EXPERIMENTAL ANALYSIS ON THE SHEAR BEHAVIOUR OF RC BEAMS STRENGTHENED WITH GFRP SHEETS

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1 INTRODUCTION

Nowadays composite sheets are widely adopted in the flexural and shear strengthening of structural elements. However, many questions still require a better understanding. One of the most important concerns the shear behaviour of beams strengthened by composite sheets and especially the performance of the composite reinforcement in terms of strain and stress levels, contribution to the beams capacity and efficiency of the strengthening intervention. This is a point which requires more experimental studies not only for a more advanced comprehension of the shear response but also for the possibility to develop a theoretical approach to predict the ultimate shear capacity of the beams and the composite contribution.

This paper presents the results of an experimental investigation carried out on reinforced concrete beams strengthened in shear by glass fibre composite sheets. The geometry of the beams was chosen to have a shear span to depth ratio equal to three in order to reduce the interaction between the arch and beam shear resisting mechanisms which is usually significant in the behaviour of beams with a shear span to depth ratio less than three. The tested beams were completely wrapped by GFRP sheets in order to avoid early failure modes of debonding or peeling-off and, hence, allow to evaluate the maximum increment of shear strength due to composite. The longitudinal and transversal steel arrangement was chosen in order to have a shear failure of beams and avoid a flexural collapse in both the unstrengthened and strengthened beams.

The present study aims to evaluate the ultimate shear capacity of the beams and the composite shear strength contribution. To this purpose the paper focuses on examining the response of the composite sheets in terms of stress and strain fields.

2 EXPERIMENTAL PROGRAM

Eight reinforced concrete beams of dimensions 150mm x 350mm x 3200mm were tested. Two of them, named UNa and UNb, were the unstrengthened reference beams and the others six were strengthened in shear by GFRP sheets. The geometry of the beams was designed in order to have a shear span to depth ratio equal to 3. The arrangement of the longitudinal and transversal steel bars is shown in Figure 1. The anchorage device of the bottom bars is a steel plate whose dimensions are reported in the same figure.

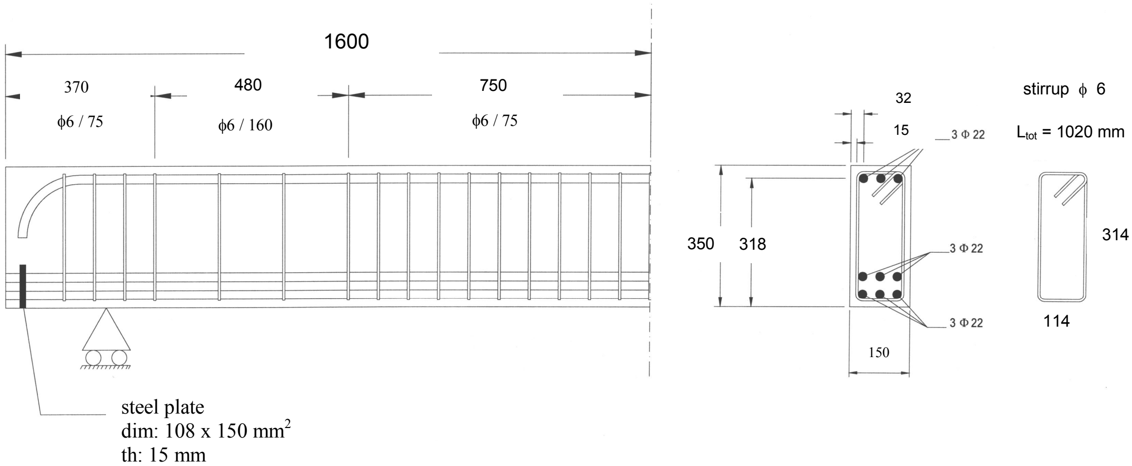


Figure 1: Beams geometry and steel reinforcement arrangement

The six GFRP reinforced beams were collected in tree couples of beams obtained by applying, respectively, one (ST1a and ST1b), two (ST2a and ST2b) and three (ST3a and ST3b) sheets wrapped around the lateral surface throughout the shear span (Figure 2).

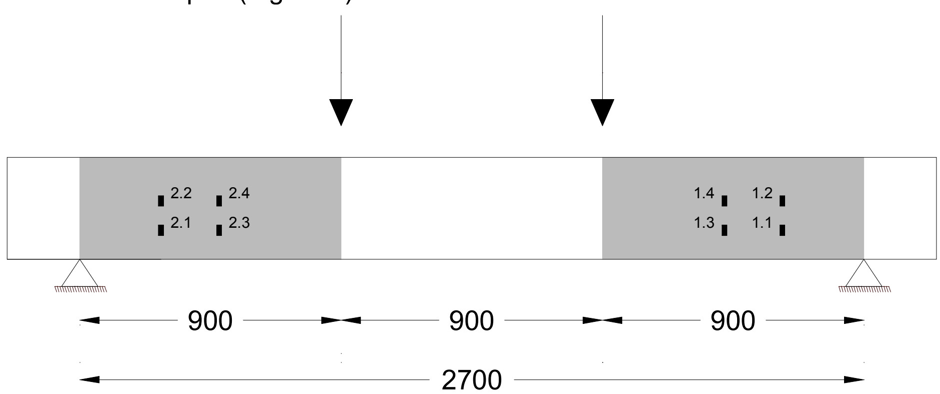


Figure 2: The wrapped zone of the beams and the positions of the strain gauges

Beams ST1b, ST2b, ST3a and ST3b were reinforced by GFRP sheets which were completely adhered to the lateral surface while beams ST1a and ST2a were strengthened by sheets which were partially bonded to the lateral surface according to the scheme of Figure 3. This last arrangement allowed to analyse the influence of the adhesion of the sheets to the lateral surface of the beams. In Figures 2 and 3 the grey filled zones represent the part of lateral surface where primer and putty were applied.

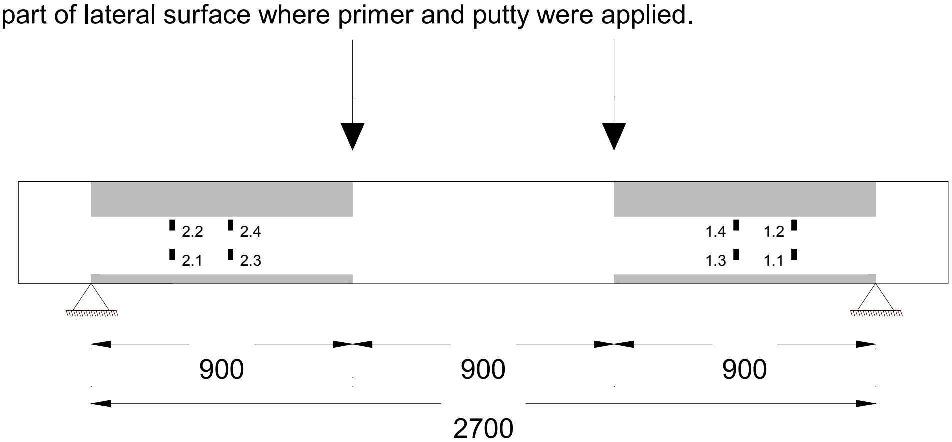


Figure 3: The scheme of the partial bonding of GFRP in beams ST1a and ST2a

As shown in Figure 4 the lateral surface placed between the upper and the lower grey zones of Figure 3 was covered by a transparent plastic sheet to avoid any contact between GFRP and concrete. In this picture is shown a detail of the lateral surface of the beam ST1a when the first GFRP sheet is applied.

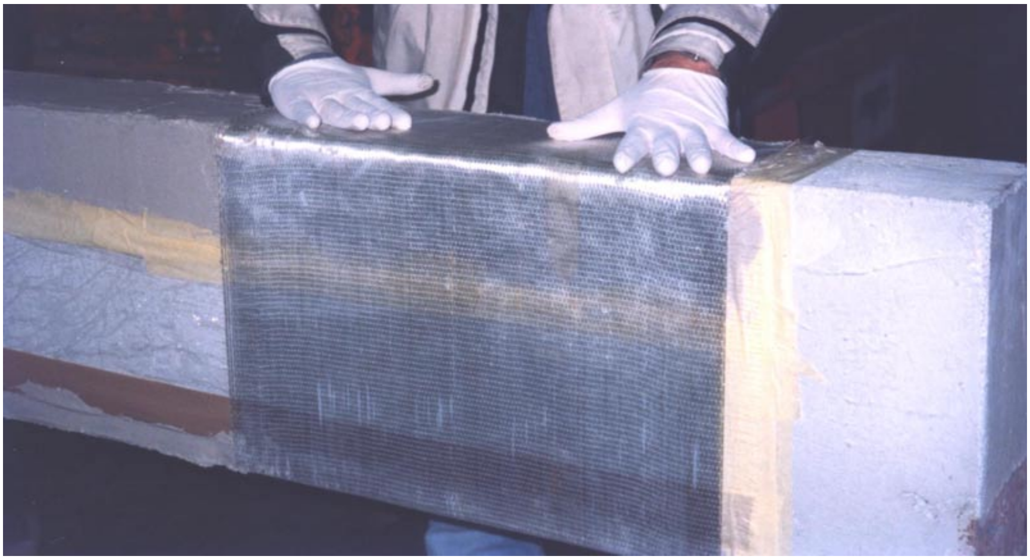


Figure 4: Beam ST1a: the preparation of the lateral surface and the application of the GFRP sheet

The same concrete composition was used for all the eight beams. The concrete strengths at the age of testing, obtained by four cubic specimens, are given in Table 1.

Table 1: Mechanical and geometrical properties of the concrete cubic specimens

specimen n.	1	2	3	4
dimensions of the cross section (mm x mm)	150,3 x 150,3	150,3 x 150,3	150,5 x 150,3	150 x 150,3
Height (mm)	15	15	15	15
Area of the cross section (mm^2)	22575	22575	22613	22538
Weight (kg)	7,65	7,73	7,63	7,67
R_c (MPa)	38,6	40,1	41,9	43,5
type of failure	bipiramidal	bipiramidal	bipiramidal	bipiramidal

The average cubic strength equals 41.03 MPa. The steel used for the internal stirrups and the longitudinal bars had a yield strength of 494.5 MPa and an ultimate strength of 621 MPa. The strengthening material was a glass fibre sheet whose properties are reported in Table 2

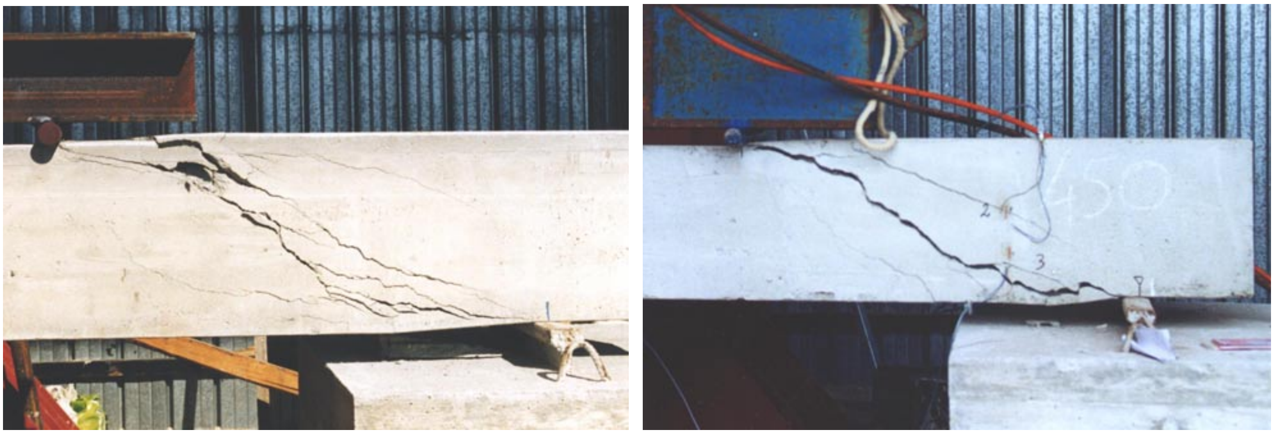
Table 2: Properties of glass fibres

Specific gravity (g/cm^3)	2.56
Single fibre tensile strength (GN/m^2)	3.6
Young modulus of elasticity (GN/m^2)	75.9
Softening point (°C)	850
Coefficient of thermal expansion (°C)	4.9 *10^{-6}
Coefficient of thermal conductivity (W/m°C)	1.04

The equivalent thickness of a single sheet was about 0.12 mm. The ultimate strain was equal to 4.74%. The GFRP sheets were applied according to the specifications of the supplier after the surface preparation (primer and putty application). In the shear span the edges of the beams were rounded. The beams were tested in four points bending according to the schemes reported in Figure 2 and 3.
During the test loading, beam deflections and vertical strains of the sheets were measured. The electric resistance strain gauges of 20 mm gauge length, placed at the positions shown in Figure 2, were excited at 1.25 V. Each excitation channel incorporated remote sensing circuitry to compensate for voltage drops due to lead resistance. Then the programmed excitation level is applied to the strain gauges.

3 TEST RESULTS

All the beams failed in shear. The crack pattern at failure was almost the same for the eight beams. In the shear span one or two mayor cracks together with many secondary cracks formed. The mayor crack propagated as the load increased up to the beams failure which was always due to concrete crushing at the compressed side of beams.



a) b)

Figure 5: The unstrengthened beams at collapse: a) beam UNa; b) beam UNb

Figure 6: The beam ST1b at collapse

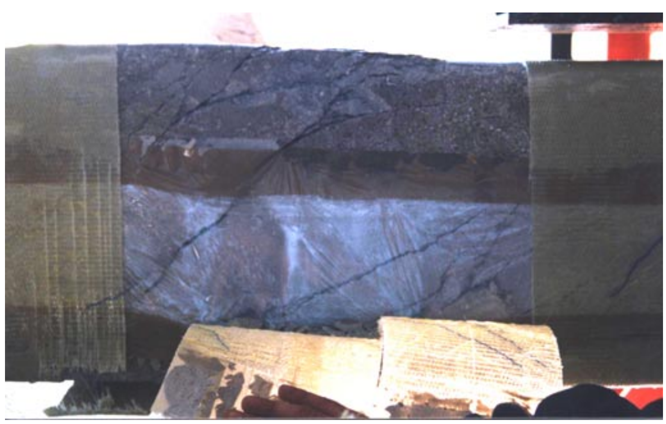


Figure 7: The beam ST2a at collapse

In Figure 5 are shown the unstrengthened beams UNa and UNb at collapse which occurred at a total vertical load equal to 304 and 372 KN respectively.

a) b)

Figure 8: The beam ST3a at failure: a) front view b) back view

a) b)

Figure 9: The beam ST3b at failure a) front view b) back view

Figures 6-9 show the beams ST1b, ST2a,ST3a and ST3b at shear failure; in every beam, at the crushing zone, soon afterwards the concrete failure, the GFRP sheets broke at the upper edge of the beam. Only the beam ST1b crushed with a sudden burst.

The crack patterns of the beams ST1a, ST2a and ST3b were characterized by closer and thinner cracks than the beams ST1b, ST2b and ST3a. It seems, but more research is needed to verify this statement, that the beams with partially bonded sheets fail with a more dense distribution of cracks at failure.

In Figure 10 are reported the curves of the load versus the midspan deflection of the beams UN1, ST1a and ST1b. These curves show a large increment of the shear strength and a small increase of the initial stiffness of the beams. The collapse load for the beams ST1a and b was almost the same and no significant difference in the global behavior of the two beams was observed. The same qualitative results were obtained for beams ST2a and b.

In Figure 11 is shown the comparison between the load-displacement curves of the beams UN2 and ST3a and b. These beams where subjected to a loading-unloading cycle which caused shear cracking. The curves UNb and ST3a show a small reduction in stiffness because of the cracking at a load approximately equal to 130 KN. The stiffness decrease in the beam ST3b occurred about at 85 KN; the beam collapsed at a load lower than the beam ST3a. This behavior was probably due to a different concrete strength in the beams.

Figure 12 shows the time history of the fibres vertical strain measured at the eight strain gauges of the beam ST3a whose position is illustrated in Figure2. The figure shows that the vertical strain in the fibres varies between 0.98 and 1.8 mε; then, at beam collapse, the fibre strain is always below the ultimate tensile strain of the fibres with the exception of the strain gauge 1.4 which suddenly reached the full-scale value before the failure.

The bottom strain gauges of both left (2.1 and 2.3) and right (1.1 and 1.3) side of the beam measured strain values lower than the corresponding upper strain gauges.

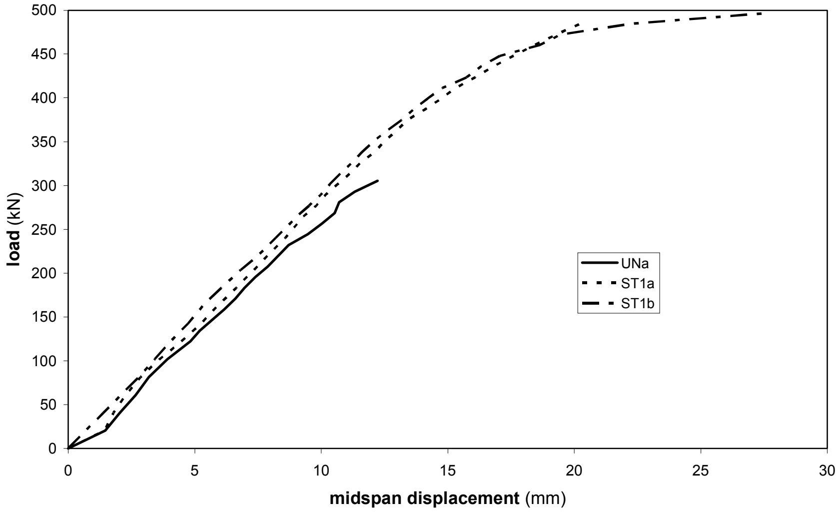


Figure 10: Load - midspan deflection curves: beams UNa, ST1a and b

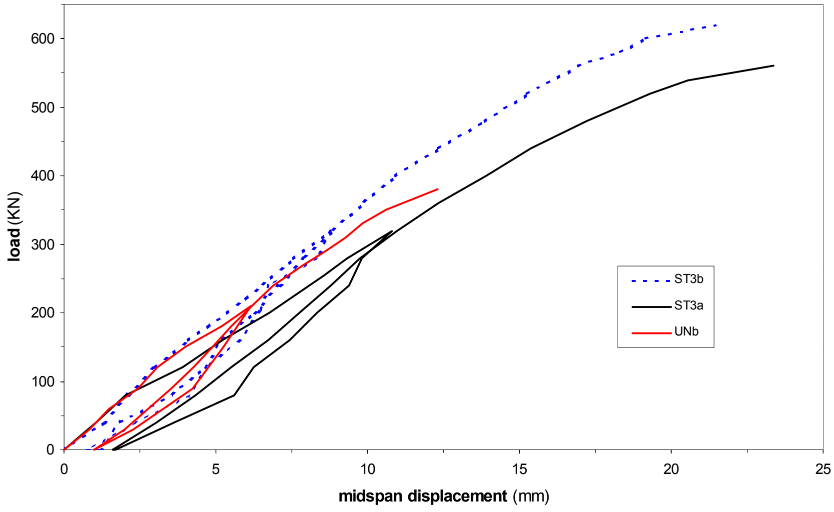


Figure 11: Load - midspan deflection curves: beams UNb, ST3a and b

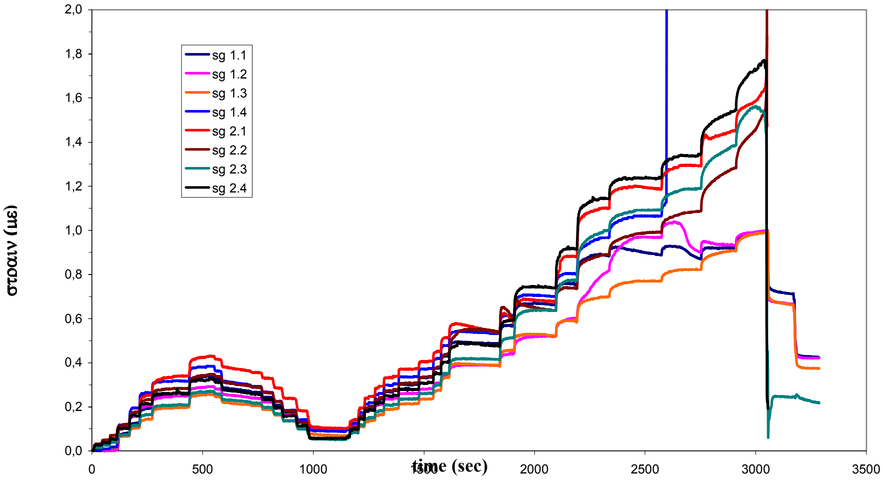


Figure 12: Beam ST3a: time-history of vertical strain measured at the eight strain gauges of the beam ST3a

The fact that often the FRP sheets do not reach their ultimate capacity has been already shown in many papers (see for instance [1, 2, 3, and 4]). Moreover the strain level measured in this experimental work was always below the strain value at collapse suggested by many authors in the case of CFRP sheets.
In Figure 13 and 14 are reported the curves of load versus vertical strain in the fibres, measured respectively at the right and left front side of the beam ST3a.

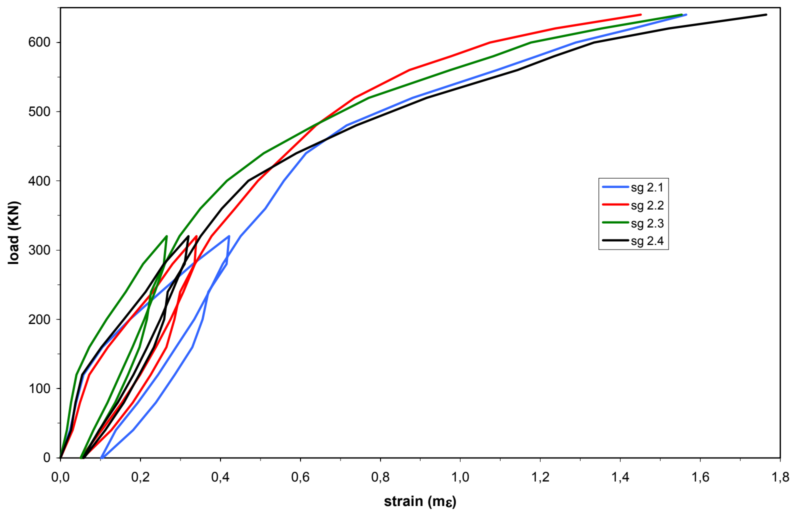


Figure 13: Beam ST3a: load versus vertical strain in the GFRP fibre: strain gauges 1.1÷1.4

Figures 12, 13 and 14 show that the strain values measured at the four strain gauges placed on the left front side where failure occurred were larger than those measured at the right side.
The last two figures show that, in analogy to the behavior of the steel stirrups, the fibers strains rose significantly at a vertical load approximately equal to 130 KN which was the value of the load causing diagonal cracking.

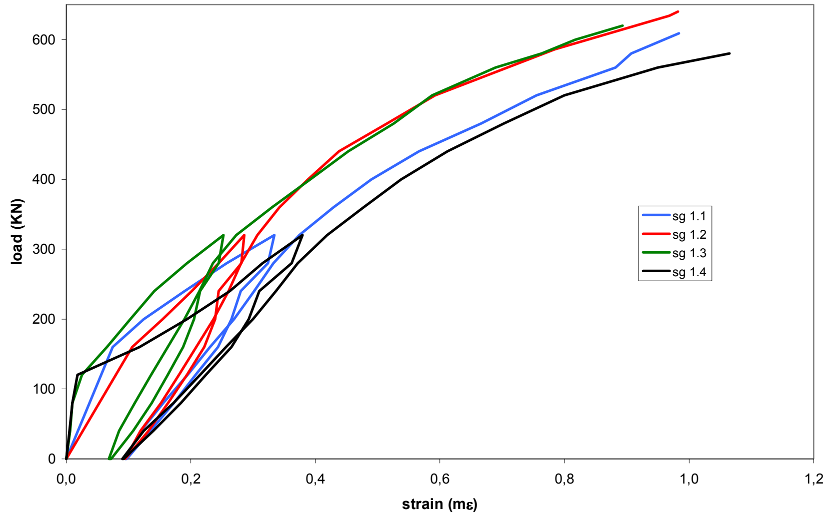


Figure 14: Beam ST3a: load versus vertical strain in the GFRP fibre: strain gauges 2.1÷2.4

The values of the ultimate shear strength, V_u, of the beams, the value of the midspan displacement at failure, v_u, and the range of measured strain in the sheets, ε_u, are given in Table 3. The vertical strains in the sheets of beams ST2a and b were not measured because the data acquisition system did not work properly.
The sheets contribution to the shear strength, $V_{frp,ex}$, evaluated by steel stirrups like formula with the maximum values of the measured vertical strain, was lower than the difference, ΔV_u, between the ultimate shear V_u of the wrapped and unwrapped beams. This result was probably due to the presence of the arch resisting mechanism which did not vanish with the designed geometry of the beam. In this framework the role of the anchorage device of the bottom bars needs further experimental investigation.

Table 3 Summary of the experimental results

<table>
<tr><td rowspan="3"></td><td colspan="2" rowspan="2">Unstrengthened beams</td><td colspan="12">Strengthened beams</td></tr>
<tr><td colspan="4">1 sheet</td><td colspan="4">2 sheets</td><td colspan="4">3 sheets</td></tr>
<tr><td><u>UNa</u></td><td><u>UNb</u></td><td colspan="2"><u>ST1a</u></td><td colspan="2"><u>ST1b</u></td><td colspan="2"><u>ST2a</u></td><td colspan="2"><u>ST2b</u></td><td colspan="2"><u>ST3a</u></td><td colspan="2"><u>ST3b</u></td></tr>
<tr><td>V_u (KN)</td><td>152</td><td>186</td><td colspan="2">242</td><td colspan="2">242</td><td colspan="2">278</td><td colspan="2">270</td><td colspan="2">318</td><td colspan="2">279</td></tr>
<tr><td>$\varepsilon_{u,frp}$ ($m\varepsilon$)</td><td>/</td><td>/</td><td colspan="2">1.24÷1.81</td><td colspan="2">1.13÷1.65</td><td colspan="2">-</td><td colspan="2">-</td><td colspan="2">0.98 ÷1.77</td><td colspan="2">0.90÷1.33</td></tr>
<tr><td>v_u (mm)</td><td>12.16</td><td>12.31</td><td colspan="2">20.14</td><td colspan="2">24.94</td><td colspan="2">25.74</td><td colspan="2">30.50</td><td colspan="2">21.49</td><td colspan="2">23.37</td></tr>
<tr><td>$V_{frp,ex}$ (KN)</td><td>/</td><td>/</td><td colspan="2">9</td><td colspan="2">8</td><td colspan="2">-</td><td colspan="2">-</td><td colspan="2">26</td><td colspan="2">19</td></tr>
<tr><td>ΔV_u (KN)</td><td colspan="2">/</td><td>88</td><td>54</td><td>88</td><td>54</td><td>122</td><td>88</td><td>122</td><td>88</td><td>166</td><td>132</td><td>127</td><td>93</td></tr>
</table>

4 CONCLUSIONS

In this paper we presented the results of an experimental investigation carried out on beams reinforced by GFRP sheets and finalised to evaluate the ultimate shear capacity of these elements and the composite contribution to the shear strength. The shear spans of the beams were entirely wrapped with these composite sheets and the collapse loads, the fibre strain and the midspan displacement were measured.
The results showed the effectiveness of the GFRP sheet to enhance the shear capacity of the beams. The increase in shear strength of the beams was between 30 and 109%.
The strain levels in the fibres were much lower than the ultimate strain values of the sheets and the FRP contribution to the shear strength was lower than the strength increment in every beam. This suggests the hypothesis that, with reference to the geometry of beams analysed in this work, the ultimate shear capacity is provided not only by a beam resisting mechanism but also by an arch mechanism. Indeed clearly, even if the number of the experimental tests performed in this work is quite limited, it is highly probable that shear span to depth ratio was not enough large to make the arch mechanism contribution negligible
Further investigations will be carried out in order to analyse the effect of the shear span to depth ratio and the role of the anchorage devices used for the bottom steel bars.

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