

1 **Developments in the use of Ultra High Performance Fiber Reinforced Concrete as**
2 **Strengthening Material**

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6 **Abstract**

7 Ultra High Performance Fiber Reinforced Concrete (UHPFRC) is a novel material which has
8 been developed the last few decades and has been applied in applications that require high
9 strength, ductility and durability. Recently, the material has been applied in strengthening
10 applications. The present study aims to investigate new techniques for the application of
11 UHPFRC as strengthening material and to provide an insight into the parameters affecting the
12 performance of elements strengthened with UHPFRC. The present research investigates for the
13 first time the effectiveness of the use of dowels at the interface between UHPFRC and concrete,
14 to improve the connection between these two materials. Additionally, the effectiveness of the
15 use of UHPFRC jackets for the strengthening of Reinforced Concrete (RC) beams has been
16 examined. In the present research, a systematic experimental study has been conducted together
17 with numerical study.

18 The results demonstrate that both examined techniques are effective and should be taken into
19 consideration when UHPFRC is applied for strengthening material. The dowels result in better
20 bonding at the interface and can delay the formation of cracks in the post elastic phase, leading
21 to reduced interface slip values and subsequent enhanced load bearing capacity. This technique
22 should be taken into consideration to eliminate the risk of premature de-bonding of the
23 strengthening layer. The construction of UHPFRC jackets on the other hand, results in a

24 dramatic increase of the stiffness and the load carrying capacity of the strengthened elements
25 and should be preferred in cases of heavily damaged RC members.

26 **Keywords**

27 Strengthening; Ultra High Performance Concrete; Full scale; Beams; UHPFRC jackets

28 **1. Introduction**

29 The structural upgrade of existing structures is a key priority worldwide. Nowadays, there
30 are many available strengthening techniques. However, the extensive preventative application
31 of these techniques for the protection of existing structures cannot be applied mostly due to
32 issues linked to difficulties during the application of the techniques, which require special
33 expertise, increased cost and construction time.

34 The present research focus on the application of an advanced material, such as the Ultra
35 High Performance Fiber Reinforced Concrete (UHPFRC), for the strengthening of Reinforced
36 Concrete (RC) members. The examined technique present crucial advantages which are related
37 to the enhanced properties of the material and durability, and ease of preparation and
38 application of the material. Another advantage of the examined technique is that thin elements
39 with high strength and ductility can be constructed and therefore the dimensions of the
40 strengthened elements do not change dramatically.

41 There are numerous published studies on the development and characterisation of the
42 mechanical properties of UHPFRC [1-7]. The fiber distribution is an important parameter
43 affecting the performance of UHPFRC and this was thoroughly investigated by Ferrara et al.
44 [1]. Nicolaidis et al. [2], developed an optimum mixture of Ultra High Performance
45 Cementitious Composite with materials which were locally available in Cyprus, while
46 Paschalis and Lampropoulos [3], investigated the effect of fiber content and curing time
47 regimes on the tensile characteristics of UHPFRC. In this study it was found that the fiber

48 content significantly affects the tensile characteristics of UHPFRC, and different stress-strain
49 models for various typical fibers contents, were proposed. The mechanical characteristics of
50 UHPFRC under different loading conditions, including static and cyclic loading, have been
51 studied in previously published studies [4-7].

52 The application of UHPFRC for the retrofitting of existing RC members has also been
53 studied recently [8-12]. Bruhwiler and Denarie [8], applied UHPFRC in a series of
54 rehabilitation applications, such as in a road bridge, a bridge pier and an industrial floor. Al-
55 Osta et al. [9], studied the effectiveness of different techniques for the strengthening of RC
56 beams using prefabricated UHPFRC strips and comparisons were made with cast-in-situ
57 UHPFRC layers. Different strengthening configurations, such as strengthening on different
58 sides of the beams were also investigated in this study. Safdar et al. [10], applied UHPFRC as
59 a repair material and found that this application can increase mainly the stiffness of the
60 strengthened elements. Lampropoulos et al. [11], presented a numerical investigation on the
61 performance of UHPFRC for the flexural strengthening of RC beams and the effectiveness of
62 the technique was highlighted. Paschalis et al. [12], presented an experimental and numerical
63 investigation on the performance of UHPFRC for the flexural strengthening of full scale RC
64 beams. In this study, UHPFRC layers with and without the use of steel bars in the layers were
65 used for the strengthening of RC beams, and it was found that the UHPFRC layers can increase
66 the stiffness of the strengthened members, while the addition of steel bars to the layers can
67 produce a big increase of the load carrying capacity. Finally, a better bonding between
68 UHPFRC and concrete was identified compared to concrete to concrete interfaces. However,
69 in this study it was found that the slips at the interface were not negligible. On the contrary,
70 high values of slip at the interface were recorded in the post-elastic region.

71 Based on existing studies, the application of UHPFRC can be effectively used to
72 enhance the stiffness of the strengthened elements, while the load bearing capacity can also be

73 further increased with the addition of steel bars. However, the addition of steel bars is a labour-
74 intensive technique and requires a minimum thickness for the strengthening layer leading to
75 significant changes in the geometry and the characteristics of the initial structure.

76 The present study aims to investigate new techniques for application of UHPFRC as
77 strengthening material with the optimum result and to provide an insight into the key
78 parameters affecting the structural performance of strengthened elements with UHPFRC.
79 Therefore, the present research investigates for the first time the effectiveness of the use of
80 dowels at the interface between UHPFRC and concrete, to improve the connection between
81 these two materials and to eliminate the risk of premature de-bonding. Also, in the present
82 study the effectiveness of the use of three-side UHPFRC jackets has been examined. Both
83 techniques aim to upgrade the structural performance of the existing elements without the use
84 of steel bars in the strengthening layers/jackets. The application of this technique could be
85 easily applied in strengthening applications reducing labouring cost and effort [13-15].

86 **2. Experimental program**

87 **2.1 Strengthening techniques**

88 Eight RC beams have been examined in the present research. Two beams were used as
89 control beams without any retrofit, two beams were strengthened with UHPFRC layers without
90 dowels and another two identical beams were strengthened with UHPFRC layers and dowels
91 at the interface. Finally, two beams were strengthened with three-side UHPFRC jackets (Table
92 1). In all the strengthened beams the surfaces of the initial beams were roughened prior to the
93 casting of the UHPFRC layer.

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96 **Table 1** Examined Beams

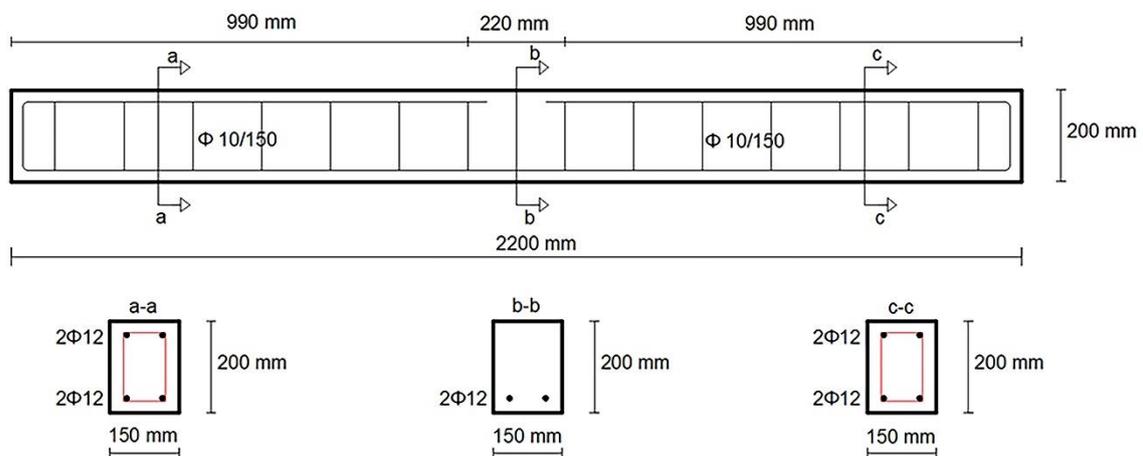
Beam	Strengthening Technique
P1	Control beam
P2	Control beam
U1	UHPFRC layer
U2	UHPFRC layer
D1	UHPFRC layer and dowels
D2	UHPFRC layer and dowels
3SJ1	UHPFRC Jacket
3SJ2	UHPFRC Jacket

97

98 The initial beams (Figure 1) were reinforced using two steel bars grade B500C with 12

99 mm diameter at the tensile side. Also, shear links of the same steel grade with 10 mm diameter

100 were placed at 150 mm spacing along the length of the beam.



101

102 **Fig. 1** Geometry and reinforcement of the RC beams

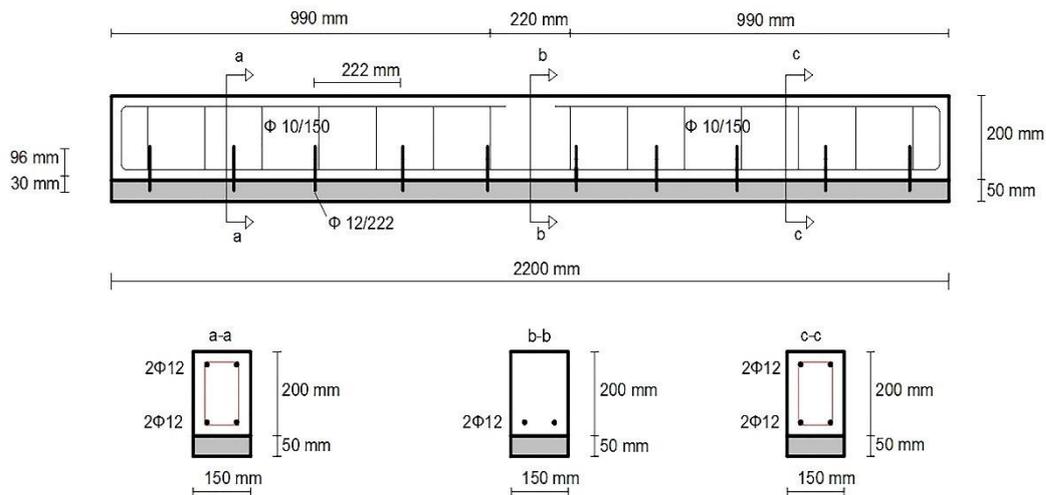
103 The layers had a depth of 50 mm, a breadth of 150 mm (equal to the breadth of the initial

104 beam) and were cast along the whole length of the tensile side of the beams. As can be seen in

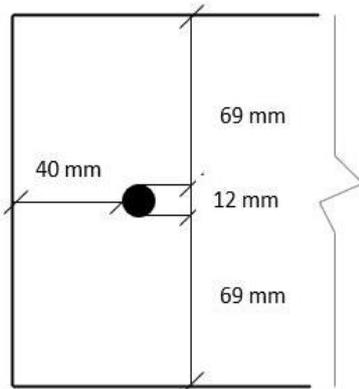
105 Figure 2a, ribbed steel bars with a length of 126 mm, in total, 12 mm diameter and a spacing

106 of 222 mm were used as dowels. The design of dowels was based on the Greek Retrofitting

107 Code [16]. When steel bars are used as dowels, the minimum required cover in the direction of
 108 loading should be at least $5d_b$ for the front cover and $6d_b$ at the back cover (where d_b is the
 109 diameter of the bars) to prevent premature failure of the concrete edges around the dowels [16].
 110 On the vertical direction, a cover of at least $3d_b$ is required, while it is also suggested that the
 111 length of the bars inside concrete should be at least $8d_b$ [16]. In Figure 2b, the position of the
 112 dowels is presented



113

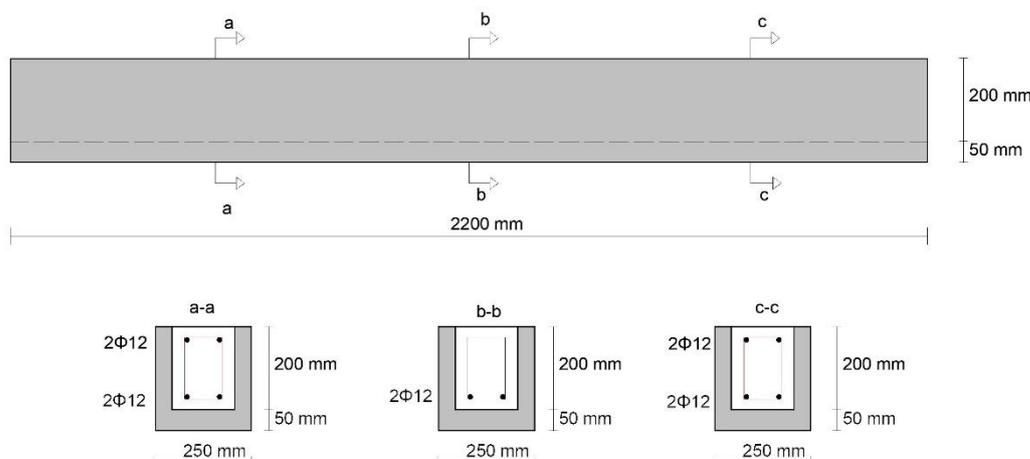
114 **Fig. 2a**

115

116 **Fig. 2b**

117 **Fig. 2 a)** Geometry and reinforcement of the strengthened beams with layers and dowels b)
 118 Position of the dowels

119 The strengthening technique and the geometry of the strengthened beams with
 120 UHPFRC jackets on three sides is presented in Figure 3.



121

122 **Fig. 3** Strengthening with three-side jackets

123 As can be seen in Figure 3, a thickness of 50 mm was used for the UHPFRC jacket
 124 leading to a beam with total breadth equal to 250 mm and a height of 250 mm. The jackets
 125 were cast along the full length of the beams.

126 2.2 Preparation of the specimens

127 For the preparation of the UHPFRC, fine sand with a maximum particle size of 500 μm
 128 was used together with microsilica, Ground Granulated Blast Furnace Slag (GGBS) and high
 129 strength cement class 52.5 R type I. In addition, steel microfibers with a length of 13 mm and
 130 a diameter of 0.16 mm were incorporated in the mix.

131 According to study [17], a fiber content in the range of 3-4 Vol-%, is considered an
 132 optimum content for the preparation of the UHPFRC considering parameters such as; the
 133 rheological properties, the performance of the material and the cost. Therefore, in the present
 134 study a fiber content of 3 Vol-% was adopted. The mixture design of UHPFRC is presented in
 135 Table 2 and the mixture design of the conventional concrete is presented in Table 3.

136 **Table 2** The mixture design for the preparation of UHPFRC

137

Material	Mix proportions (kg/m ³)	
Cement	620	138
GGBS	434	139
Silica fume	140	140
Silica Sand	1051	141
Superplasticizer	59	142
Water	185	143
Steel fibers	235.5 (3 Vol.-%)	144
		145

146 **Table 3** The mixture design for the preparation of concrete

147

Material	Mix proportions (kg/m ³)	
Cement	340	148
Fine Aggregates	1071	149
Coarse Aggregates	714	150
Water	205	151
		152

153

154 The initial beams were wet cured daily for 28 days. A pistol grip needle scaler was
 155 employed and all the strengthened beams were roughened to a depth of 2-2.5 mm. To quantify
 156 the surface texture, the sand patch method was used [12]. Once the desired roughening depth
 157 was achieved and the surface was ready, the beams were drilled, using an impact drill, and the
 158 dowels were placed in position. Based on Technical Specifications on the placement of dowels
 159 in concrete elements [18], when steel bars are used as dowels, it is suggested that the diameter
 160 of the hole (d_h) should be 4 mm higher than the diameter of the connectors (d_b).

161 For the connection of the dowels with the beams, a thixotropic structural two-part
162 adhesive was used. According to the specifications from the manufacturer, the compressive
163 strength of the epoxy over 14 days at +15°C was 70-80 MPa, while the tensile strength was in
164 the range of 25-28 MPa.

165 Before the application of the jackets on the other hand, the beams were roughened on
166 all the three sides. Once the coating was removed and the desired depth was achieved, the
167 beams were cleaned carefully before the casting of the jackets.

168 The UHPFRC layers and jackets were applied two months after the casting of the initial
169 beams and the strengthened beams were tested after four months. In Figure 4, the procedure
170 for the preparation of the strengthened beams with layers and dowels (Figure 4a and 4b) and
171 jackets (Figure 4c) is presented.

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183 **Fig. 4a**

Fig. 4b

Fig.4c

184 **Fig. 4** a) Drilling of the initial RC beam b) dowels in place c) roughened beam on 3 sides

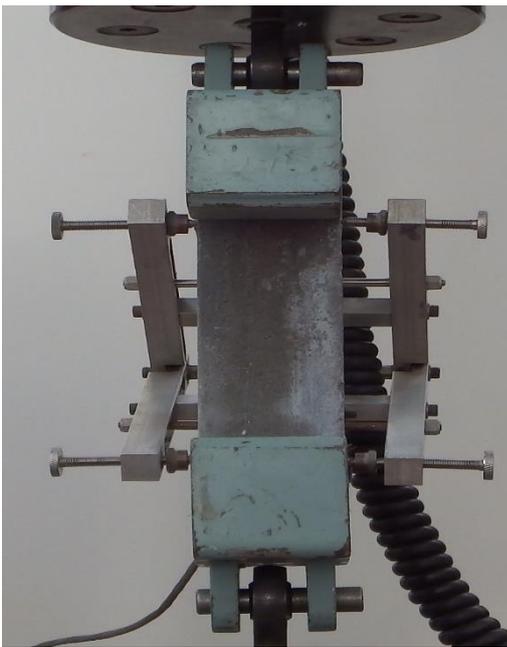
185 **2.3 Properties of the materials**

186 Three cubes of 100 mm side were tested to identify the compressive strength of both
187 conventional concrete and the UHPFRC. The compressive tests were conducted following the
188 BS EN 12390-3:2009 [19]. The samples were obtained from the same batch with the examined
189 specimens, water cured at ambient temperature for 28 days and tested the same day with the
190 testing of the beams. The average compressive strength of the conventional concrete cubes was

191 30.9 MPa and the standard deviation was found to be equal to 2.34 MPa. The average
192 compressive strength of the UHPFRC on the other hand, was found to be equal to 136.5 MPa
193 and the standard deviation was 5.5 MPa.

194 The tensile properties of the UHPFRC were evaluated using six dog-bone shaped
195 specimens (Figures 5a-b). Tests were conducted under constant loading with a displacement
196 rate of 0.007 mm/sec, which is the same loading rate has been used in other studies [3,4,12]
197 leading to comparable results. The tensile stress-strain results are illustrated in Figure 5c. From
198 the average curve, the maximum stress was found to be equal to 11.5 MPa, the elastic limit was
199 5 MPa and the modulus of elasticity was calculated equal to 51 GPa. All the specimens were
200 tested the same day with the strengthened beams.

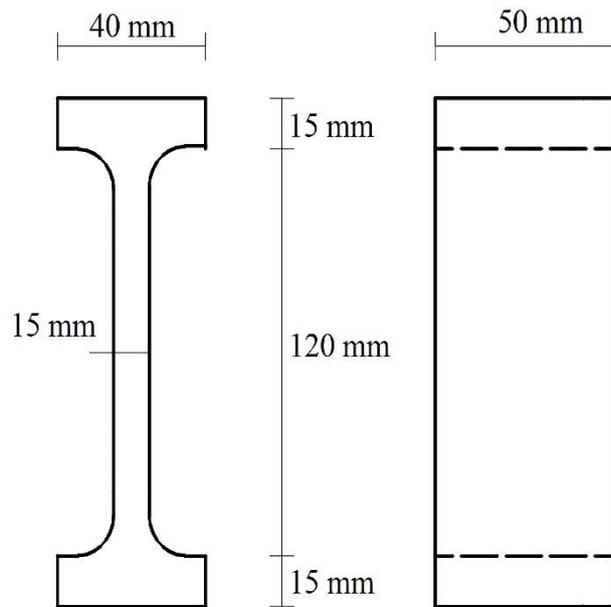
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202

203 **Fig. 5a**

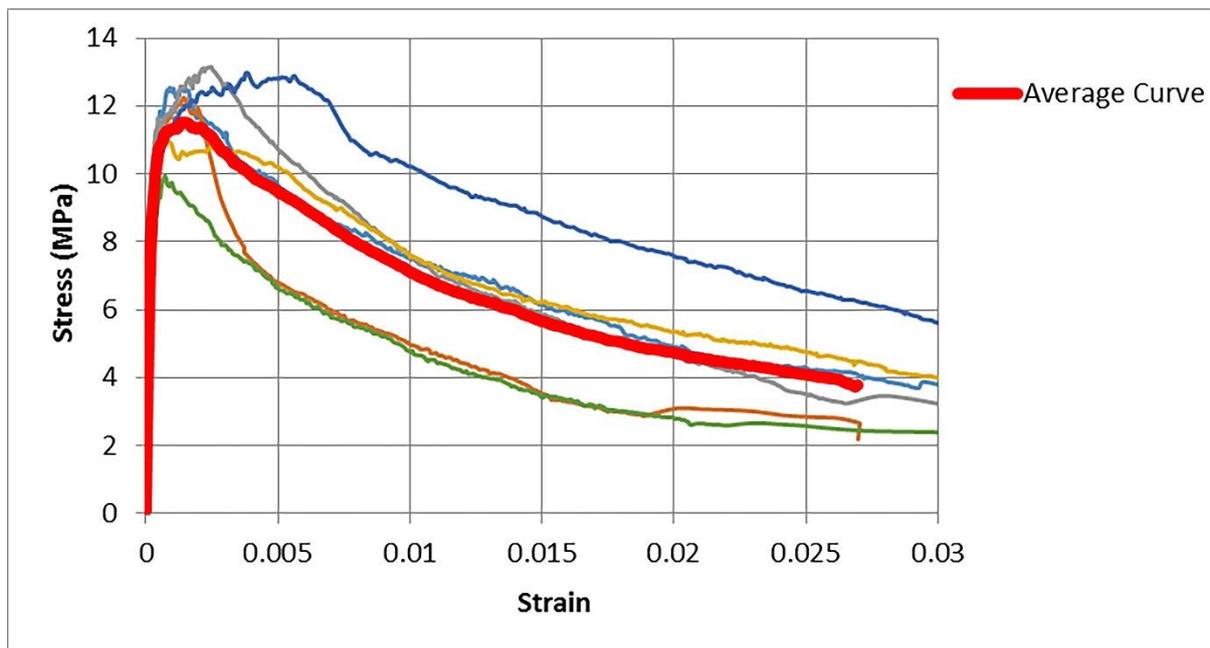
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205

206 **Fig. 5b**

207



208

209 **Fig. 5c**210 **Fig. 5** a) Dog bone specimen before Testing b) Dimensions of the Dog bone Specimen c)

211 Experimental results from the direct tensile tests of UHPFRC

212 2.4 Testing of the beams

213 The examined beams, were tested under a four-point loading test with a displacement
214 rate of 0.008 mm/sec, which is in agreement with the loading rate used by other researchers
215 [12,22] leading to comparable results The experimental setup for the examined beams is
216 presented in Figures 6 a-d.



217

218 **Fig. 6a**



219

220 **Fig. 6b**



221

222 **Fig.6c**



223

224 **Fig.6d**225 **Fig. 6** Experimental setup for the a) control beam b) strengthened beam with layers c)

226 strengthened beam with layer and dowels d) strengthened beam with jacket

227 During the testing of the strengthened beams with layers, apart from the load carrying

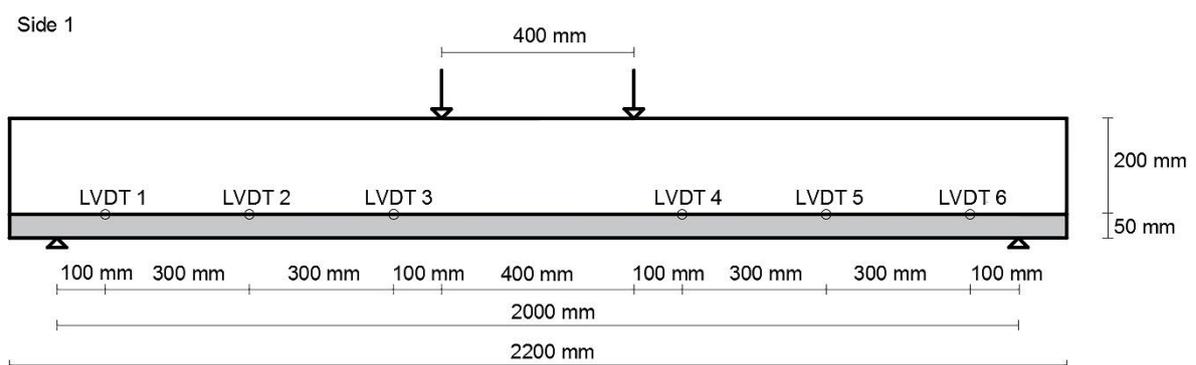
228 capacity and the deflection, the slips at the interface between UHPFRC and RC were recorded

229 using nine Linear Variable Differential Transformers (LVDTs) in total. As can be seen in

230 Figures 7a and 7b, six LVDTs were placed on side 1 along the full length of the beam, while

231 three LVDTs were placed at the second side to validate the results obtained from side 1.

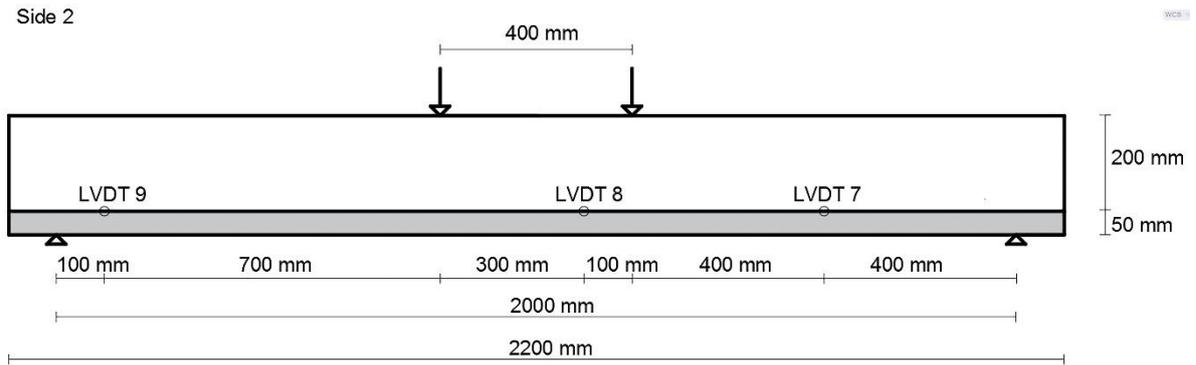
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233

234 **Fig. 7a**

235



236

237 **Fig. 7b**238 **Fig. 7 a)** Positions of the LVDTs for the measurement of slips on side 1 b) positions of the

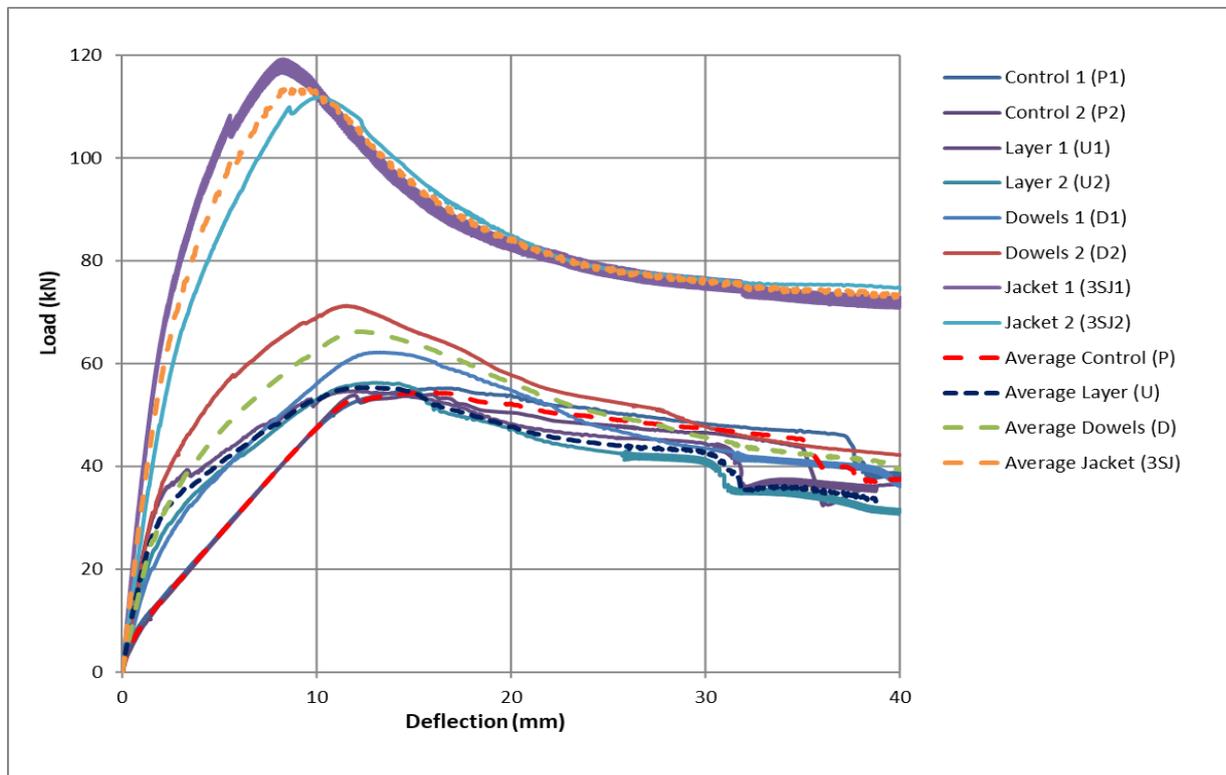
239 LVDTs for the measurement of slips on side 2

240 **2.5 Experimental results of the examined beams and discussion**

241 The experimental results for the control beams, the strengthened beams with UHPFRC

242 layers, the strengthened beams with UHPFRC layers and dowels and the strengthened beams

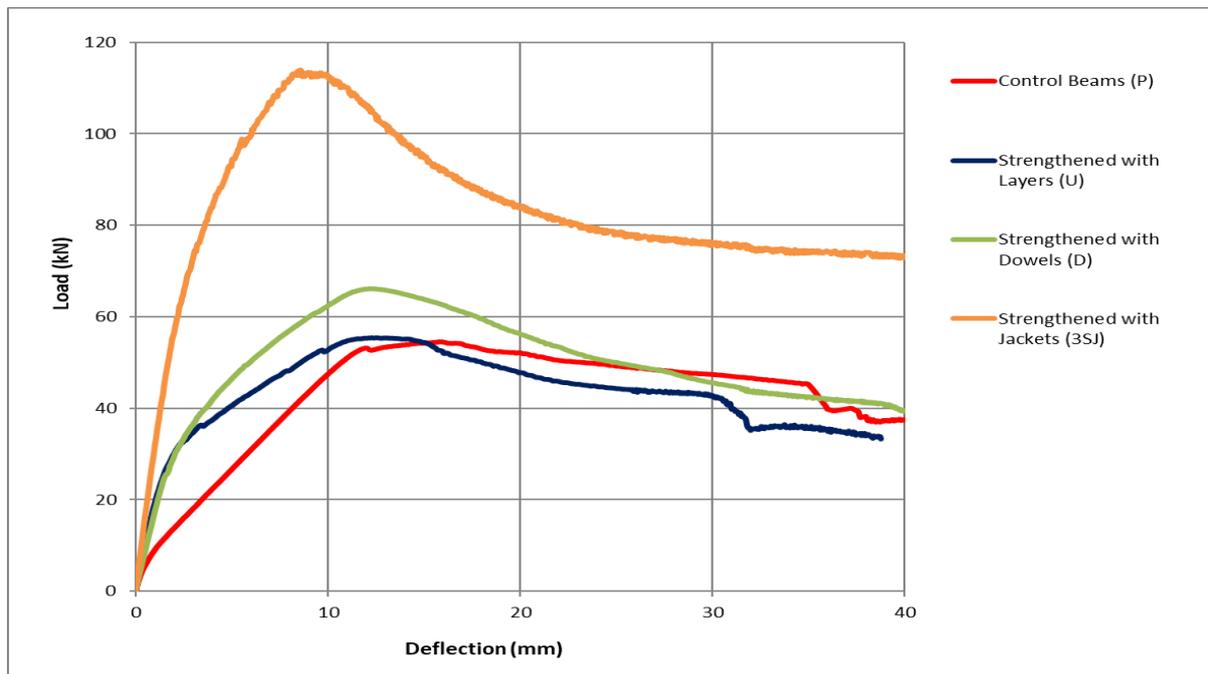
243 with jackets are presented in Figure 8.



244

245 **Fig. 8** Experimental results for all the examined beams

246 The average load-deflection results, for all the examined beams, are presented in the same
 247 graph in Figure 9.



248
 249 **Fig. 9** Average load-deflection results

250 From Figure 9, the end of the linear part for each configuration was identified and is
 251 representing the formation of significant cracks, and the values are presented in Table 4. In this
 252 table, the average values for the maximum load, the stiffness and the displacement at the
 253 maximum load are also presented.

254 **Table 4** Maximum load and Stiffness for all the examined beams

Beam	Maximum load (kN)	Stiffness (kN/mm)	Start of Cracking (kN)	Displacement At Peak Load (mm)
Average P1,2	54.6	9.2	5	15.8
Average U1,2	55.3	21.5	15	12.3
Average D1,2	66.2	18.3	24	12
Average 3SJ 1,2	114.5	34	58	8.6

255

256 The results of Figure 8, show a small scatter of the results of the identical samples. As
257 illustrated in this figure, similar values were achieved for the maximum load of beams P1 and
258 P2 (55.1 kN for beam P1 versus 54 kN for beam P2) and the stiffness (10.5 kN/mm for beam
259 P1 versus 8.4 kN/mm for beam P2). Considering the average load-deflection curve, the
260 maximum load was found to be equal to 54.6 kN, the stiffness was calculated to be 9.2 kN/mm
261 and the deflection at the maximum load was equal to 15.8 mm.

262 The load-deflection results obtained for the identical beams U1 and U2 are also presented
263 in the same graph in Figure 8. From this figure, it is clear that there is a positive agreement in
264 the experimental results for both the identical beams in terms of maximum load (54.6 kN for
265 beam U1 versus 56.3 kN for beam U2) and the stiffness (23 kN/mm for beam U1 versus 19.2
266 kN/mm for beam U2). Based on the average load-deflection curve, the maximum load was
267 found to be equal to 55.3 kN, the stiffness was calculated to be equal to 21.5 kN/mm and the
268 deflection at the maximum load was equal to 12.3 mm.

269 From Figure 8, it can also be noticed that the load carrying capacity of beam D2 (71.3
270 kN) and the stiffness (21 kN/mm), were higher compared to the respective values of beam D1
271 (62.2 kN the maximum load and 16.5 kN/mm the stiffness), which is attributed to imperfections
272 during the preparation of the samples and during the installation of the dowels. However, both
273 beams exhibited higher load carrying capacity compared to beams U1 and U2. Considering the
274 average curve, the maximum load was found to be equal to 66.2 kN, the stiffness was 18.3
275 kN/mm and the deflection at the maximum load was equal to 12 mm.

276 From Figures 8 and 9, a big increase in the load the load carrying capacity and the
277 stiffness of the strengthened beams with jackets can be distinguished, compared to all the other
278 techniques. The maximum load of beam 3SJ1 was found to be equal to 119.2 kN and the
279 stiffness was calculated to be equal to 38.8 kN/mm. The maximum load of beam 3SJ2 on the
280 other hand, was found to be equal to 112 kN and the stiffness 30.5 kN. Considering the average

281 curve, the maximum load was equal to 114.5 kN, the stiffness was 34 kN/mm and the deflection
282 at the maximum load was equal to 8.6 mm.

283 Considering the average results of Figure 9 and Table 4, it is clear that in all the examined
284 cases the stiffness of the strengthened members was significantly enhanced. Based on the
285 results of the present investigation, the contribution of UHPFRC layers without the use of
286 dowels or any other reinforcement, relies mainly on the delay of the formation of cracks and
287 the significant stiffness enhancement. Based on the average results of Table 4, strengthening
288 with UHPFRC layers resulted in significant stiffness increment (134%), while the load capacity
289 was only slightly increased by 1.5% . These results are also in agreement with other studies in
290 the literature review [10,12]. In these studies it was highlighted that the main contribution of
291 UHPFRC layers is on the delay of the formation of cracks and the significant increment of the
292 stiffness. On the contrary, the contribution UHPFRC layers in the load carrying capacity is
293 lower.

294 From Figure 9 and Table 4, it can also be observed that the addition of dowels, apart
295 from the increment of stiffness, resulted in a significant enhancement of the load bearing
296 capacity of the strengthened beams. More specifically, an average increment of 21.5% was
297 identified in this case. On the contrary, the average increment in the load carrying capacity of
298 the strengthened beams U1, U2 was only 1.5%.

299 During the testing of the strengthened beams with UHPFRC layers and dowels, it was
300 observed that the formation of cracks was delayed compared to the strengthened beams with
301 UHPFRC layers only. This was obvious from the visual inspection of the beams during the
302 testing, and was also reflected to the load-deflection results. From the results of Table 4 it is
303 clear that the addition of dowels at the interface had as a result the delay in the formation of
304 the cracks compared to the strengthened beams without dowels. When dowels were placed at

305 the interface between RC and UHPFRC, cracking was initiated at a load value 60% higher
306 compared to the beams with UHPFRC layers without dowels.

307 The results of the present investigation also indicate that the construction of jackets on
308 three sides is also a very effective strengthening technique which can improve the performance
309 of the strengthened members. From Figure 9 and Table 4, it can be seen that the addition of
310 jackets on three side resulted to a remarkable increment of both the stiffness and the load
311 carrying capacity of the strengthened members. In this case, the maximum load was increased
312 by 110% and the stiffness was enhanced by 270%. It can also be seen that the formation of
313 cracks was initiated at a value of load 334% higher compared to the beams strengthened with
314 UHPFRC layer only.

315 In the literature there are limited applications of jackets for strengthening of RC
316 members. Al-Osta et. [9] investigated different configuration such as strengthening on one side,
317 on two sides and three sides. The biggest load increment (89%) was noticed when three sides
318 jacket was applied. In the present study, the biggest load carrying capacity was also identified
319 for strengthening with UHPFRC jackets, but with a higher increment of 110%.

320 In comparison with other strengthening techniques in the literature using UHPFRC, the
321 addition of three-side jacket appears to be the most effective in terms of structural performance.
322 Paschalis et al [12], used steel bars in the UHPFRC layers as extra reinforcement for the
323 strengthening of existing RC beams at the tensile side. The geometry of the beams and layers,
324 and the material properties in this study were the same with the present study. The results
325 showed an increment of 89.5% on the maximum load and 111% on the stiffness of the
326 examined beams. These values are lower compared to the jacketed beams, where increment of
327 110% on maximum load and 334% on the stiffness were achieved.

328

329 In Figures 10a-f the failure mode of the examined beams is presented.



330

331 **Fig. 10a**



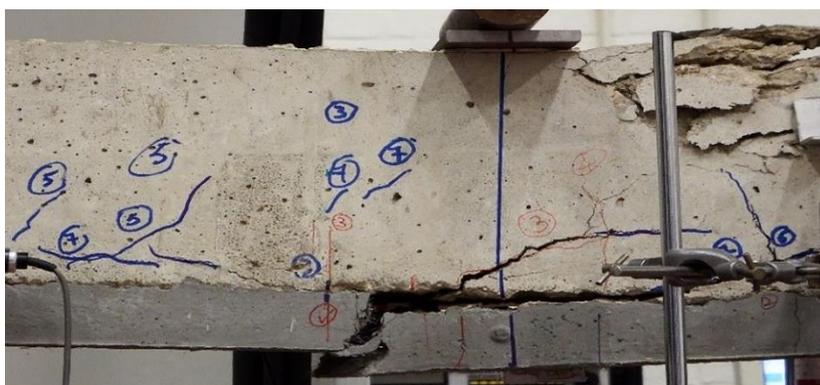
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333 **Fig. 10b**



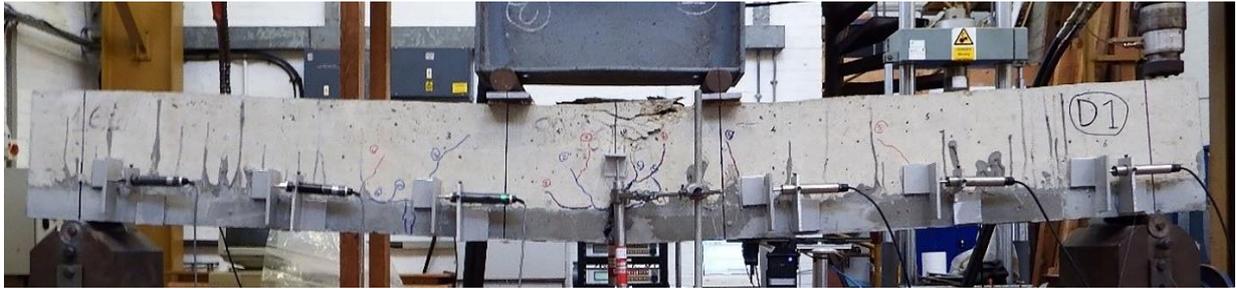
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335 **Fig. 10c**



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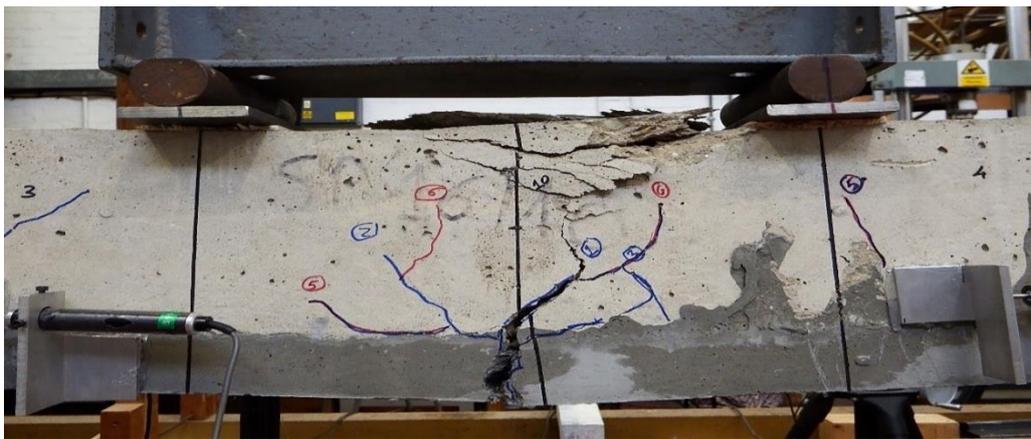
337 **Fig. 10d**



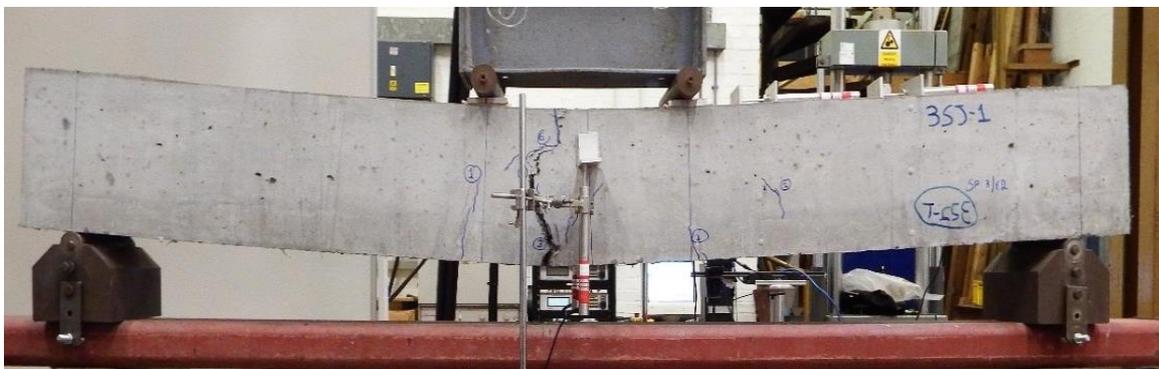
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339 **Fig. 10e**

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341

342 **Fig. 10f**

343

344 **Fig. 10g**345 **Fig. 10** Beams at failure: a) control beam P1 b) strengthened beam with UHPFRC layers U1 c)

346 strengthened beam with UHPFRC layers U2 d) local de-bonding at the interface of beam U1

347 e) strengthened beam with dowels D1 f) bonding at the interface of beam D1 g) strengthened

348 beam with jacket 3SJ1

349 The failure mode of the control beams P1 and P2 was identical and was characterized by
350 flexural cracks (Figure 10a). A single crack appeared initially in the middle of the span
351 followed by additional cracks which propagated towards the supports. Inclined shear cracks
352 appeared at a later stage, followed by the crushing of the compressive side at the middle of the
353 span which occurred in the descending branch of the load-deflection curve.

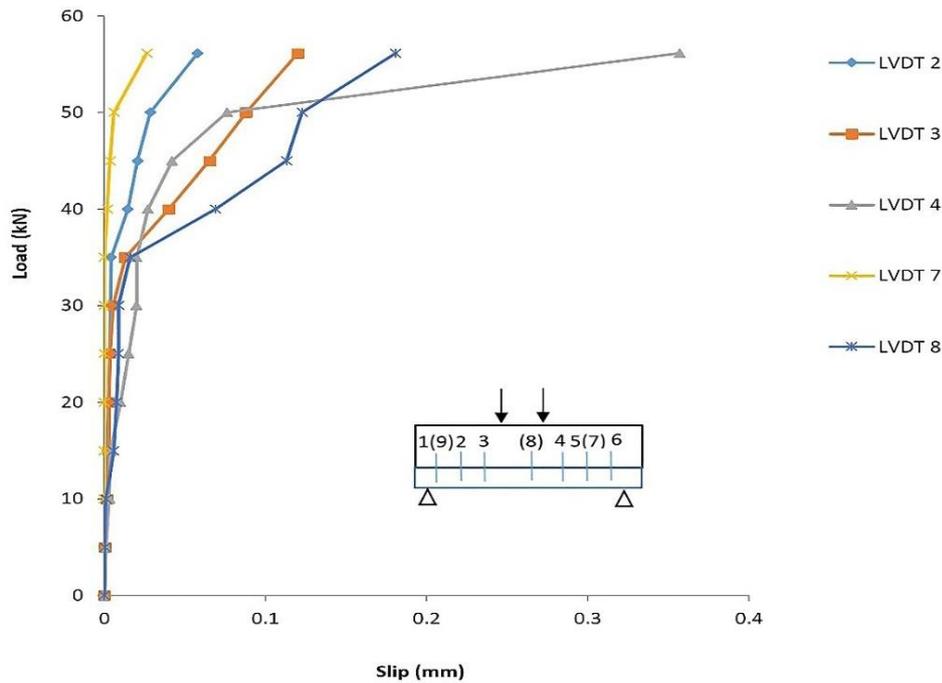
354 The failure of the beams which were strengthened with UHPFRC layers only, started
355 with a major flexural crack which was initiated at the UHPFRC layers and propagated
356 progressively to the RC beams. Additional minor cracks appeared near the interface during the
357 testing. As can be seen in Figures 10 b-c less cracks appeared in these beams compared to the
358 control beams. However, during of the testing of beam U1, a local de-bonding at the interface
359 between UHPFRC and concrete started when the load reached a value of 48 kN and before the
360 maximum load carrying capacity (Figure 10d). After post-testing inspection of the specimen,
361 it was observed that parts of the interface of this specimen had lower roughness depth, therefore
362 the premature failure of this specimen is attributed to the insufficient interface treatment. On
363 the contrary, the bonding at the interface of beam U2 was effective (Figure 10c), and in this
364 case de-bonding was prevented and typical flexural failure occurred at the middle of the span
365 length, followed by crushing of the compressive side.

366 The failure mode of beams D1 and D2 was identical. The dowels resulted in a strong
367 bonding between UHPFRC and concrete and de-bonding was prevented in both cases. The
368 specimens failed with a single flexural crack in the middle of the span. These cracks appeared
369 on the UHPFRC layer and progressively propagated to the RC beam resulted in the failure of
370 the beams. Additional minor inclined cracks appeared during the testing of the beams (Figure
371 10e). Crushing of the compressive side occurred at the post peak load-deflection region. In
372 Figure 10f it can be seen that one single crack started from the layer and propagated at exactly
373 the same position of the RC beam. This shows very good connection at the UHPFRC-to-

374 concrete interface which is attributed to the addition of dowels. The enhancement of the
375 interface connection and the prevention of de-bonding which was achieved in this case shows
376 the beneficial contribution of the dowels. This needs to be considered in real life strengthening
377 applications where interface treatment imperfections may exist and this may lead to de-bonding
378 and failure of the elements as occurred in specimen U1.

379 Beams 3SJ1 and 3SJ2 which were strengthened with jacket on three sides, failed in a
380 similar way with a main flexural crack at the tensile side. As can be seen in Figure 10g, the
381 position of the major crack of beam 3SJ1 was in the middle of the span length. Beam 3SJ2 on
382 the other hand, failed in the position of close to the loading point. After post testing inspection
383 of this specimen, it was found that at this position the steel fibers were not evenly distributed
384 which may have resulted in failure at this point.

385 To further investigate the crucial topic of the bonding between UHPFRC and concrete,
386 measurements of the slips at the interface in different positions, were also recorded during the
387 testing of the strengthened beams (Figure 7). As aforementioned, a local de-bonding at the
388 interface of beam U1 occurred, and for this reason, the measurements of this beam were
389 ignored. The load versus interface slip results in different positions of beam U2 are presented
390 in Figure 11.

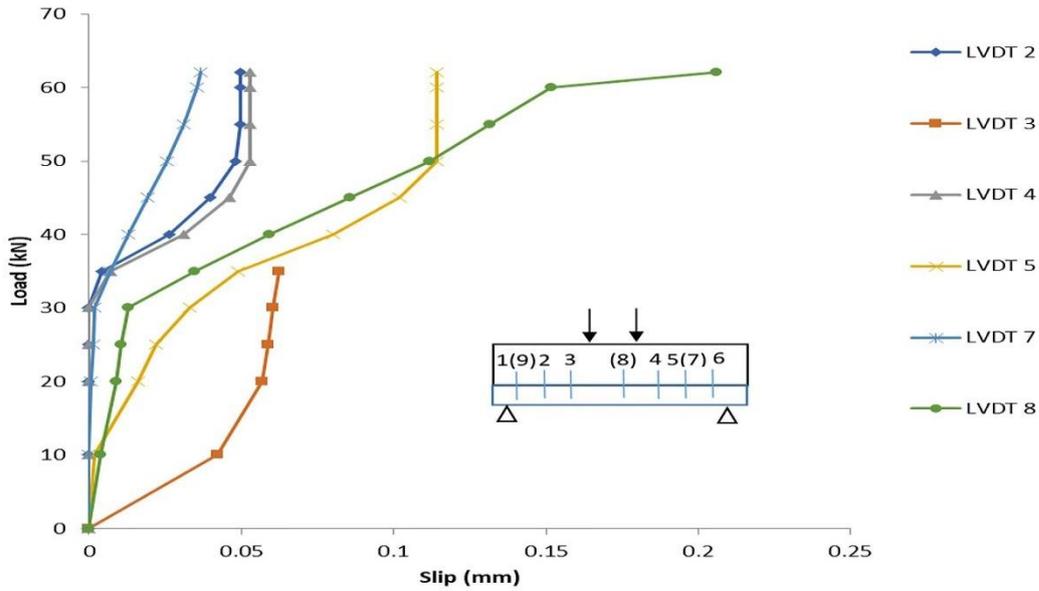


391

392 **Fig. 11** Load versus slip in different positions for beam U2

393 As can be seen in Figure 11, slips were recorded at the positions of the LVDTs 2, 3, 4, 7
 394 and 8, while for all the other positions negligible values were recorded indicating good
 395 connection (and therefore these points are not included in Figure 11). The maximum value of
 396 slip was recorded at the position of LVDT 4 with 0.36 mm (where the shear is maximum),
 397 while a value of 0.18 mm was recorded at the position of LVDT 8, which is located between
 398 the two loading points. Values in the range of 0.03-0.12 mm were recorded at the positions of
 399 LVDTs 2, 3 and 7. Zero values were recorded at the positions close to supports, as expected,
 400 due to the high shear strength at these points resulting from the support reactions.

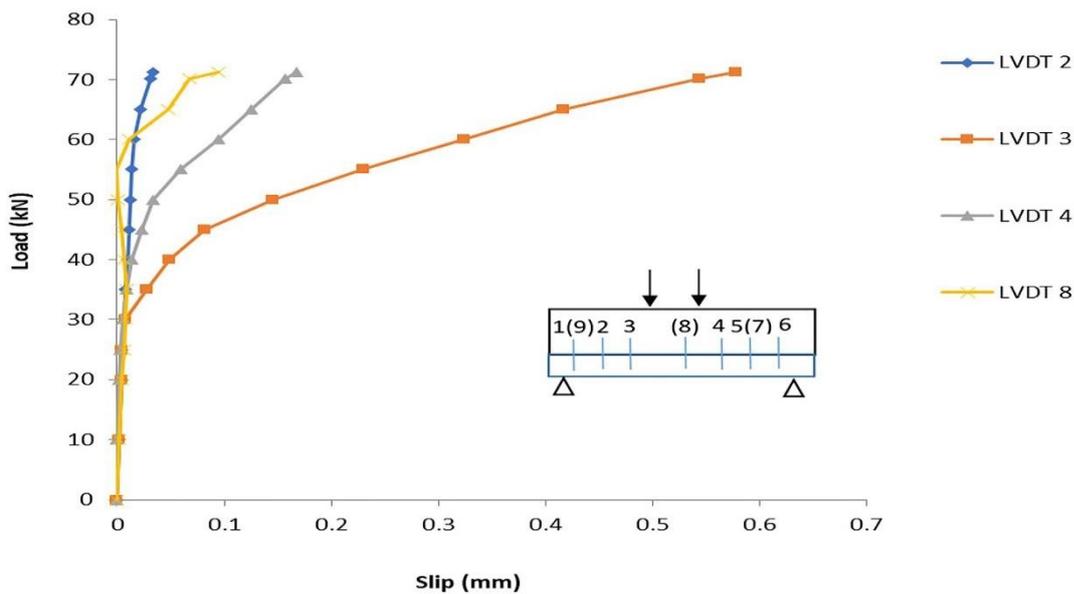
401 The measurements of the slips at the interfaces of beams D1 and D2 are presented in
 402 Figures 12a and 12b.



403

404 **Fig. 12a**

405



406

407 **Fig. 12b**

408 **Fig. 12** a) Load versus slip in different positions for beam D1 b) load versus slip in different
 409 positions for beam D2

410 As shown in Figure 12a, the maximum value of slip of beam D1 was recorded at the
 411 position of LVDT 8, located between the two loading points, while only negligible slip values
 412 were recorded at the area close to the supports (and these points are not presented in Figure

413 12a). By contrast, values between 0.036 mm and 0.055 mm were recorded at the positions of
 414 LVDTs 2, 4 and 7. Due to the fact that the metal angle section which supported LVDT 3 was
 415 detached from the beam at a load value of 35 kN, these measurements are missing.

416 The measurements of slips at the interface of beam D2 are presented in Figure 12b. As
 417 can be seen, slips were recorded only at the positions of LVDTs 2,3,4 and 8. Small values of
 418 0.033 mm and 0.088 mm were recorded at positions 2 and 8 respectively. The highest values
 419 of slip were recorded at the positions 3 and 4. It is worth mentioning that the failure of the beam
 420 occurred in the area near the position of LVDT 3, leading to higher readings at this point.

421 A comparison between the recorded values of slip of beams U2, D1, D2 for a load value
 422 equal to the maximum load of beam U2 (56.3 kN) is presented in Table 5.

423 **Table 5** Slip measurement for the beams U2, D1, D2 for value of load 56.3 kN

Position	Beam U2	Beam D1	Beam D2
LVDT 2	0.06	0.04	0.01
LVDT 3	0.11	0.01	0.19
LVDT 4	0.357	0.05	0.06
LVDT 7	0.03	0.03	0
LVDT 8	0.18	0.14	0.01

424 The result of Table 5 indicate that the recorded values of slip at beams D1 and D2 were
 425 significantly lower compared to the respective values of beam U2 in almost all the examined
 426 positions. Based on these results, it can be seen that only the recordings of LVDT 3 of beam
 427 D2 were slightly higher than the respective values of U2 beam, which is attributed to the local
 428 failure of the beam at this point.

429 Existing codes [20, 21] set limit values for different performance levels. The Greek
 430 Code of Interventions [20], proposes a maximum slip value of 0.2 mm for the immediate

431 occupancy performance level, 0.8 mm for the life safety performance level and 1.5 mm and the
432 collapse prevention performance level. All the recorded values in the present study are lower
433 than 0.8 mm, which corresponds to the life safety prevention level. It is worth noticing that
434 most recorded values of the strengthened beams with dowels, are lower than 0.2 mm and only
435 in the position of LVDT3 of beam D2, which was affected by the failure of the beam in this
436 area, a higher value was recorded. According to the fib Bulletin 43 [21] on the other hand, a
437 maximum interface slip of 0.2 mm is suggested for the serviceability limit state and 2.0 mm
438 for the ultimate limit state.

439 In the literature there are recorded slip values for concrete to concrete interfaces in case
440 of beams strengthened with conventional concrete layers. Tsioulou et al [22], reported a
441 maximum value of slip of 1.1 mm for conventional concrete to concrete interfaces. In this
442 study, RC beams strengthened with RC layers without the use of dowels at the interface. This
443 value is significantly higher compared to the recorded values of the present investigation for
444 UHPFRC to concrete interfaces and according to Greek retrofitting code corresponds to
445 collapse prevention performance level.

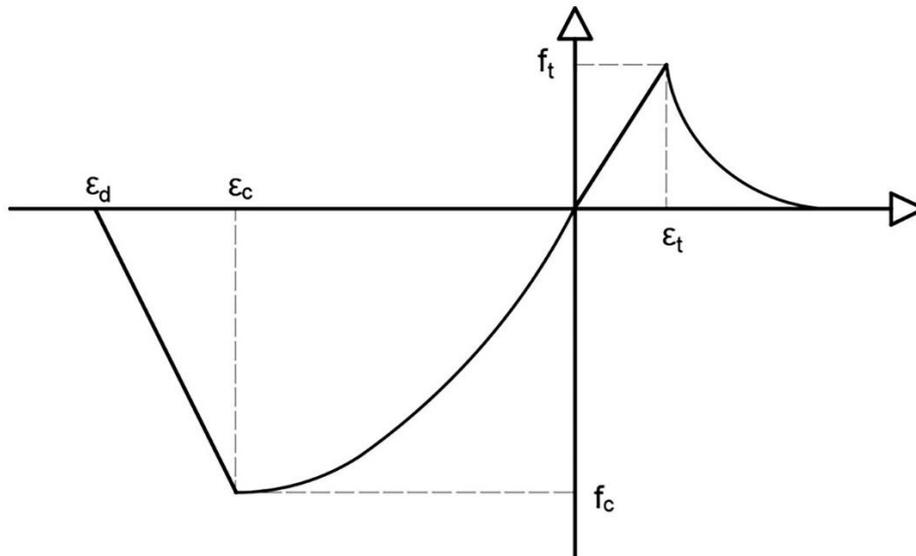
446 The results of the present experimental investigation indicate that the use of dowels at
447 the interface is an effective technique which should be taken into consideration when UHPFRC
448 layers are used for the structural upgrade of existing RC structures. The use of dowels delay
449 the formation of the cracks and result to lower slip values and higher load carrying capacity.

450 **3. Numerical modelling of the examined technique**

451 **3.1 Modelling of the technique**

452 To further investigate the crucial topic of the effect of the interface conditions on the
453 performance of the strengthened elements a numerical investigation has been conducted.
454 Concrete was modelled with an eight-node element using the SBETA constitutive model [23].

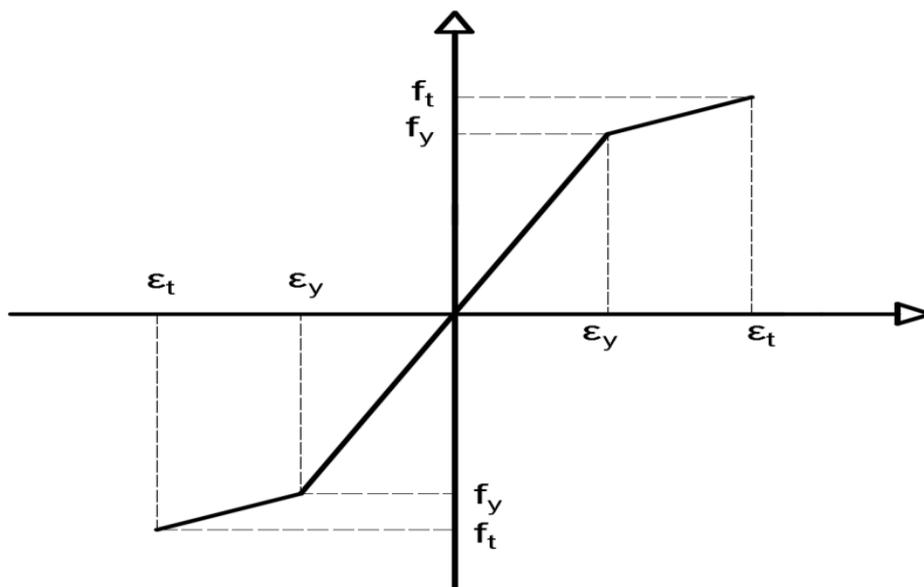
455 In Figure 13, the model used to simulate the behaviour of concrete in tension and compression,
 456 is presented.



457

458 **Fig.13** Constitutive model in tension and compression adopted in ATENA software

459 The steel bars were simulated using linear elements with bi-linear behavior and
 460 hardening, as presented in Figure 14. The properties of steel grade B 500C, according to BS
 461 4449:2005 [24] were adopted for the modelling of the steel bars, while the cover of the steel
 462 bars was the same as the experimental investigation.



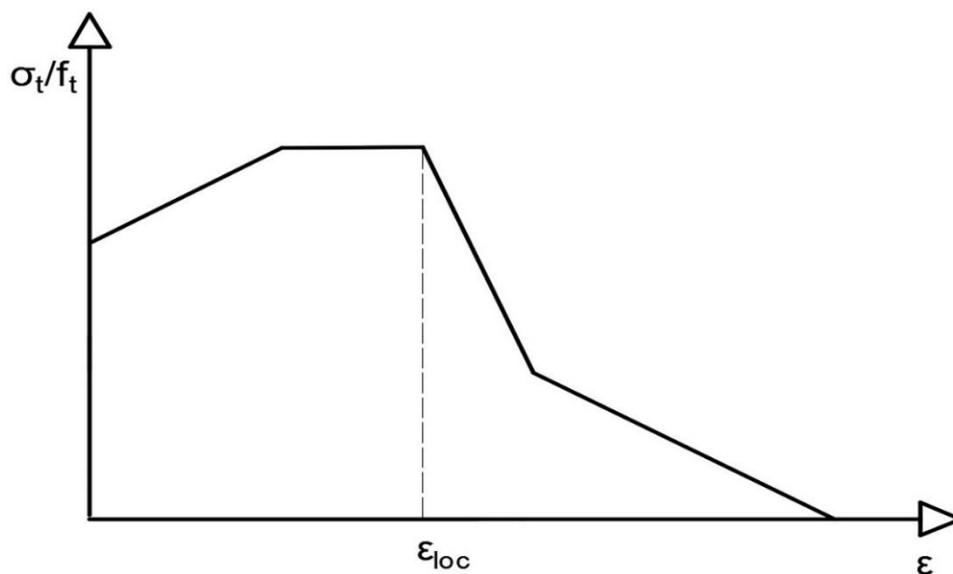
463

464 **Fig. 14** Stress-strain model of the reinforcement

465 The concrete to steel bars bond was also taken into consideration. More specifically,
 466 the CEB-FIB model code 90 [25] bond-slip model was adopted for the numerical modelling.

467 The interface between UHPFRC and concrete was modelled using two dimensional
 468 contact elements with a coefficient of friction equal to 1 and cohesion 1.8 MPa, which were
 469 determined using experimental data from a previously published study [12]. For the numerical
 470 modelling of the strengthened beams with dowels, perfect connection at the interface was
 471 considered.

472 For the modelling of the UHPFRC, the compressive and tensile characteristics obtained
 473 experimentally were adopted. In compression, SBETA constitutive model [23] was used, while
 474 for the modelling of the performance of the material in tension, the average stress-strain results
 475 of Figure 5b were used. Based on these results, the response of the material in tension was
 476 considered linear up to a value of stress level equal to 5 MPa, while a second part, up to the
 477 maximum point, was used to model the strain hardening. After the ultimate stress, there is the
 478 strain softening phase which was modelled with a bi-linear model. The model adopted in
 479 tension is presented in Figure 15.



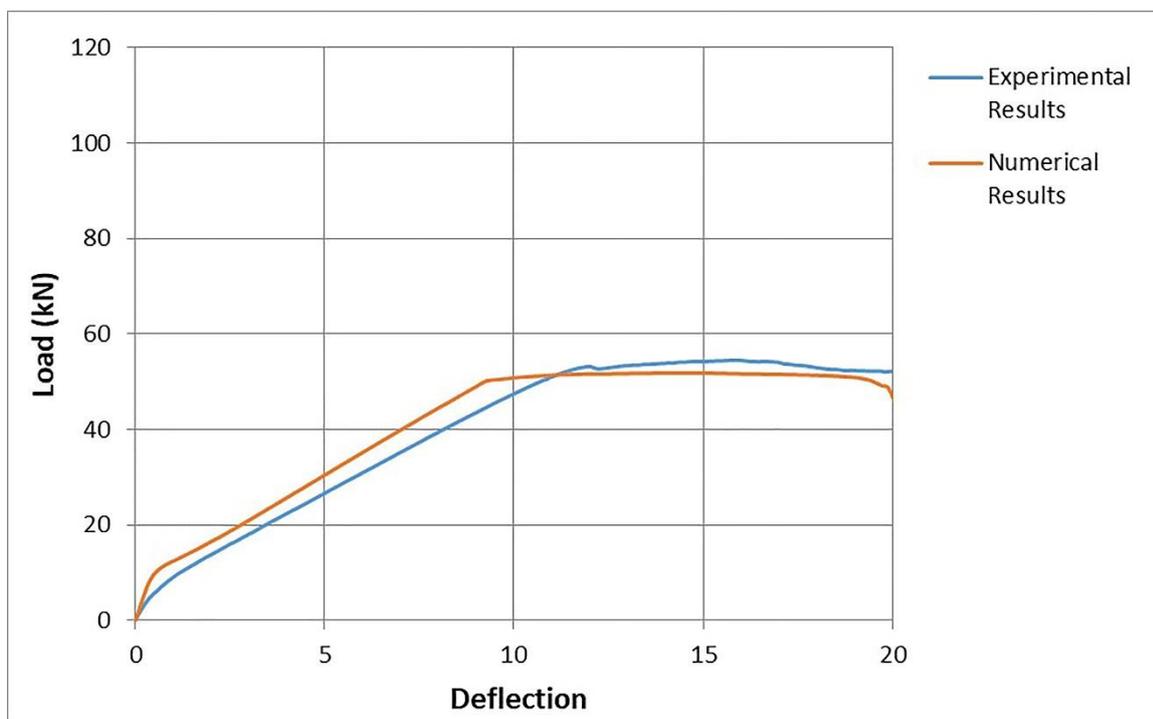
480

481 **Fig.15** Tensile function adopted in ATENA software for the UHPFRC

482 For the modelling of the UHPFRC, the shrinkage was also taken into consideration
483 using a negative volumetric strain value to the UHPFRC elements. Based on existing study in
484 the literature review [3], an ultimate value of shrinkage equal to 565 microstrain was recorded
485 for a fiber content of 3% and this value applied to the UHPFRC layers.

486 3.2 Validation of the numerical model

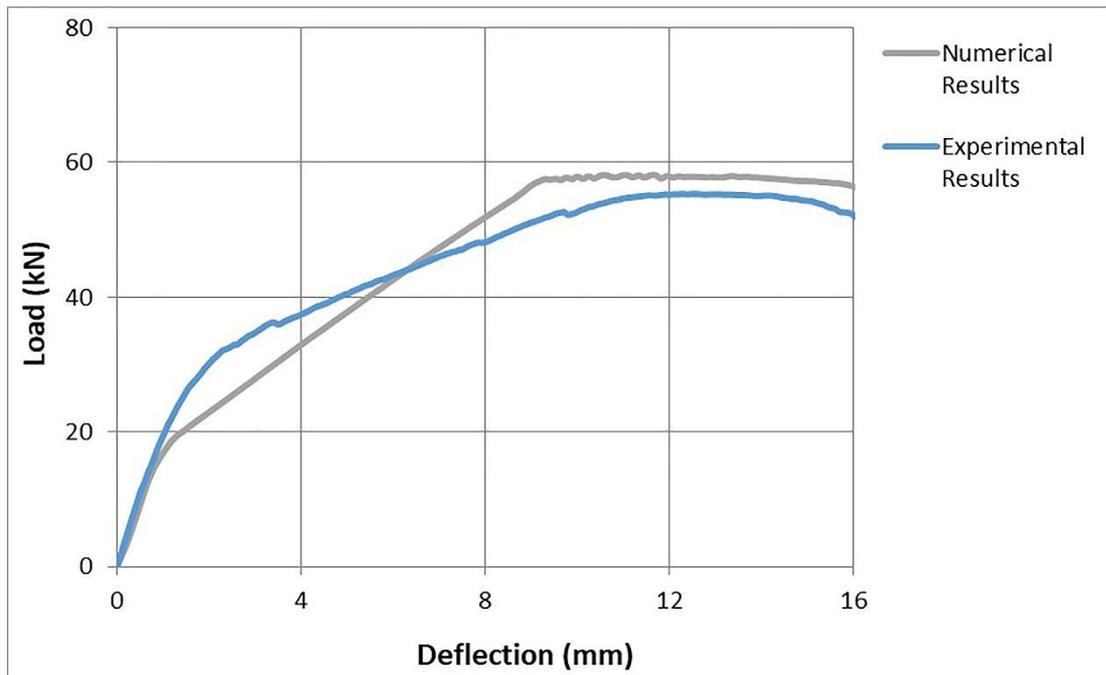
487 A comparison between the average experimental and numerical results of the control
488 beam (P), the strengthened beams with UHPFRC layers without dowels (U), the strengthened
489 beams with dowels (D) and the strengthened beams with jackets ((3SJ) are presented in Figures
490 16a-d respectively.



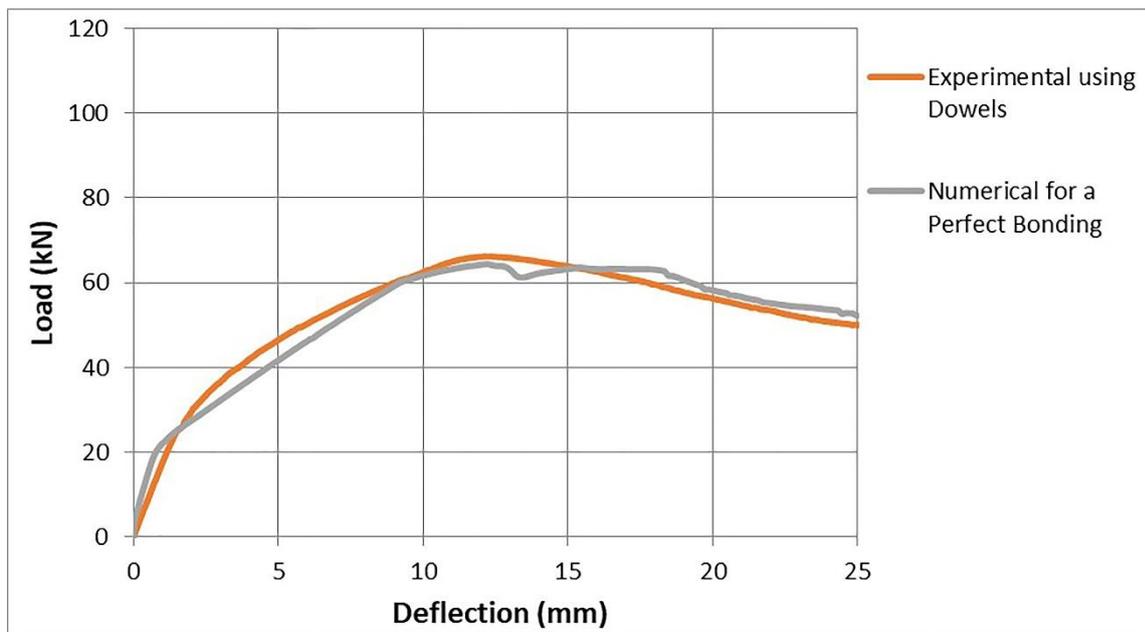
491

492 **Fig.16a**

493

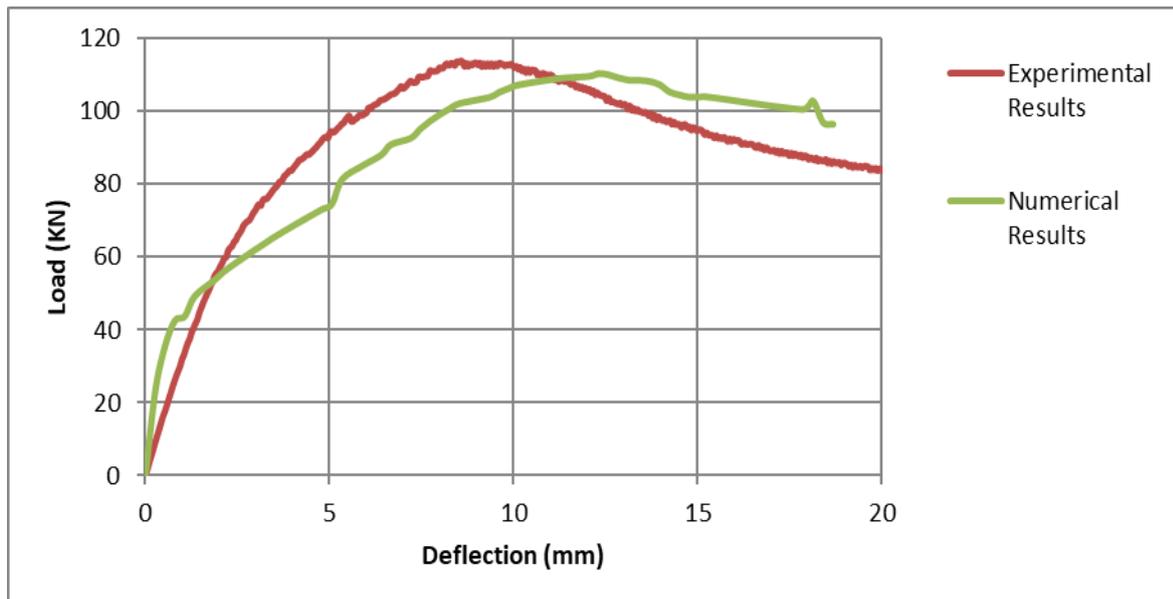


494

495 **Fig.16b**

496

497 **Fig.16c**



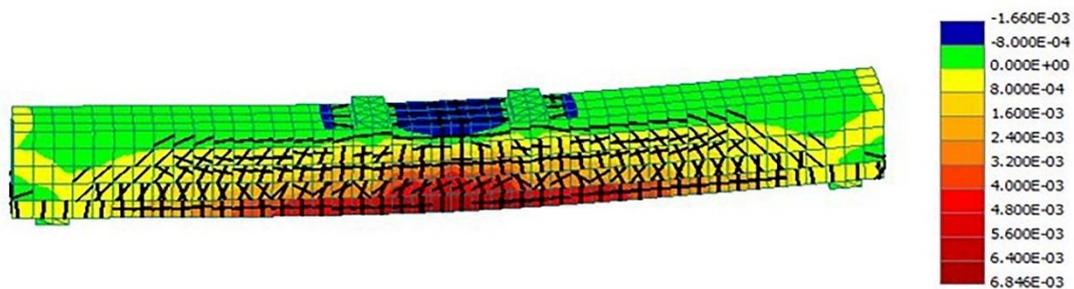
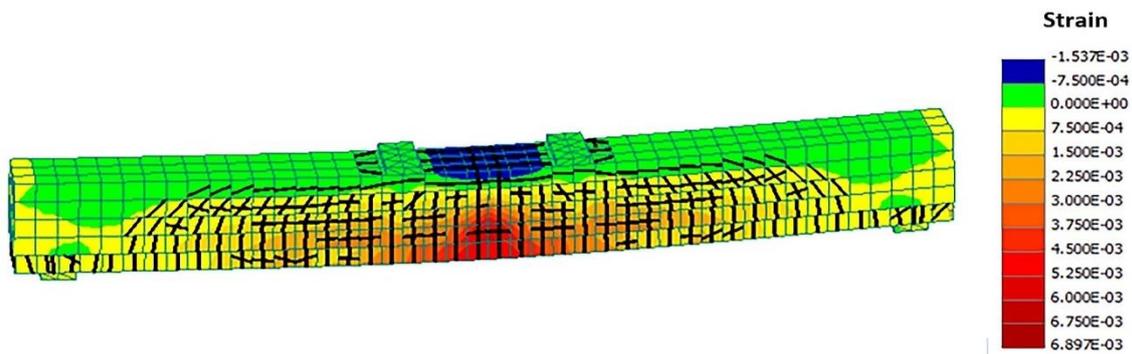
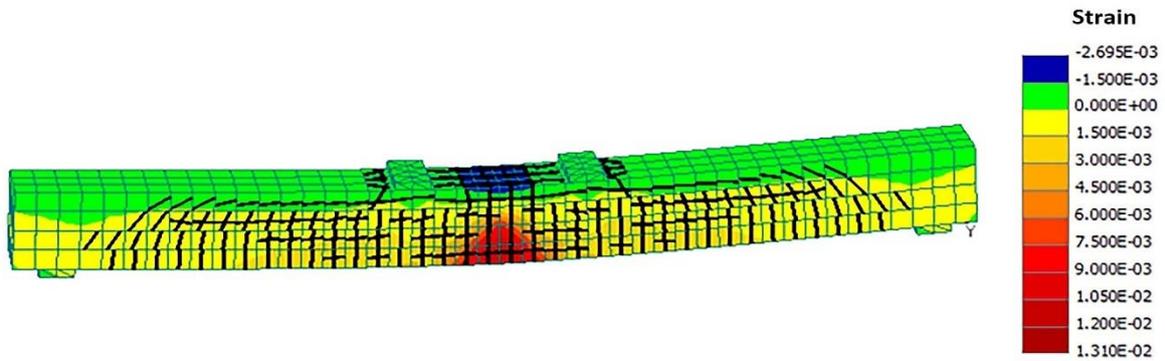
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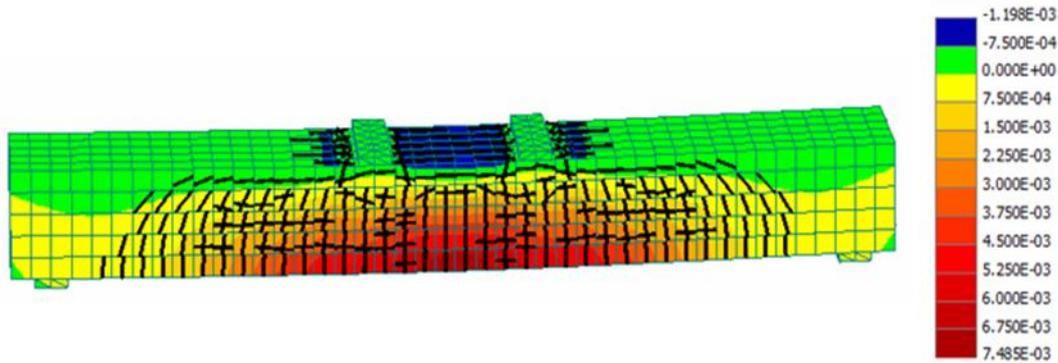
499 **Fig.16d**

500 **Fig. 16** Experimental versus numerical results for a) the control beam P b) the beam
 501 strengthened with layers U c) the beams strengthened with beams and dowels D d) the beams
 502 strengthened with jackets

503 The results of Fig. 16a indicate that there is a very good agreement between the
 504 experimental and the numerical results for the control beams. For the strengthened beams with
 505 layers (Fig. 16b), the use of contact elements with coefficient of friction equal to 1 and cohesion
 506 equal to 1.8 MPa leads to numerical results very close to the experimental. In case of the
 507 strengthened specimens with dowels (Fig. 16c), the assumption of perfect bonding at the
 508 interface of the beam of the numerical model shows a remarkable agreement with the beam of
 509 the experimental investigation with the addition of dowels. Finally, a good agreement between
 510 the experimental and the numerical results can be observed for the beams strengthened with
 511 jackets. The load carrying capacity of the numerical model (110 kN) is very close to the
 512 respective experimental value (114.5 kN), while slightly higher stiffness is obtained
 513 numerically.

514 The failure mode of the examined numerical models at the point where the load capacity
 515 has been reduced to 80% of the maximum load, is presented in Figures 17a-c.





522

523 **Fig.17d**

524 **Fig. 17** Failure mode of a) control beam b) beam strengthened with UHPFRC layer c) beam
 525 strengthened with UHPFRC layers and perfect bonding d) beam strengthened with Jacket

526 The results of Figure 17 indicate that the crack patterns of the numerical models are in
 527 a very good agreement with the failure mode observed from the experimental investigation,
 528 with high values of strain concentration at the tensile side of the beams and localization of the
 529 damage in the middle of the span. Also, in Figures 17a-c high values of strain at the
 530 compressive side can be distinguished, similar to the experimental observations where failure
 531 at the compressive side occurred (Figures 10a-g). From the experimental results it is evident
 532 that the use of dowels results in better bonding between UHPFRC and RC with lower values
 533 of slip, while the formation of cracks is delayed. From Figure 16c and the comparison between
 534 the numerical results for monolithic connection and the experimental results using dowels, it
 535 is clear that when dowels are used, the response of the strengthened beams approaches the
 536 respective monolithic of the beam of the numerical investigation, as assumed for the
 537 application of the model. In addition, as can be seen in Figure 17c, for a monolithic connection,
 538 the strain is distributed along the whole length of the beam and is not concentrated on a small
 539 area of the examined specimens. This allows the material of the layer (UHPFRC) to be better
 540 utilized and the crack localization to be delayed leading to higher load capacity and enhanced

541 structural performance, highlighting the beneficial effect of the use of dowels at the UHPFRC-
542 to-concrete interfaces.

543 **4. Conclusions**

544 The present research focused on the investigation of new techniques for the application
545 of UHPFRC as strengthening material without the use of additional steel bars in the
546 strengthening layers, aiming to offer improved structural performance. The first examined
547 technique focused on the investigation of the effectiveness of dowels at the interface between
548 UHPFRC and concrete for the strengthening of RC beams and the second technique
549 investigated the performance of strengthened beams using three-side UHPFRC jackets.

550 Extensive systematic experimental work has been conducted followed by a numerical
551 study focused on the effect of the interface conditions. The following conclusions can be
552 drawn:

- 553 • The contribution of UHPFRC layer (without dowels) can be efficient for the serviceability
554 limit state. The experimental results of the present study recorded high values of slip after this
555 state, which can lead to debonding and lower load carrying capacity.
- 556 • Dowels at the interface between UHPFRC and RC result in better bonding, with lower
557 interface slip values preventing de-bonding. Also, dowels delay the formation of cracks.
- 558 • The addition of dowels at the interface leads to enhanced load carrying capacity of the
559 strengthened beams. The experimental results of the present study indicated an increase of
560 21.5% of the load carrying capacity when dowels were used at the interface. The respective
561 increase without the use of dowels was only 1.5%.
- 562 • The addition of dowels at the interface leads to better distribution of the stresses along the
563 length of the strengthened elements.

- 564 • Superior performance between the examined techniques was achieved for strengthening
565 with three-side jackets. In this case, the load carrying capacity was increase by 110% and the
566 stiffness by 270%.
- 567 • A remarkable agreement was identified from the comparison of the experimental results
568 using dowels with the numerical results considering monolithic connection at the interface
569 between UHPFRC and RC.

570 From the results of the present study it is clear that both, the addition of dowels and
571 jackets, are effective techniques which can improve the performance of the strengthened
572 elements. The selection of the appropriate technique should be depended on the desired
573 outcome of the technique to increase the strength, the stiffness or both.

574 The use of dowels is a technique which should be taken into consideration to reduce
575 the effect of imperfections during the roughening of the interface and to eliminate the risk of
576 premature de-bonding of the strengthening layer. The construction of jackets on the other hand,
577 should be preferred in heavily damaged RC members or in cases where the load carrying
578 capacity and stiffness need to be significantly increased.

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