).

261

262 *SP-3L45-B beam*

263 The SP-3L45-B beam was identical to SP-3L45 beam, except that the HCPs were bonded using epoxy adhesive and  
264 fixed by 12 through bolts and nuts. A torque of 20 N.m was applied to tighten the nuts on both sides of the beam.  
265 Figures 3a and 3c show the position of the bolts and the CFRP laminates.

266 The first shear crack was detected at a load level of about 246 kN in the same position of the first crack in the SP-  
267 3L45 beam (between laminates number 1 and 2). By increasing the load, this crack widened and propagated. Figure  
268 9d shows the crack pattern of this beam at failure load. At the load of 363 kN and a deflection of 6.2 mm the major  
269 shear crack was opened, with an abrupt load decay at peak load. After this abrupt load decay, the resisting load was  
270 almost maintained, at a load level of about 52% of maximum load (190 kN) up to the end of the test (a deflection of

271 about 16 mm). The anchors contributed for the higher shear strengthening effectiveness of the HCPs in the post peak  
272 stage of the beam, avoiding the detachment of the HCPs. The deflection at maximum load of this beam was 13% and  
273 24% higher than the deflection at maximum load of SP-3L45 and SP beams, respectively.

274 As shown in Fig. 9d, the laminate number 1 was torn off. However, since the SG3 did not function properly during  
275 the test, the highest longitudinal strain was recorded in the SG1 (Fig. 4b), and was approximately 0.69%, which  
276 corresponds to 42% of the ultimate strain of the CFRP laminate (Fig. 8).

277

#### 278 *SP-5L45 beam*

279 The SP-5L45 beam was strengthened with HCPs reinforced with five CFRP laminates ( $\rho_{fw} = 0.14\%$ ), spaced at 157  
280 mm (Fig. 2b). The first flexural-shear crack formed at a load level of 166 kN. By increasing the load, several micro-  
281 cracks formed on the surface of the HCPs. The crack pattern in this beam presents much more cracks than in the other  
282 two previous beams, which is assumed to be caused by the higher percentage of CFRP laminates. The failure of this  
283 beam was premature debonding of adhesive at the load of 306 kN and a deflection of 5.1 mm. After an abrupt load  
284 decay, the load was stabilized at a load level of about 30% of the maximum load (dowel resistance of the longitudinal  
285 bars). The crack pattern of HCPs of this beam at failure load is shown in Fig. 9e. Due to the premature debonding of  
286 adhesive, no shear failure crack was visible on the surface of the HCPs, which indicates that the NSM CFRP laminates  
287 were not mobilized effectively. The highest tensile strain was recorded by the SG1 (Fig. 4b), and was approximately  
288 0.33%, which corresponds to only 20% of the ultimate strain of the CFRP laminate (Fig. 8).

289

#### 290 *SP-5L45-B beam*

291 In SP-5L45-B beam the HCPs were bonded to the lateral faces of the beam using epoxy adhesive and applying 12  
292 through bolts and nuts (Fig. 3b), like the procedure adopted in the SP-3L45-B beam. The first shear crack was detected  
293 at a load of about 220 kN, intersecting the laminate number 4 (Fig. 9f). The failure of this beam occurred at a load of  
294 364 kN and a deflection of 6.3 mm. The crack pattern of the HCPs presents much more shear and flexural-shear cracks,  
295 whose energy in their formation, as well as the resistance of the HCPs to the propagation of the shear failure crack  
296 have contributed for the significant increase in terms of ductility registered in this beam. In the SP-5L45-B beam, the  
297 reinforcement effectiveness of the CFRP laminates avoided the degeneration of the micro cracks into macro-shear  
298 failure crack on the SHCC, and the mechanical anchors prevented the premature detachment of the HCPs, and the

299 failure was localized at the web-flange zone of the beam (marked with an ellipse in Fig. 9f). Since a strengthening  
300 discontinuity existed in this web-flange transition zone, and considering that no internal stirrups were available to  
301 offer resistance to the propagation of this type failure crack, the beams strengthened with HCPs fixed with adhesive  
302 and anchors could not exceed the maximum load attained by the SP-5L45-B beam, regardless the percentage of the  
303 CFRP laminates and number of the bolts used. However, as it is visible in the post-peak stage of this beam, the load  
304 decay was much smoother, and the residual load carrying capacity of this beam (55% of the maximum load) was much  
305 higher than the one registered in the previous beams, due to the larger fracture surface mobilized in the failure mode  
306 of the SP-5L45-B beam.

307 The maximum longitudinal strain measured in the CFRP laminates (SG3, Fig. 4b) was 1.12% (Fig. 8), which  
308 corresponds to 68% of the ultimate strain of the CFRP laminate, that is higher than the maximum strain recorded in  
309 the NSM-3L45 beam. This result shows the effectiveness of the mechanical anchors to avoid premature detachment  
310 of the HCPs and to assure higher collaboration of the CFRP laminates for the shear strengthening.

311

#### 312 **Energy Evaluation:**

313 Toughness indicator, as a measure of the energy absorption capacity, was obtained for each beam by determining the  
314 area behind the force vs. loaded section deflection curve up to three times  $\Delta_u^{SP}$  of the SP beam, whose values are  
315 indicated in Table 5.

316 The SP-5L45 beam, which failure was governed by debonding of adhesive, shows the minimum toughness amongst  
317 the strengthened beams with HCPs. In the post-peak stage of the SP-3L45-B and SP-5L45-B beams, the load decay  
318 was much smoother (Fig. 5a), and the residual load carrying capacity of these beams was much higher than the one  
319 registered in the corresponding beams without mechanical anchors. The toughness of these beams was about 5% and  
320 68% higher than the one of the corresponding beams without mechanical anchors (SP-3L45 and SP-5L45),  
321 respectively. The toughness of the SP-5L45-B beam was around 10% higher than the one corresponding to the SP-  
322 3L45-B beam. Therefore, by increasing the number of CFRP laminates and using mechanical anchors for applying  
323 the HCPs to the RC beams is an effective strategy for enhancing the energy absorption capacity of the strengthened  
324 beam in its post-peak stage. The mechanical anchors prevented a premature debonding of the HCPs, and applied a  
325 certain concrete confinement in the zone of the beam to be strengthened, resulting in favorable effects in terms of  
326 shear strengthening.

327

328 **Numerical Simulations:**

329 The three dimensional multi-directional fixed smeared crack model described in detailed in Ventura-Gouveia (2011),  
330 implemented in the FEM-based computer program, FEMIX, was used in the numerical simulations carried out in this  
331 work.

332 To simulate the crack initiation and the fracture mode I propagation of plain concrete and SHCC, the tri-linear tension-  
333 softening diagram presented in Fig. 10 was adopted (Sena-Cruz 2004), which is defined by the parameters  $\alpha_i$  and  $\xi_i$   
334 , relating stress with strain at the transitions between the linear segments that compose this diagram. The ultimate  
335 crack strain,  $\varepsilon_{n,u}^{cr}$ , is defined as a function of the parameters  $\alpha_i$  and  $\xi_i$ , the fracture energy,  $G_f^I$ , the tensile strength,  
336  $\sigma_{n,1}^{cr} = f_{ct}$ , and the crack bandwidth,  $l_b$ . The values of the relevant quantities defining of this diagram are indicated in  
337 Tables 6 and 7 for plain concrete and SHCC, respectively. These tables also include the data necessary to define the  
338 shear-softening relationship that simulates the degradation of crack shear stress transfer after crack initiation (Ventura-  
339 Gouveia et al. 2008; Ventura-Gouveia 2011; Barros et al. 2013) presented in Fig. 11.

340 To simulate the fracture mode II modulus, a shear retention factor is used (Eq. 2):

$$D_{i_1}^{cr} = D_{i_2}^{cr} = \frac{\beta}{1-\beta} G_c \quad (2)$$

341 where  $G_c$  is the concrete elastic shear modulus and  $\beta$  is the shear retention factor.  $D_{i_1}^{cr}$  and  $D_{i_2}^{cr}$  represent the  
342 modulus correspondent to the sliding mode stiffness modulus in the  $\hat{t}_1$  direction and the sliding mode stiffness  
343 modulus in the  $\hat{t}_2$  direction, respectively (Fig. 12). The parameter  $\beta$  is defined as a constant value or as a function  
344 of the current crack normal strain,  $\varepsilon_n^{cr}$ , and of the ultimate crack normal strain,  $\varepsilon_{n,u}^{cr}$ , as follows (Eq. 3):

$$\beta = \left(1 - \frac{\varepsilon_n^{cr}}{\varepsilon_{n,u}^{cr}}\right)^{P_1} \quad (3)$$

345 When  $P_1 = 1$  a linear decrease of  $\beta$  with the increase of  $\varepsilon_n^{cr}$  is assumed. Larger values of the exponent  $P_1$  correspond  
346 to a more pronounced decrease of the  $\beta$  parameter. In structures governed by flexural failure modes, this strategy  
347 leads to simulations with good accuracy (Barros et al. 2011). Exceptions occur in structures that fail by the formation  
348 of a critical shear crack. To simulate accurately the deformational response and the crack pattern up to the failure of

349 this type of structures, the adoption of a softening crack shear stress *versus* crack shear strain relationship was adopted  
 350 to model the crack shear transfer in  $\hat{t}_1$  and  $\hat{t}_2$  direction.

351 The adopted crack shear diagram is represented in Fig. 11. The crack shear stress increases linearly until the crack  
 352 shear strength is reached,  $\tau_{t,p}^{cr}$ , (first branch of the shear crack diagram), followed by a decrease in the shear residual  
 353 strength (softening branch). This diagram is defined by Eq. 4:

$$\tau_t^{cr}(\gamma_t^{cr}) = \begin{cases} D_{t,1}^{cr} \gamma_t^{cr} & 0 < \gamma_t^{cr} \leq \gamma_{t,p}^{cr} \\ \tau_{t,p}^{cr} - \frac{\tau_{t,p}^{cr}}{(\gamma_{t,u}^{cr} - \gamma_{t,p}^{cr})} (\gamma_t^{cr} - \gamma_{t,p}^{cr}) & \gamma_{t,p}^{cr} < \gamma_t^{cr} \leq \gamma_{t,u}^{cr} \\ 0 & \gamma_t^{cr} > \gamma_{t,u}^{cr} \end{cases} \quad (4)$$

354 The initial shear fracture modulus,  $D_{t,1}^{cr}$ , is defined by Eq. 2 ( $D_{i_1}^{cr}$  is replaced by  $D_{t,1}^{cr}$ ) by assuming for  $\beta$  a constant  
 355 value in the range  $]0,1[$ . The peak crack shear strain,  $\gamma_{t,p}^{cr}$ , is obtained using the crack shear strength (from the input  
 356 data),  $\tau_{t,p}^{cr}$ , and the crack shear modulus (Eq. 5):

$$\gamma_{t,p}^{cr} = \gamma_{t_2,p}^{cr} = \frac{\tau_{t,p}^{cr}}{D_{t,1}^{cr}} \quad (5)$$

357 The ultimate crack shear strain,  $\gamma_{t,u}^{cr}$ , depends on the crack shear strength,  $\tau_{t,p}^{cr}$ , on the shear fracture energy (mode II  
 358 fracture energy),  $G_{f,s}$ , and on the crack bandwidth,  $l_b$  (Eq. 6):

$$\gamma_{t,u}^{cr} = \gamma_{t_2,u}^{cr} = \frac{2G_{f,s}}{\tau_{t,p}^{cr} l_b} \quad (6)$$

359 In this approach it is assumed that the crack bandwidth, used to assure that the results are independent of the mesh  
 360 refinement (Rots 1988), is the same adopted for the fracture mode I. It is also assumed that the crack shear behavior  
 361 in both  $t_1$  and  $t_2$  directions is simulated by the same constitutive law.

362 The crack mode II modulus of the first linear branch of the diagram is defined by Eq. 2, the second linear softening  
 363 branch is defined by Eq. 7:

$$D_{t_1}^{cr} = D_{t_2}^{cr} = D_{t,2}^{cr} = -\frac{\tau_{t,p}^{cr}}{\gamma_{t,u}^{cr} - \gamma_{t,p}^{cr}} \quad (7)$$

364 and the crack shear modulus of the unloading and reloading branches is obtained from Eq. 8:

$$D_{t_1}^{cr} = D_{t_2}^{cr} = D_{t,3-4}^{cr} = \frac{\tau_{t,\max}^{cr}}{\gamma_{t,\max}^{cr}} \quad (8)$$

365 being  $\gamma_{t,\max}^{cr}$  and  $\tau_{t,\max}^{cr}$  the maximum crack shear strain already attained and the corresponding crack shear stress  
 366 determined from the softening linear branch. Both components are stored to define the unloading/reloading branch  
 367 (see Fig. 11).

368 In free - sliding status ( $|\gamma_t^{cr}| > |\gamma_{t,u}^{cr}|$ ) the crack mode II stiffness modulus,  $D_{t_1}^{cr} = D_{t_2}^{cr} = D_{t,5}^{cr}$ , is null. To avoid numerical  
 369 instabilities in the calculation of the stiffness matrix and in the calculation of the internal forces, when the crack shear  
 370 status is free - sliding, a residual value is assigned to this term. A free - sliding status is assigned to the shear crack  
 371 status when  $\varepsilon_n^{cr} > \varepsilon_{n,u}^{cr}$  (Barros et al. 2013). More details of the remaining variables of this constitutive model can be  
 372 found in Barros et al. (2013) and Ventura-Gouveia (2011).

373 The data for the shear softening diagram of plain concrete was determined by fitting as best as possible the force-  
 374 deflection relationship registered in the control beam tested experimentally (C-R), while for the SHCC this data was  
 375 obtained by simulating the SP beam. For the analysis of the remaining beams of the experimental program the values  
 376 of the constitutive model applied to each intervening material were preserved constant. Figure 13 represents the finite  
 377 element mesh adopted for the control beam (C-R). In the simulations, this finite element mesh was only altered in  
 378 order to take into account the strengthening provided by CFRP laminates (NSM-3L45 beam), the SHCC plate (SP  
 379 beam), and HCPs.

380 Only half of the full size beam was modeled, taking advantage of the symmetry of the beams in order to reduce the  
 381 computational time of the numerical simulations. Serendipity 8 nodes solid elements with  $2 \times 2 \times 2$  Gauss-Legendre  
 382 integration scheme were used for both the concrete and SHCC (three degrees-of-freedom per node). The steel stirrups,  
 383 longitudinal steel bars and CFRP laminates were modeled with 3D embedded cables of 2 nodes (one degree-of-  
 384 freedom per node), by using a 2 Gauss-Legendre integration scheme, and perfect bond to the surrounding medium  
 385 was assumed. The bolts are modeled with 3D two-node truss elements, and the confinement effect locally induced on  
 386 concrete by the torque of anchors was simulated by applying a temperature decrease of  $-25.5^\circ\text{C}$  in these elements,  
 387 evaluated according to the following equations (Eq. 9):

$$F = \frac{\tau}{r} \qquad F = \frac{20}{0.005} = 4000 \text{ N} \qquad (9a)$$

$$\sigma = \frac{F}{A} \qquad \sigma = \frac{4000}{78.5} = 51 \text{ MPa} \qquad (9b)$$

$$\varepsilon = \frac{\sigma}{E} = T \cdot \alpha \qquad \varepsilon = \frac{51}{200000} = T \times 10^{-5} \qquad (9c)$$

388 where  $\tau$  is torque (N.m), and  $r$ ,  $A$ , and  $E$  are the radius, cross sectional area, and elasticity modulus of the bolt (5 mm,  
389 78.5 mm<sup>2</sup>, 200 GPa), respectively. In these equations  $T$  and  $\alpha$  are the temperature variation and the coefficient of  
390 thermal expansion, respectively. The relation between applied torque and axial tension force fastener is presented in  
391 Fig. 14. More information about this formulation can be found in Baghi (2015).

392 An elasto-perfectly plastic model was adopted to simulate the tension and compression behavior of the steel  
393 reinforcements, whose fundamental information is indicated in Table 3. For modeling the NSM CFRP laminates, a  
394 linear elastic stress-strain relationship was adopted. The assumption of perfect bond between substrate and SHCC  
395 plates was assumed. Newton-Raphson Standard method was applied, and energy convergence tolerance of  $10^{-4}$  was  
396 taken.

397 The experimental and the numerical relationships between the applied load and the deflection at the loaded section for  
398 the tested beams are compared in Fig. 15. The crack pattern of these beams at the end of the analysis is represented in  
399 Fig. 16. During the last not converged loading step, several cracks in the critical shear zone open completely (fracture  
400 energy is completely exhausted – red color), and since this abrupt crack propagation is restricted to the critical shear  
401 zone (which resembles what happened in the experimental tests), convergence was no more possible to be assured.

402 Figures 15 and 16 show that the numerical model is capable of predicting with high accuracy the load vs. deformational  
403 response of the beams, and to capture with a good precision the localization and profile of the failure cracks. These  
404 results confirm the capability of the developed model to simulate to behavior of RC beams failing in shear. For the  
405 beams strengthened with SHCC/HCP, the crack pattern is represented for the lateral surface of the concrete substrate,  
406 as well as for the SHCC/HCP. The higher load predicted for the SP-5L45 indicates that the assumed perfect bond  
407 conditions between HCPs and concrete substrate was not assured in this beam, pointing out that some deficiency was  
408 occurred in the bonding process, as already reported. The crack pattern of the beams strengthened with SHCC/HCP  
409 shows the tendency of the failure crack to propagate at the web-flange interface due to the discontinuity of beam's  
410 cross section stiffness and shear strengthening contribution of the SHCC/HCP.

411 Figure 17 also shows that the numerical simulations fit with good accuracy the strains measured in the NSM laminates,  
412 which means that the assumption of perfect bond between NSM laminates and surrounding SHCC is acceptable, at  
413 least in the design point of view for the serviceability and ultimate limit states.



414 Based on results, it can be concluded that the implementation of the shear softening diagram in the multi-directional  
415 fixed smeared crack model available in the FEMIX computer program is capable of predicting with high accuracy the  
416 deformational behavior, load carrying capacity, and crack patterns of structures failing in shear.

417 The applicability of this type of numerical simulations in real strengthening interventions requires the assessment of  
418 the concrete compressive strength from non-destructive (extraction of core samples) or non-invasive techniques (e.g.  
419 ultra-sonic). From the concrete compressive strength and using the recommendations of design codes, like CEB-FIP  
420 Model Code, the mode I fracture parameters can be determined. However, the evaluation of the mode II fracture  
421 parameters, like the ones required for defining the crack shear softening diagram, is still a challenging task.  
422 Nonetheless, based on experience of the authors, shear retention factor ( $\beta$ ), crack shear strength ( $\tau_{t,p}^{cr}$ ), and fracture  
423 energy mode II ( $G_{f,s}$ ) for the normal concrete are between 0.4 to 0.6, 0.9 to 1.1, and 0.03 to 0.06, respectively.  
424 Parametric studies have also been carried out by Barros et al. (2013) and Breveglieri (2015) to investigate the influence  
425 of these parameters on the load carrying capacity and deflection performance of shear strengthened RC beams. Finally,  
426 there is also a database for the shear strengthening of RC beams (<http://dabasum.civil.uminho.pt/>) that can be useful  
427 for deriving the model parameters by simulating by inverse analysis the experimental tests included in this database.

428

#### 429 **Parametric Study:**

430 The computer program, whose good predictive performance for the simulation of the behavior of the type of structures  
431 under consideration was confirmed in the previous section, was adopted to execute a parametric study to evaluate  
432 effectiveness of shear reinforcement ratio and also the thickness of the HCP on the load carrying capacity and failure  
433 mode of the beams strengthened with HCPs. The arrangement of the steel reinforcements, the material properties of  
434 concrete and SHCC, the support and load conditions, and the finite element mesh were the same ones adopted in the  
435 numerical simulations of the previous section.

436

#### 437 ***Influence of Shear Reinforcement Ratio***

438 Based on the results of the experimental program, the failure of the SP-5L45-B beam was localized at the web-flange  
439 zone, which avoided to exploit the full strengthening potential of these HCPs. Thus, to access the shear strengthening  
440 effectiveness of these HCPs in situations where a certain percentage of steel stirrups exists, two values for the  
441 reinforcement ratio of existing steel stirrups were considered: 2  $\phi 6 @ 300$  mm ( $\rho_{sw} = 0.10\%$ ) and 4  $\phi 6 @ 150$  mm ( $\rho_{sw}$   
442  $= 0.20\%$ ). The relationship between load and deflection at loaded section for the simulated beams are presented in Fig.  
443 18. As mentioned in the introduction, the effectiveness of the EBR and NSM techniques has been decreased by

444 increasing the percentage of steel stirrups. However, based on the results of the numerical simulation, in the HCPs  
445 technique the strengthening effectiveness has increased by increasing the percentage of existing steel stirrups. In these  
446 beams a critical diagonal shear crack was formed and beam failed at peak load without the occurrence of yield  
447 initiation of the longitudinal reinforcement.

448

#### 449 *Influence of thickness of SHCC panel*

450 In this case, the influence of thickness of the HCPs on the load-deflection and stiffness was investigated. For this  
451 purpose, two other thicknesses of HCPs were assumed: 15 mm and 40 mm, the first one is lower and the last one is  
452 higher than the one corresponding to the thickness of the HCPs of the SP-5L45-B beam. To avoid the localization of  
453 the failure at web-flange zone, 2 steel stirrups  $\phi 6 @ 300$  ( $\rho_{sw} = 0.10\%$ ) were added in the monitored shear span. The  
454 obtained results, depicted in Fig. 19, show that the thickness of the HCPs has a relatively small influence on the  
455 stiffness of the strengthened beams. However, by increasing the thickness of the HCPs the load carrying capacity and  
456 ultimate deflection have increased significantly. All simulated beams failed in shear.

457

#### 458 **Conclusions:**

459 The effectiveness of Hybrid Composite Plates (HCPs) for the shear strengthening of reinforced concrete (RC) beams  
460 was investigated by carrying out an experimental program, which was complemented with FEM-based advanced  
461 numerical simulations. From the obtained results, the following conclusion can be drawn:

- 462 • The HCPs increased the shear capacity of the beams around 43% compared to the beam strengthened with  
463 SHCC plates.
- 464 • The strain hardening cement composites (SHCC) surrounding the carbon fiber reinforced polymer (CFRP)  
465 laminates has offered effective resistance to the degeneration of micro-cracks on macro-cracks.
- 466 • The effectiveness level of the HCP technique was limited by the tensile strength of the concrete substrate of  
467 the RC beams and maximum tensile strain in the CFRP laminates did not exceed 25% of the ultimate strain  
468 of these laminates. Therefore mechanical anchors were used to prevent this premature detachment.
- 469 • The load carrying capacity of the strengthened beams with HCP technique was limited by the shear  
470 strengthening discontinuity at the web-flange of the beam, since in these strengthened beams the failure  
471 crack propagated through this zone.

- 472       • The HCPs were capable of increasing not only the load carrying and deflection capacity, but also the post-  
473       peak resisting load, with favorable effects in terms of energy absorption capacity.
- 474       • The shear crack softening diagram available in the multi-directional fixed smeared crack model implemented  
475       in the FEMIX computer program, allowed to predict with high accuracy the load carrying capacity, crack  
476       patterns, strain in the CFRP laminates, and failure modes of the tested beams.
- 477       • The results of the parametric study show that by increasing the reinforcement ratio of existing steel stirrups,  
478       the strengthening effectiveness of the HCPs has increased, which is an extremely important attribute of this  
479       technique, since the opposite occurs with EBR and NSM techniques. The load carrying capacity and ultimate  
480       deflection of the beams have increased with the thickness of the HCPs.

481

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489   of the beams.

490

491

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585

## LIST OF Figure CAPTIONS

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- Fig. 3- Position of the CFRP laminates and mechanical anchors a) SP-3L45-B, b) SP-5L45-B, c) position of the bolts inside of the RC beams (dimensions in mm)
- Fig. 4- Monitoring system- position of the: a) LVDTs; and b) strain gages in CFRP laminates (dimensions in mm)
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- 16 - Crack patterns of the beams (in red color: crack completely open; in black color: crack in the opening process)
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- Fig. 18 – Influence of the reinforcement ratio of existing steel stirrups on the load carrying capacity of HCP strengthened beams
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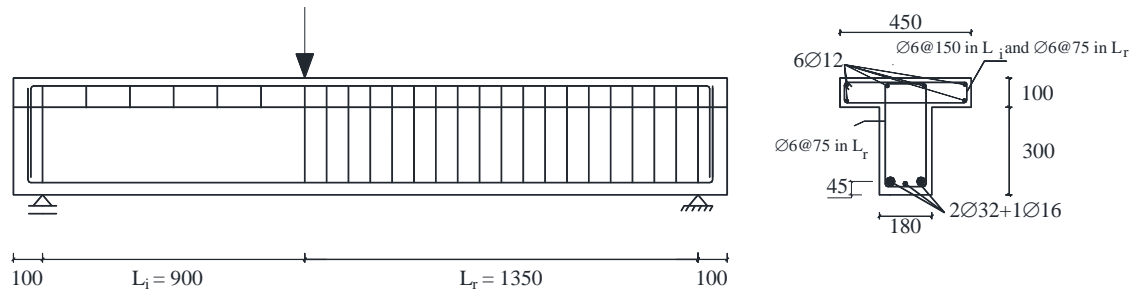
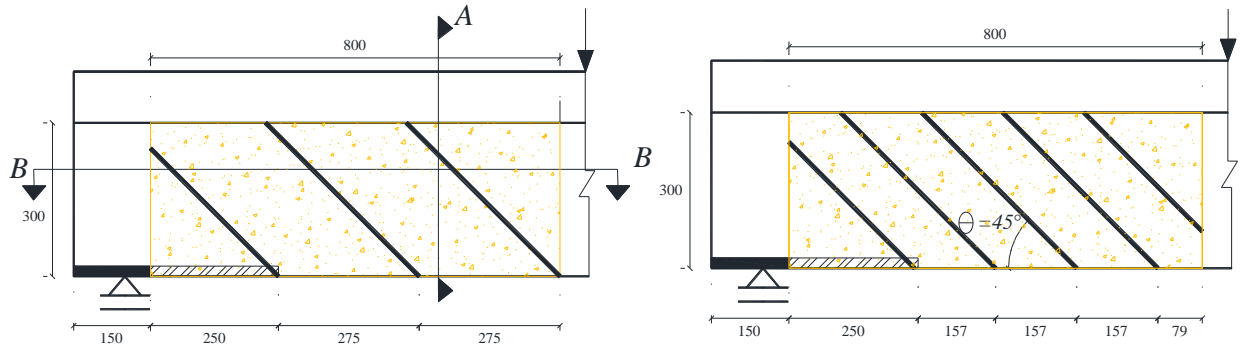
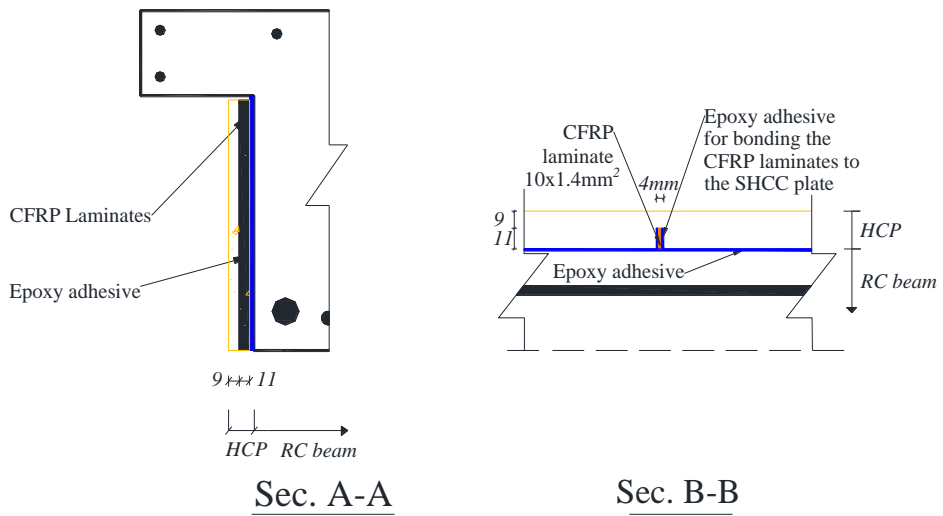


Fig. 1 - Geometry and reinforcement arrangement of the concrete beams (dimensions in mm)



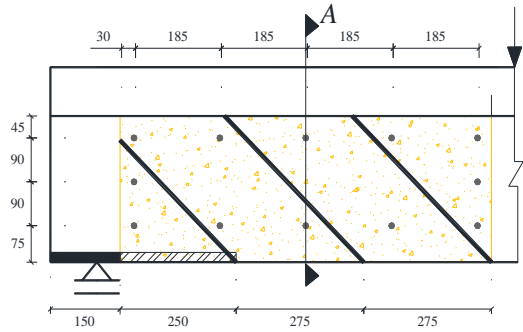
a) SHCC plate with three inclined CFRP laminates ( $\rho_{fw} = 0.08\%$ )

b) SHCC plate with five inclined CFRP laminates ( $\rho_{fw} = 0.14\%$ )

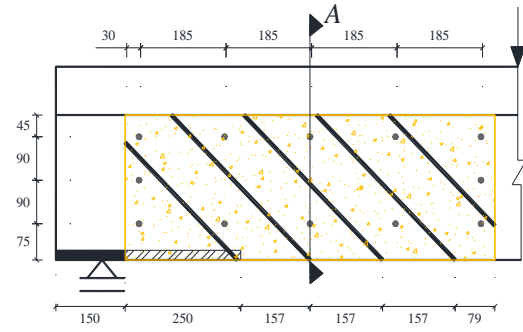


c) A section view of the strengthened beam with HCPs

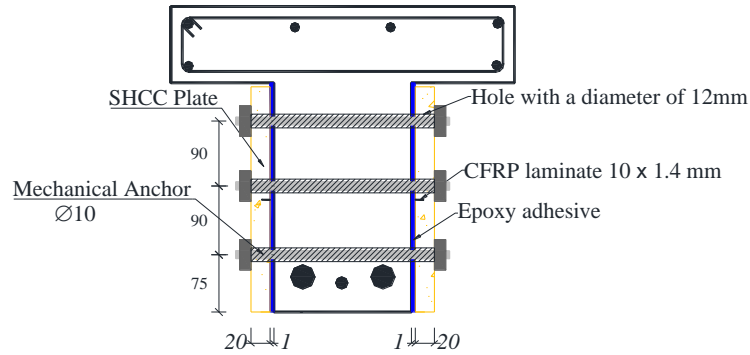
Fig. 2 – Position of the CFRP laminates inside of the SHCC plate a) SP-3L45 beam, b) SP-5L45 beam (dimensions in mm)



a) SHCC plate with three inclined CFRP laminates ( $\rho_{fw} = 0.08\%$ ) and 12 anchors



b) SHCC plate with five inclined CFRP laminates ( $\rho_{fw} = 0.14\%$ ) and 12 anchors



Sec. A-A

c) position of the bolts inside of the RC beams

Fig. 3- Position of the CFRP laminates and mechanical anchors a) SP-3L45-B, b) SP-5L45-B, c) position of the bolts inside of the RC beams (dimensions in mm)

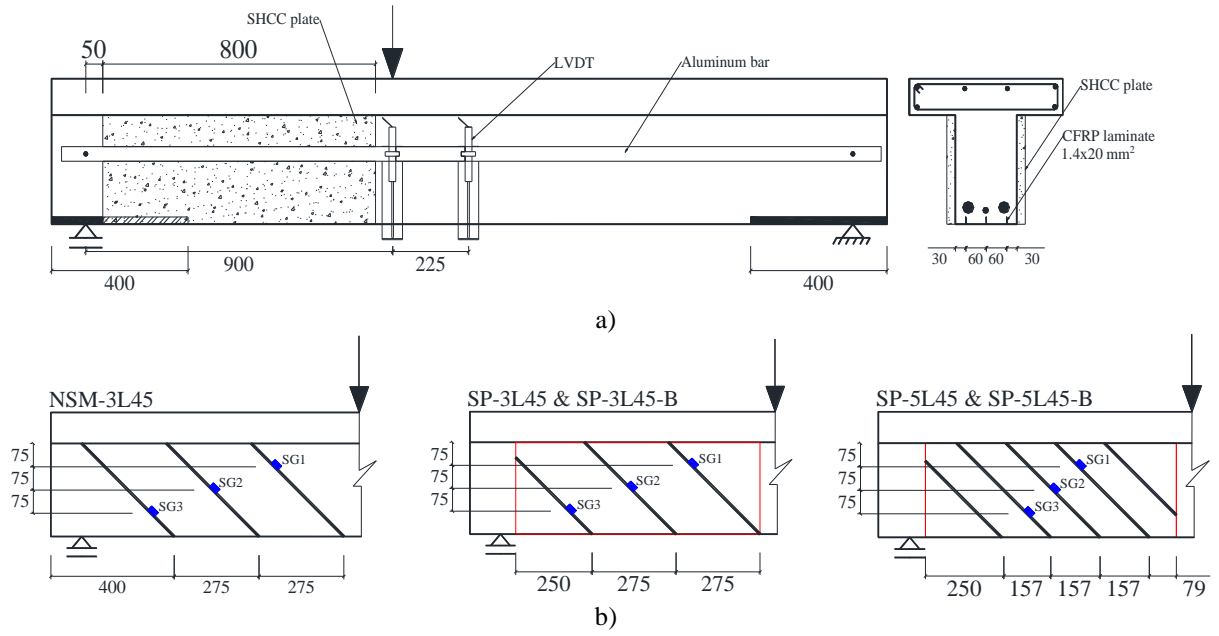
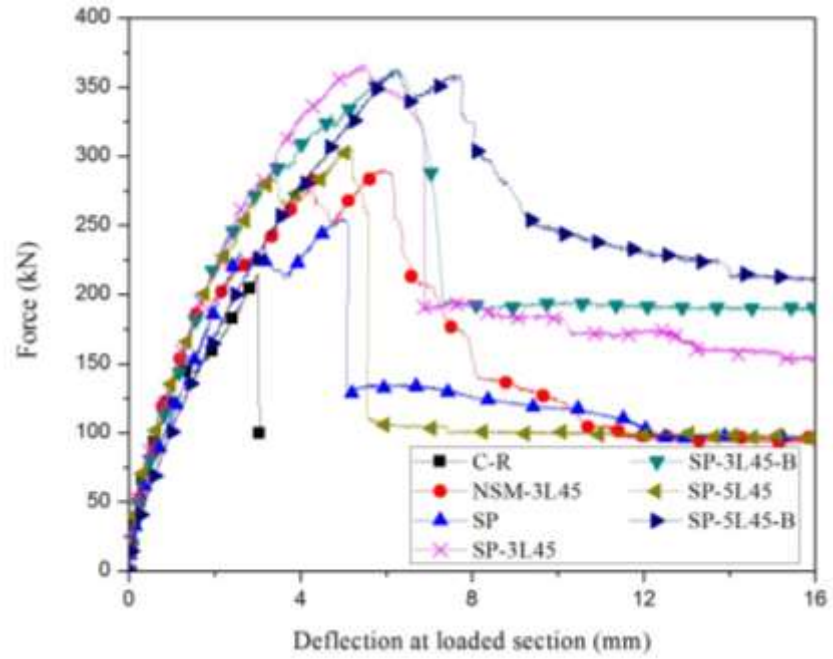
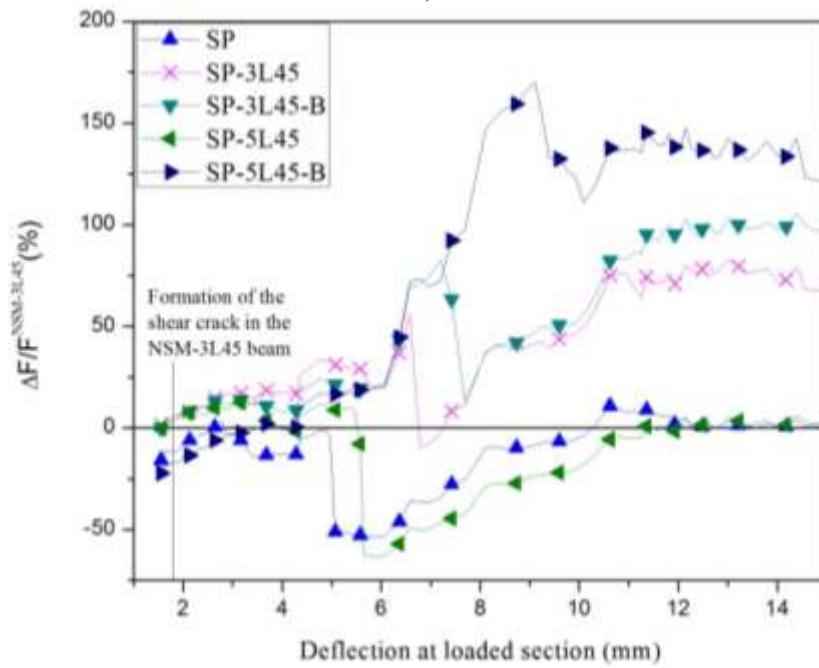


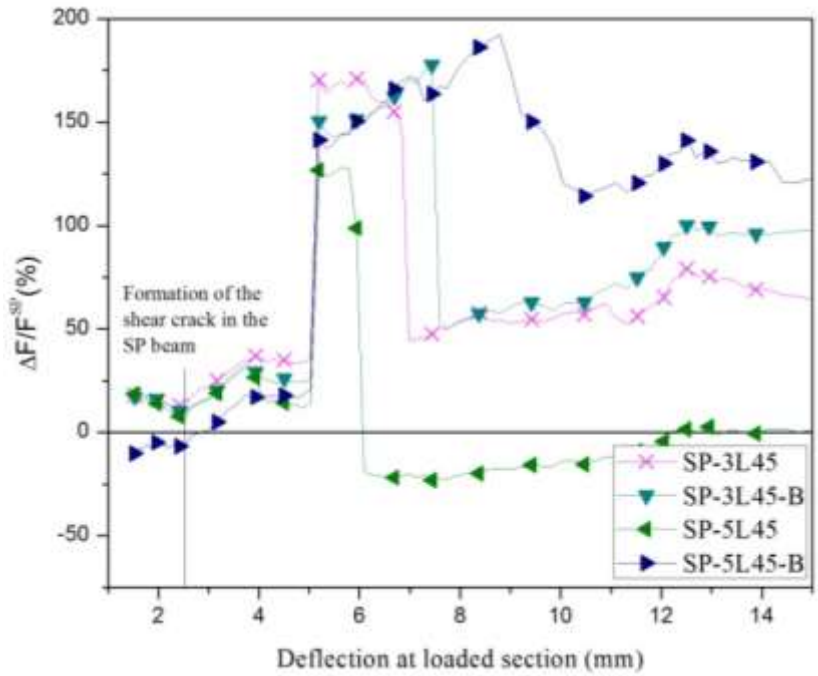
Fig. 4- Monitoring system- position of the: a) LVDTs; and b) strain gages in CFRP laminates (dimensions in mm)



a)

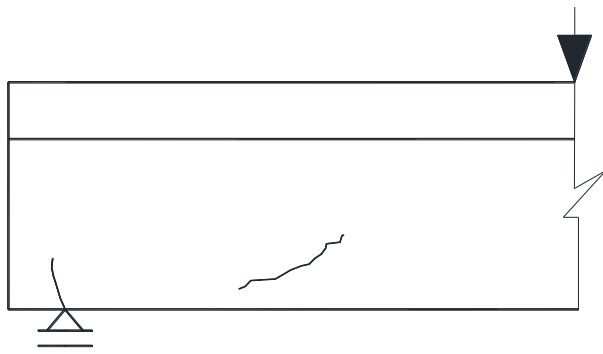


b)

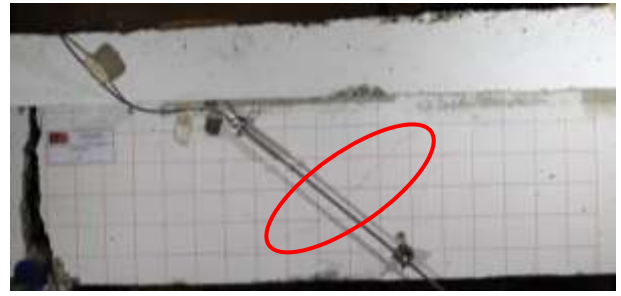


c)

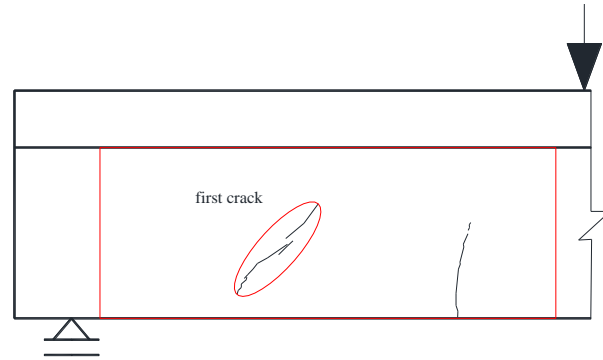
Fig. 5- a) Force vs. deflection at the loaded-section, b)  $\Delta F / F^{NSM-3L45}$  vs. deflection at the loaded-section for the beams strengthened with SHCC/HCPs, and c)  $\Delta F / F^{SP}$  vs. deflection at the loaded-section for the beams strengthened with HCPs



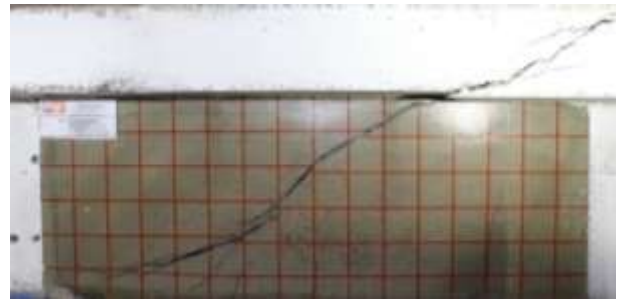
a) Crack pattern at the load level of 100 kN of the C-R beam



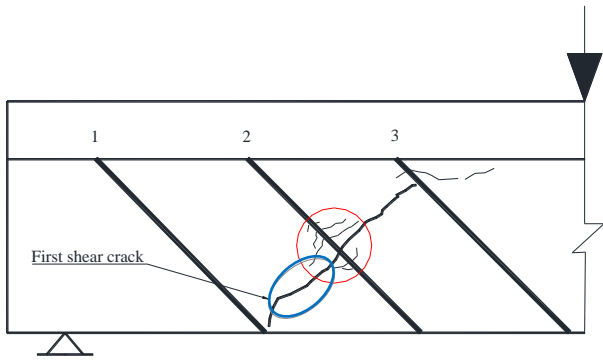
b) Crack pattern at failure load of C-R beam



c) Crack pattern at the load level of 230 kN of the SP beam



d) Final crack pattern of the SP beam

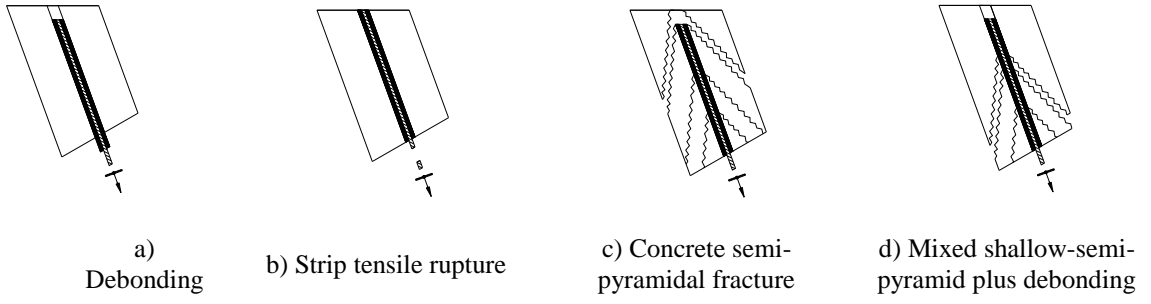


e) Crack pattern at the load level of 225 kN of the NSM-3L45 beam



f) Final crack pattern of the NSM-3L45 beam

Fig. 6 - Crack patterns and failure modes of the C-R, NSM-3L45, and SP beams



*Fig. 7 - The mode of failure of an NSM CFRP laminate subjected to an imposed end slip*



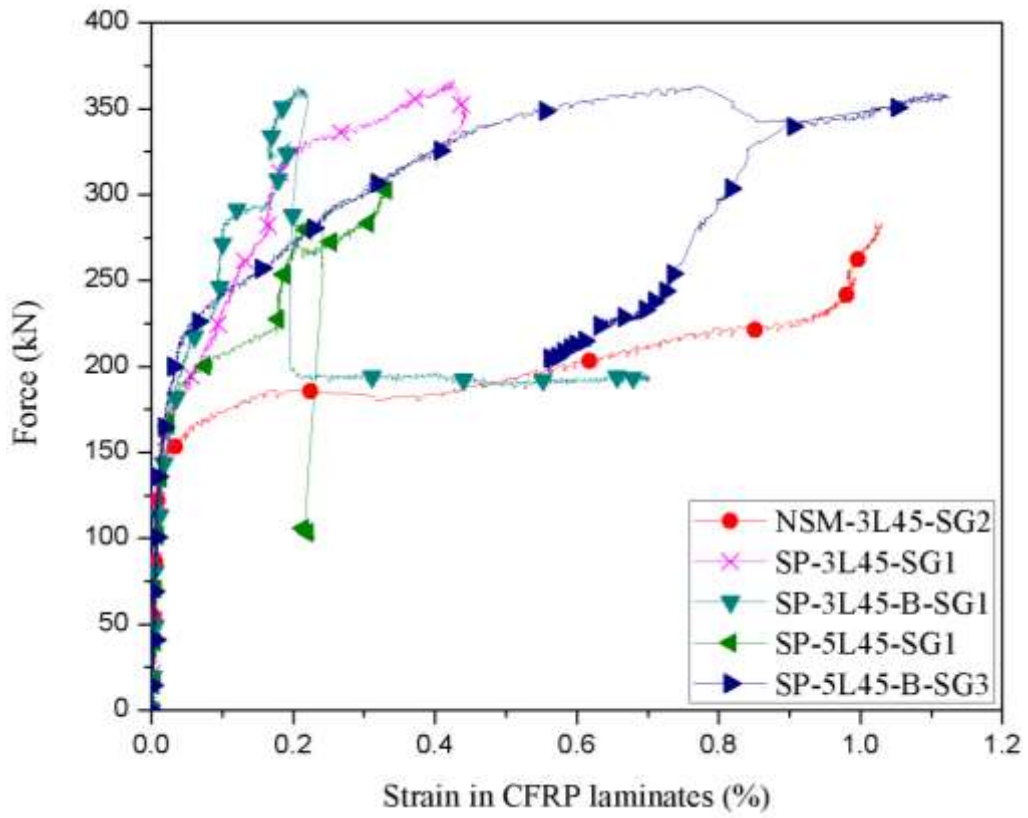
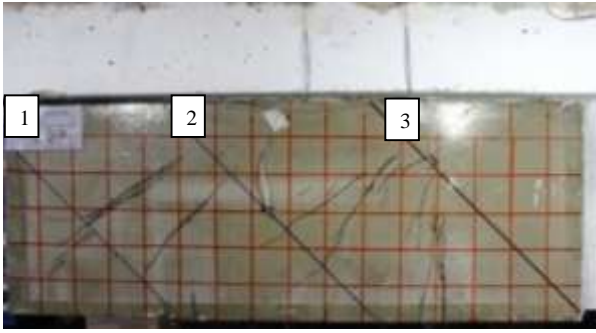


Fig. 8 - Force vs. strain in monitored laminates in SGs where the maximum strains were registered



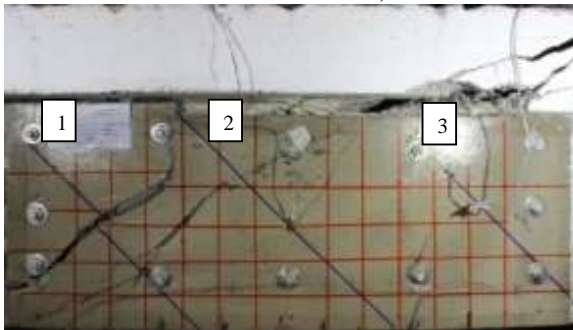
a) Crack pattern at the load level of 350kN of the SP-3L45 beam



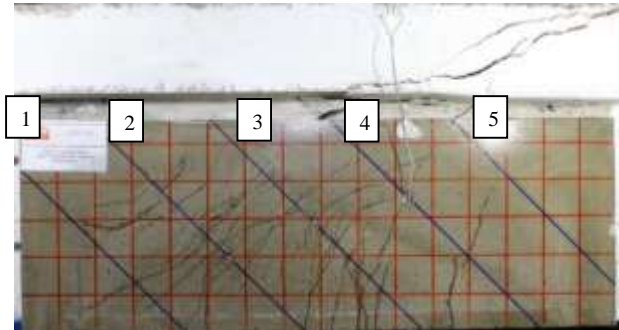
b) Final crack pattern of the SP-3L45 beam when HCP was peeled off after the test



c) Local detachment of HCPs at failure load



d) Final crack pattern of the SP-3L45-B beam



e) Final crack pattern of the SP-5L45 beam



f) Final crack pattern of the SP-5L45-B beam

Fig. 9 - Crack patterns and failure modes of the strengthened beams with HCPs

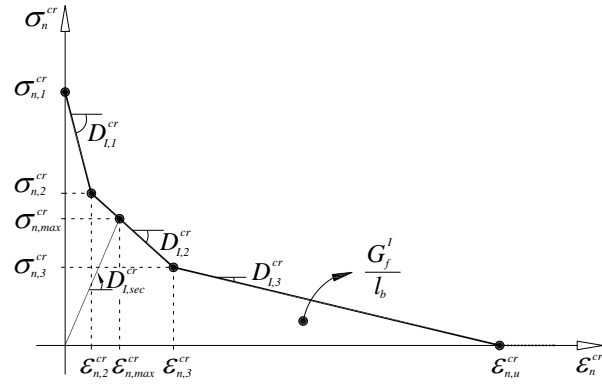


Fig. 10 - Trilinear stress-strain diagram to simulate the fracture mode I crack propagation

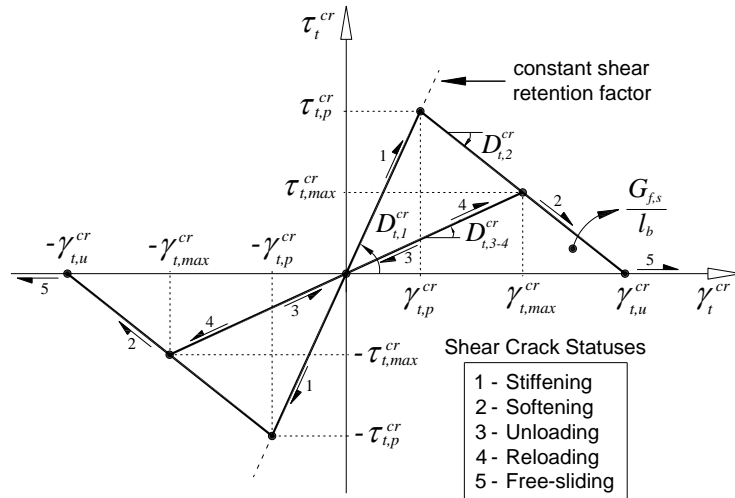


Fig. 11 - Diagrams to simulate the relationship between the crack shear stress and crack shear strain component, and possible shear crack statuses

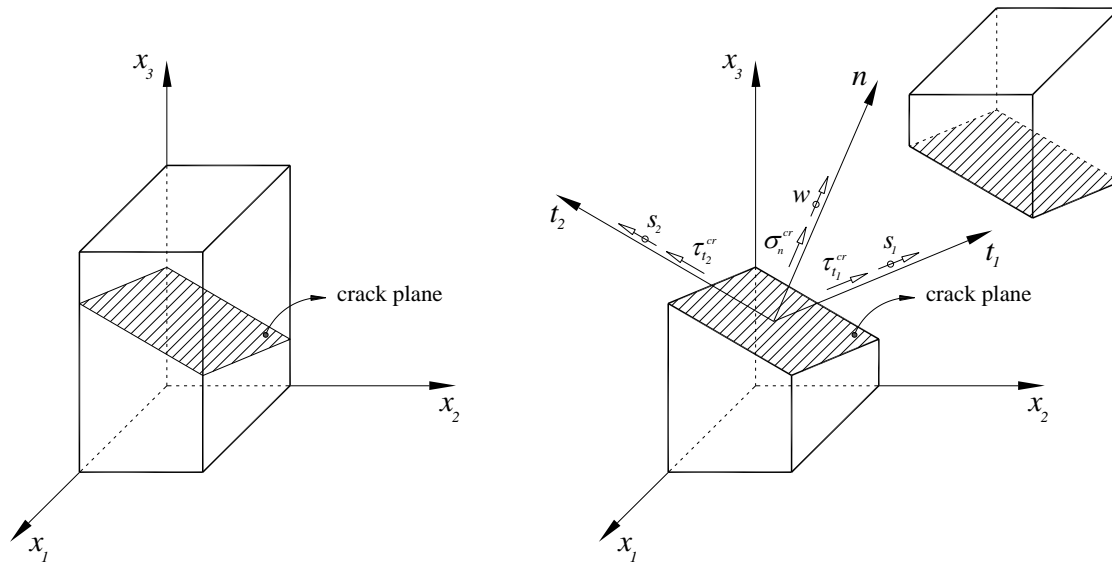


Fig. 12 - Crack stress components, displacements and local coordinate system of the crack.

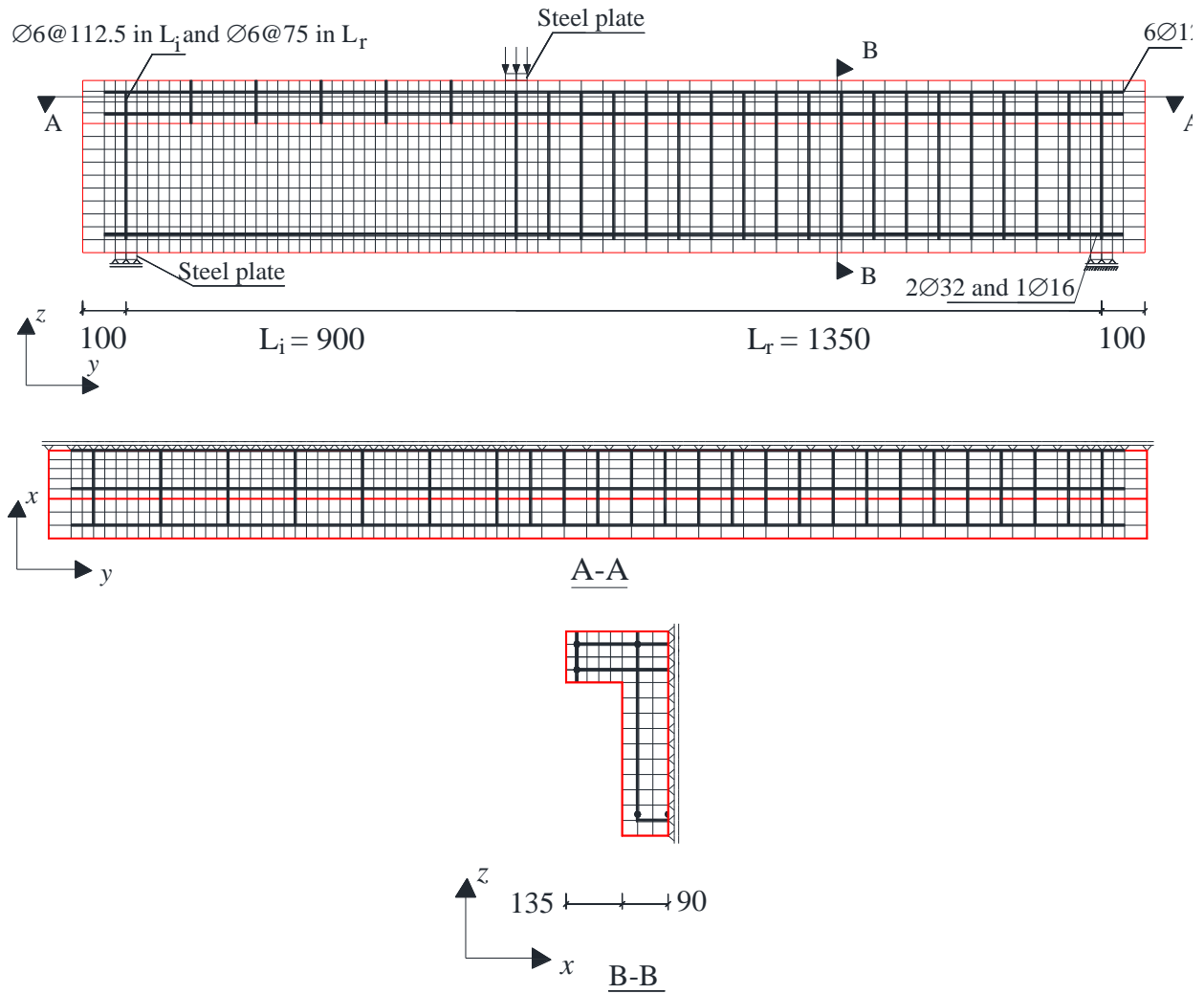
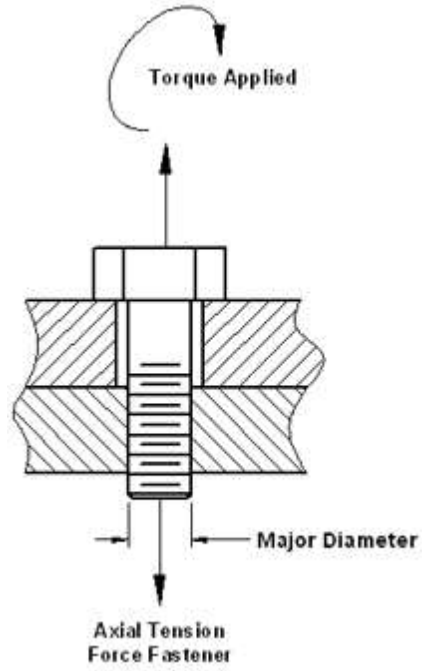
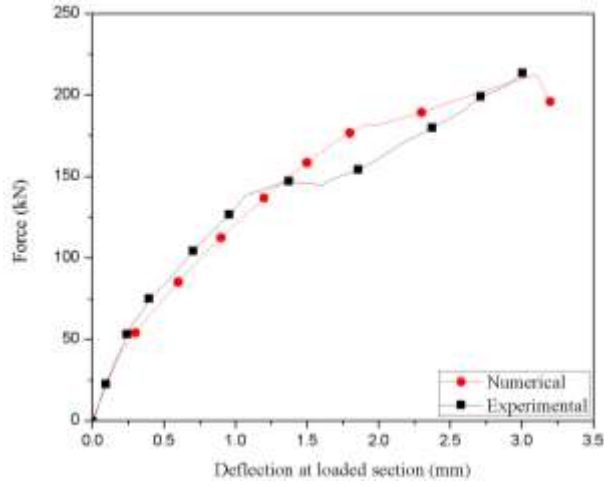


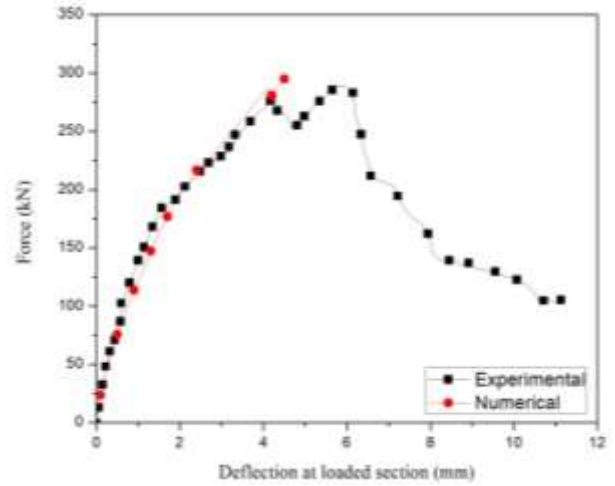
Fig. 13 - Finite element mesh of the C-R beam (dimensions in mm)



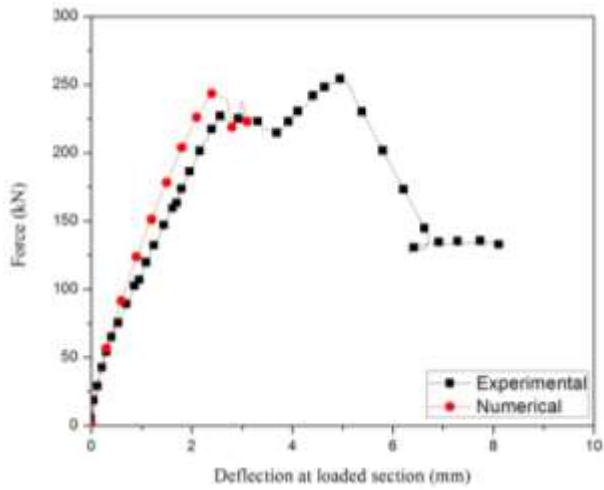
*Fig. 14 – The relation between applied torque to axial tension force fastener*



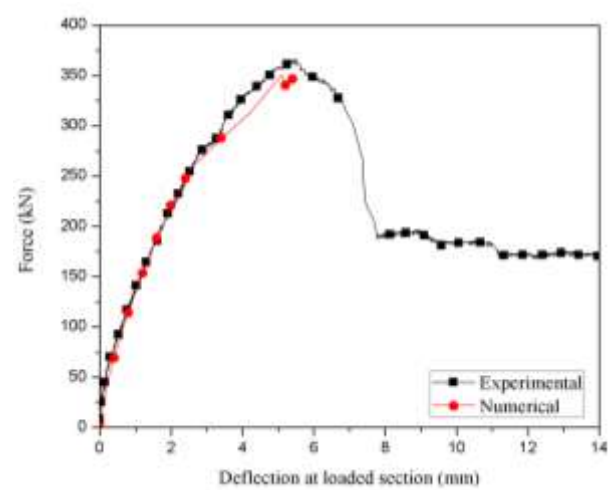
C-R



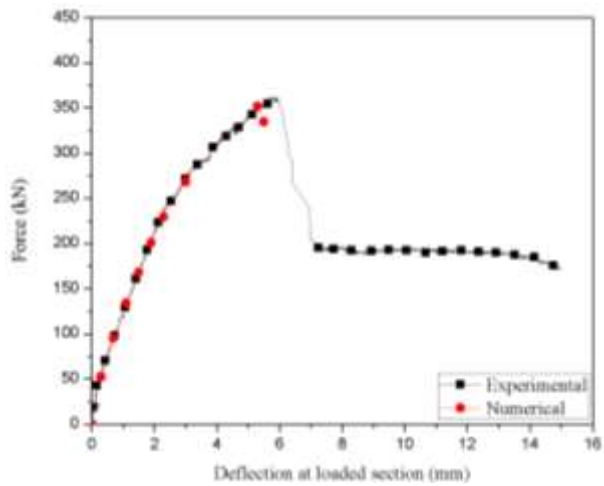
NSM-3L45



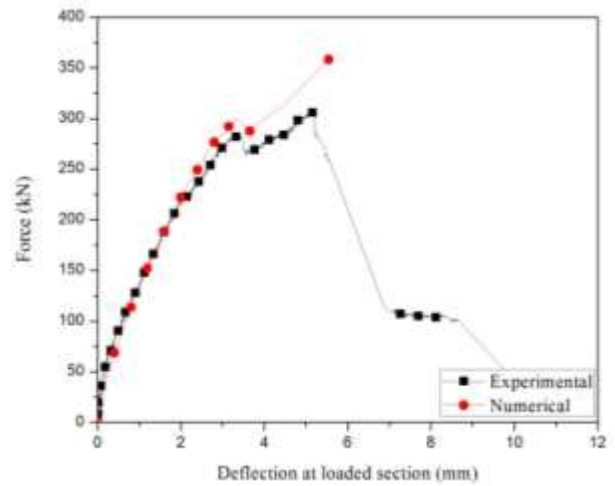
SP



SP-3L45

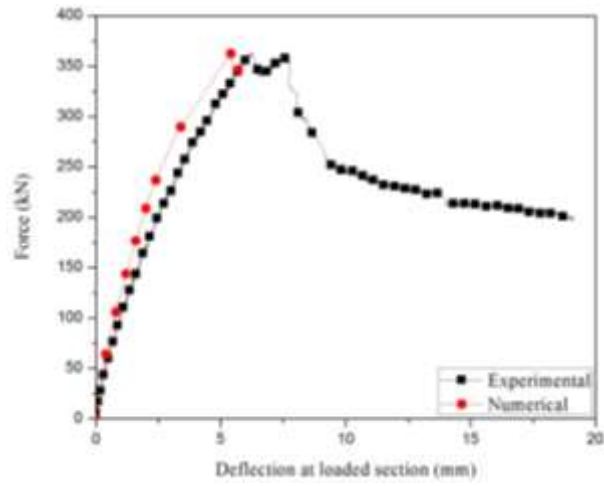


SP-3L45-B



SP-5L45





SP-5L45-B

Fig. 15 - Comparison between experimental and numerical force vs. deflection at the loaded section relationships

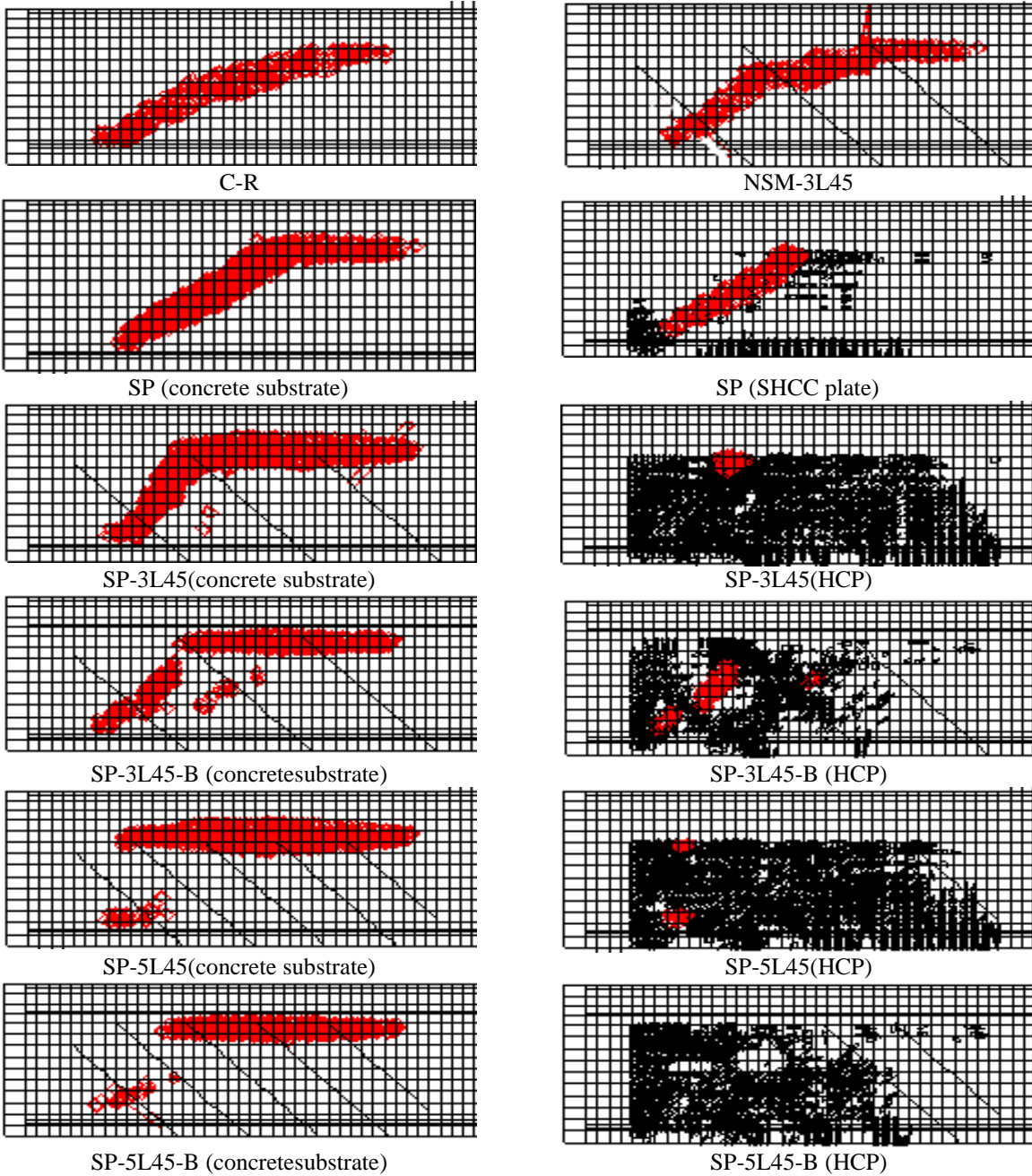


Fig. 16 - Crack patterns of the beams (in red color: crack completely open; in black color: crack in the opening process)

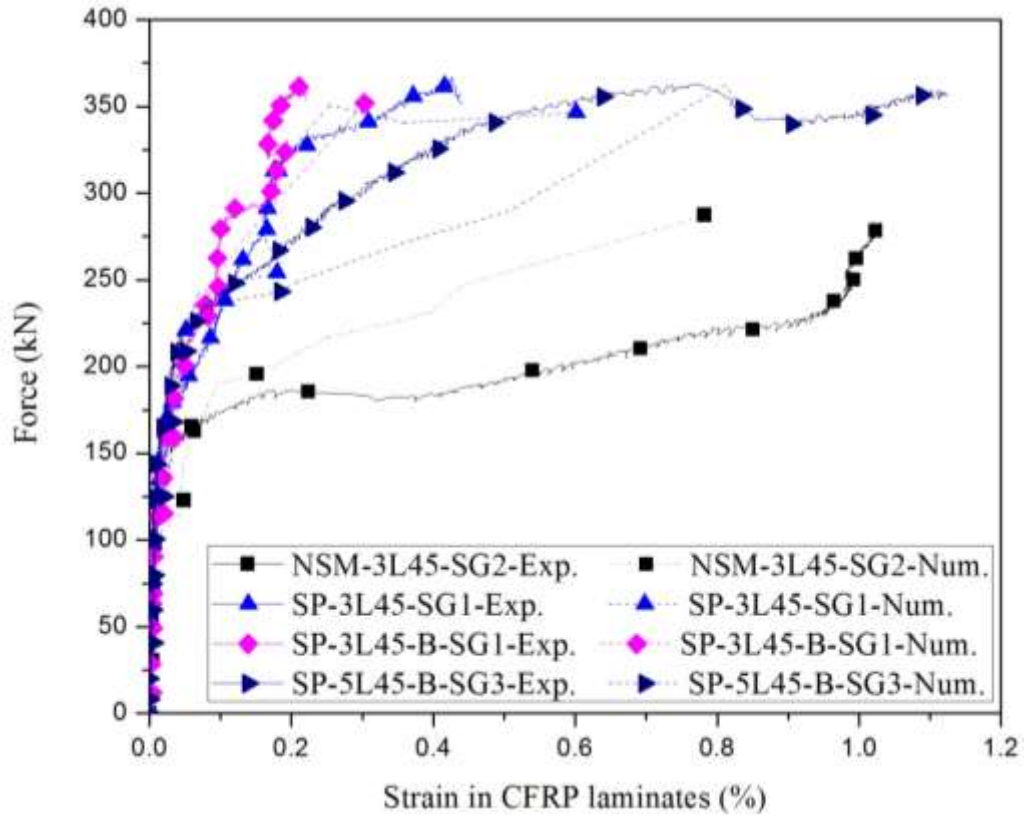
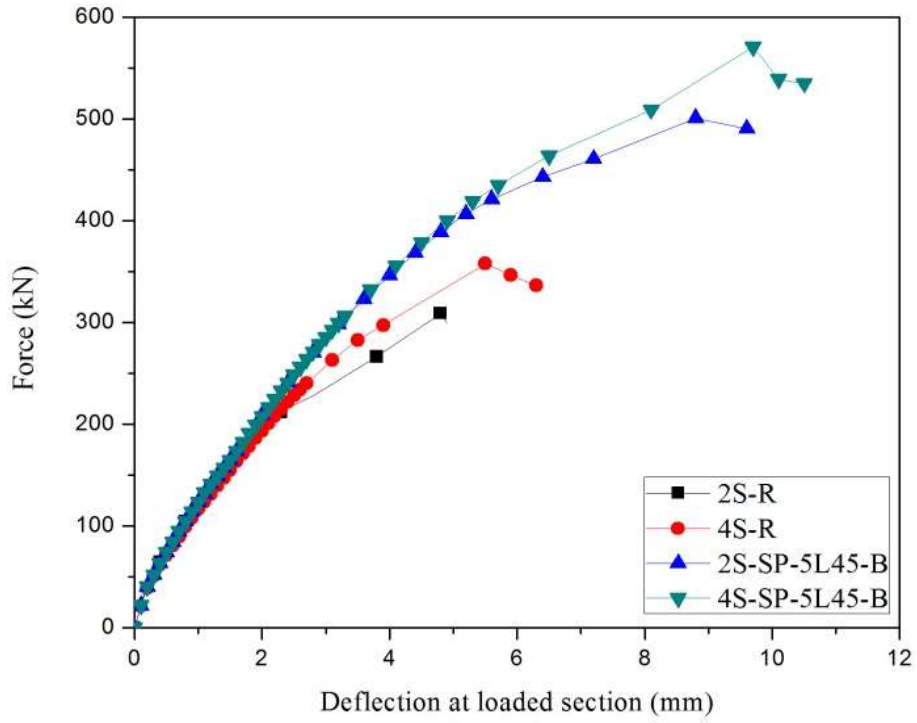


Fig. 17 - Comparison between experimental and numerical force vs. strains in the CFRP laminates



*Fig. 18 – Influence of the reinforcement ratio of existing steel stirrups on the load carrying capacity of HCP strengthened beams*

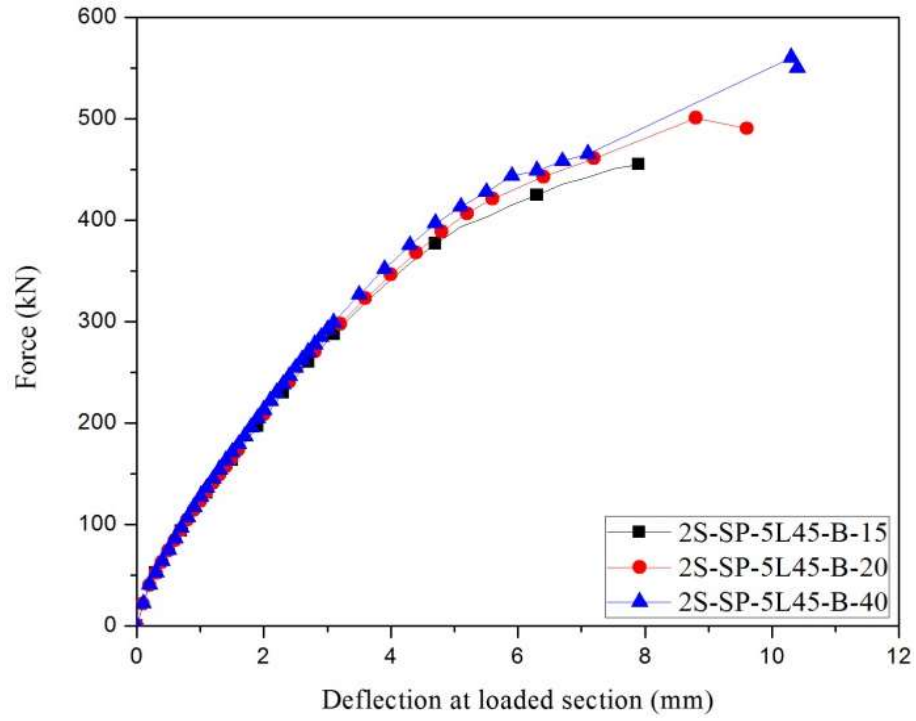


Fig. 19 – Influence of thickness of HCPs on the load carrying capacity and ultimate deflection

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Table 1- Shear strengthening/reinforcement in the monitored shear span of the tested beams

<i>Beam designation</i>	<i>Shear strengthening/reinforcement configuration</i>	<i>Quantity</i>	<i>Connection of the SHCC/HCP to substrate</i>	<i>Percentage of CFRP laminates (%)</i>	<i>Spacing, <math>s_f</math> (mm)</i>
<i>C-R</i>	-	-	-	-	-
<i>SP</i>	SHCC Plates	20 mm thickness of SHCC	Adhesive	-	-
<i>NSM-3L45</i>	NSM CFRP laminates of 1.4×10 mm <sup>2</sup> cross section	2×3CFRP laminates	-	0.08	275
<i>SP-3L45</i>	HCPs (20 mm thickness of SHCC reinforced with CFRP laminates of 1.4×10 mm <sup>2</sup> cross section)	2×3CFRP laminates	Adhesive	0.08	275
<i>SP-5L45</i>		2×5CFRP laminates		0.14	157
<i>SP-3L45-B</i>	HCPs (20 mm thickness of SHCC reinforced with CFRP laminates of 1.4×10 mm <sup>2</sup> cross section)	2×3CFRP laminates	Adhesive and mechanical anchors	0.08	275
<i>SP-5L45-B</i>		2×5CFRP laminates		0.14	157

*Table 2 – Material properties*

<i>Property</i>	<i>Concrete</i>	<i>CFRP laminate</i>	<i>SHCC series B</i>
<i>Compressive strength (MPa)</i>	33.0	-	32.0
<i>Tensile strength (MPa)</i>	-	2620	-
<i>Elasticity modulus (GPa)</i>	-	150	-
<i>Maximum strain (%)</i>	-	1.75	-
<i>Tensile stress at crack initiation (MPa)</i>	-	-	2.5
<i>Tensile strength (MPa)</i>	-	-	3.4
<i>Tensile strain at tensile strength (%)</i>	-	-	1.3



*Table 3- material properties of the steel bars*

<i>Property</i>	$\phi$ 6	$\phi$ 12	$\phi$ 16	$\phi$ 32
$f_{sym} (N/mm^2)$	500	490	470	625
$f_{sum} (N/mm^2)$	595	590	565	905
$E_{sm} (N/mm^2)$	217	196	181	208

Table 4 - Relevant results in terms of load and deflection capacity

Beam designation	$F_{\max}$ (kN)	Deflection at loaded section $\Delta_u$ (mm)	Shear resistance (kN)	$(\frac{\Delta F^{NSM-3L45}}{F^{NSM-3L45}})_{\max}$ (%)	$(\frac{\Delta F^{SP}}{F^{SP}})_{\max}$ (%)	$\frac{\Delta F_{\max}}{F_{\max}^{SP}}$ (%)
C-R	214	3.0	128	-	-	
NSM-3L45	290	5.9	174	0	-	14
SP	255	5.0	153	11	0	0
SP-3L45	367	5.5	220	85	176	44
SP-3L45-B	363	6.2	218	106	178	43
SP-5L45	306	5.1	184	14	131	20
SP-5L45-B	364	6.3	218	174	196	43

*Table 5- Toughness indicator of the beams*

<i>Beam designation</i>	<i>Energy absorption up to the deflection at maximum load (kN.mm)</i>
<i>NSM-3L45</i>	2450
<i>SP</i>	2070
<i>SP-3L45</i>	3200
<i>SP-3L45-B</i>	3360
<i>SP-5L45</i>	2200
<i>SP-5L45-B</i>	3700

*Table 6 - Values of the parameters of the concrete constitutive model*

$\nu_c$	$E_c$ ( $N/mm^2$ )	$f_c$ ( $N/mm^2$ )	$f_{ct}$ ( $N/mm^2$ )	$G_f$ ( $N/mm$ )	$\xi_1$	$\alpha_1$	$\xi_2$	$\alpha_2$	$\tau_{t,p}^{cr}$ ( $N/mm^2$ )	$G_{f,s}$ ( $N/mm$ )	$\beta$
0.19	31381	33.0	2.1	0.08	0.005	0.3	0.1	0.3	1.1	0.045	0.6

1

Table 7 - Values of the parameters of the SHCC constitutive model

$\nu_c$	$E_c$ ( $N/mm^2$ )	$f_c$ ( $N/mm^2$ )	$f_{ct}$ ( $N/mm^2$ )	$G_f$ ( $N/mm$ )	$\xi_1$	$\alpha_1$	$\xi_2$	$\alpha_2$	$\tau_{r,p}^{cr}$ ( $N/mm^2$ )	$G_{f,s}$ ( $N/mm$ )	$\beta$
0.15	18420	32.0	2.5	0.41	0.98	1.18	0.99	1.0	0.9	2.5	0.5

2

3

4

5