**Performance of Shallow Reinforced Concrete Beams with Externally Bonded Steel-Reinforced Polymer**

by Andrea Prota, Kah Yong Tan, Antonio Nanni, Marisa Pecce, and Gaetano Manfredi

The application of steel-reinforced polymer (SRP) composites in structural strengthening is a new concept based on the use of high-strength steel cord. This paper presents the results of an experimental program on the flexural behavior of reinforced concrete (RC) beams strengthened with SRP, including the performance of epoxy resin versus cementitious grout to impregnate and bond SRP to concrete, as well as the feasibility of nailing the SRP to prevent peeling. The use of cementitious grout is highly relevant as it could overcome the issue of fire resistance and further reduce the cost of the strengthening system. Test results were compared to those from beams strengthened with carbon fiber-reinforced polymer (CFRP) under the same experimental program. This preliminary work shows the high potential of SRP strengthening systems and identifies some critical issues that should be investigated next in order to optimize the effectiveness of the proposed strengthening solution.

**Keywords:** anchorage; epoxy; fibers; flexure; grout; polymer; steel; strength.

**INTRODUCTION**

The steel cord of piano wire used as the reinforcement for radial tires is among the strongest of industrial materials. It comprises twisted pearlite steel filaments that have been strengthened by drawing to an ultra-fine diameter (0.20 to 0.35 mm) and its strength is higher than alloyed steel. The use of steel cord to upgrade steel, wood, or concrete members in both new construction and retrofit applications is an emerging concept in composite reinforcement. This reinforcement is varied between the highly twisted cords, for optimum ductility, and slightly twisted cords, which are more open to allow resin penetration, yet maintain cable-like properties (Hardwire LLC 2002). The shape of the steel cord functions the way the threads act on a screw, forming a mechanical interlock to the matrix, resulting in short development lengths.

The steel cords are coated with either zinc or brass and then aligned to form a steel tape that has very high strength and stiffness and is economical to produce (Hardwire LLC 2002). The density of the steel tape ranges from 1.6 to 9.0 cords per centimeter to meet the requirement of reinforcement, viscosity of resin, and cosmetic application. No special resin is required for wetting steel cord reinforcement, as is required for glass and carbon fiber where fiber sizing plays a critical role. Once the steel tape is impregnated with resin and turns into steel-reinforced polymer (SRP), it is well protected and it is expected to have satisfactory corrosion resistance (Tashito et al. 1999); however, this aspect requires more careful investigation.

Huang et al. (2004) reported on a series of ASTM standard tests of representative SRP specimens. The work included a comparison between theoretical and experimental results. They found that the tensile and compressive moduli in the direction of the steel cord, the in-plane shear modulus, and the tensile axial strength could be accurately predicted by mechanics of materials using micromechanical models. The transverse tensile modulus and Poisson’s ratio could also be estimated analytically, though with less accuracy. The transverse compressive modulus could not be accurately determined from micromechanics.

Experimental studies have been carried out on the use of fiber-reinforced polymer (FRP) systems for flexural strengthening (Fanning and Kelly 2001; Breña et al. 2003; Shin and Lee 2003). No systematic testing has been conducted yet on concrete elements strengthened using SRP laminates. To investigate the flexural behavior of reinforced concrete (RC) beams strengthened with SRP composites, two different types of steel tape with medium and high densities, respectively, were used at University of Naples, Italy, to strengthen seven RC beams using cementitious grout and epoxy resin and tested to failure under a quasi-static loading. Arrays of nail anchors were used on two of these beams to fasten the steel tape adhered with cementitious grout to prevent peeling. Two additional RC beams, strengthened with a comparable amount of unidirectional carbon FRP (CFRP) laminates, were tested and compared with those strengthened with SRP composites.

**RESEARCH SIGNIFICANCE**

The research demonstrates the feasibility of strengthening RC beams using externally bonded SRP and represents a first step toward the development of a novel strengthening material system for structural upgrade.

**EXPERIMENTAL PROGRAM**

A total of 11 RC shallow beams, 400 x 200 x 3700 mm in size, were cast. The stirrups were 8 mm-diameter steel bars spaced at 100 mm center-to-center. For all specimens, two 8 mm-diameter steel bars were used as compression reinforcement. Five 18 mm-diameter bars were used as tensile reinforcement for the reference beam (Beam U) (Table 1); for the remaining ten, a deficiency in steel reinforcement area (due for example to a construction or design error, or to structural deterioration) was simulated by using five 10 mm-diameter steel bars as tensile reinforcement. Apart from a second beam left as a control specimen (Beam D), the potential of emerging strengthening techniques was assessed by upgrading the nine remaining beams using two different types of steel tape, namely 3X2 cord (Type A) and 12X cord (Type B), and CFRP laminates (Type C) (Table 1).

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All beams were tested as simply supported members, over a clear span of 3.40 m. They were loaded up to failure under a four-point configuration, with a constant moment region of 1.0 m across the midspan. The load was applied through a 500 kN hydraulic actuator and the test was carried out under displacement control.

### Table 1—Test matrix and summary of experimental results

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Tension steel</th>
<th>External reinforcement</th>
<th>Impregnated matrix</th>
<th>No. of plies</th>
<th>Axial stiffness ratio S</th>
<th>Equivalent reinforcement ratio $\rho_{eq}$, %</th>
<th>$F_{adh}$, kN</th>
<th>$F_{cr}$, kN</th>
<th>$\delta_u$, mm</th>
<th>$F_{cr}$, kN</th>
<th>$\delta_u$, mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>U</td>
<td>5Φ18</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>1.87</td>
<td>136.1</td>
<td>13.6</td>
<td>1.7</td>
<td>141.4</td>
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<td>147.6</td>
</tr>
<tr>
<td>D</td>
<td>5Φ10</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>0.58</td>
<td>47.7</td>
<td>9.2</td>
<td>2.5</td>
<td>43.3</td>
<td>25.1</td>
<td>49.3</td>
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<td>Z-3X2</td>
<td>Epoxy</td>
<td>1</td>
<td>0.16</td>
<td>0.66</td>
<td>85.3</td>
<td>20.7</td>
<td>5.9</td>
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<td>0.74</td>
<td>110.5</td>
<td>20.8</td>
<td>4.5</td>
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<td>29.9</td>
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<td>0.32</td>
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<td>20.1</td>
<td>5.87</td>
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<td>B-12X</td>
<td>Epoxy</td>
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<td>0.65</td>
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<td>10.1</td>
<td>1.4</td>
<td>60.4</td>
<td>31.2</td>
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<td>B-12X</td>
<td>Cementitious</td>
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<td>0.14</td>
<td>0.65</td>
<td>80.4</td>
<td>10.4</td>
<td>1.8</td>
<td>60.0</td>
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<td>B-12X</td>
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<td>0.65</td>
<td>80.4</td>
<td>11.5</td>
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</tr>
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<td>0.72</td>
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<td>2.4</td>
<td>108.6</td>
<td>37.0</td>
</tr>
</tbody>
</table>

*Ply width indicated in Fig. 4.
†With anchor nails.

### Design material properties

1. For the traditional materials, the design properties were assumed equal to 30 MPa for the concrete compressive strength and 500 MPa for the yield strength of the reinforcing steel bars.

2. The carbon-fiber ply is a unidirectional fiber system with a density of 300 g/m². The equivalent fiber thickness is 0.167 mm. According to the manufacturer, the ultimate strength and modulus of elasticity related to fiber volume are 3450 MPa and 230 GPa, respectively (Mapei 2000). The epoxy used to impregnate the dry carbon fibers was a two-component, medium-viscosity, gelatinous solvent-free adhesive (Mapei 2000). Table 2 shows the technical data of the epoxy provided by the manufacturer.

3. For the steel cord strengthening material, a more detailed discussion follows. The 3X2 steel cord (Hardwire 2002) is made by twisting five individual zinc-coated wires together—three straight filaments wrapped by two filaments at a high twist angle (Fig. 1). The density of the 3X2 tape used in this research program consists of 8.7 cords per centimeter, which is considered high-density tape. The 12X steel cord (Hardwire 2002) is made by twisting two different individual brass-coated wires together in 12 strands and then over-twisting one wire around the bundle (Fig. 2). The ridge provided by the wrap wire works to transfer load into the matrix and tighten the cord during the tensile loading. The density of the 12X tape consisted of 6.3 cords per centimeter, which is considered medium-density tape. Table 3 summarizes the geometrical and mechanical properties (tensile strength $f_{tu}$, ultimate rupture strain $\epsilon_{fu}$, and tensile modulus of elasticity $E_0$) of the steel cords. A typical stress-strain curve of SRP tapes is depicted in Fig. 3, where it is shown that this material behaves linearly to failure. Experimental tests have shown that the nonlinear behavior is negligible and there is practically no yielding of the steel. The stress-strain relationship of Fig. 3 was used for design.

A high-performance, two-component, 100% solid epoxy resin (Sika 2000) was used to impregnate and bond the steel tape to the concrete substrate. The technical data of the epoxy resin, supplied by the manufacturer, are shown in Table 2. The cementitious grout (Sika 2000) used to bond the steel tape was a two-component, polymer-modified, pore-sealing mortar with the additional benefit of a penetrating corrosion inhibitor. It has a finishing time of 45 to 60 minutes.
depending on temperature and relative humidity. The technical data of the cementitious grout, supplied by the manufacturer, are shown in Table 4.

### Upgrade strategy

The nominal flexural strength of Beams U and D was computed according to ACI 318-02 recommendations without reduction factors; for the remaining specimens strengthened with either SRP or CFRP, the theoretical calculations were conducted according to ACI 440.2R-02 guidelines (Prota et al. 2004).

As for the strengthening strategy, the design of Beam C-1 was performed to double the capacity of the control specimen, Beam D; and then, the strengthening of Beam C-2 aimed at attaining a flexural strength comparable to that of control specimen Beam U by doubling the CFRP area installed on C-1. To carry out the strengthened beam design, the following parameters were established (Table 1):

1. The axial stiffness ratio $S = \frac{E_{\text{ext}}A_{\text{ext}}}{E_sA_s}$ (with $E_{\text{ext}}$ and $A_{\text{ext}}$ being the elastic modulus and the total area of externally bonded composites and internal steel bars, respectively); and

2. The equivalent reinforcement ratio $\rho_{eq} = \rho_s + \frac{1}{E_{\text{ext}}}E_s$ (where $\rho_s$ and $E_{\text{ext}}$ are the reinforcement ratios of $A_s$ and $A_{\text{ext}}$ over the concrete cross-sectional area computed as the width of the cross section times the depth of the internal reinforcement, respectively).

The amount of SRP to be installed on seven specimens was such to provide values of $\rho_{eq}$ similar to that of Specimens C-1 or C-2. Once the SRP layout was determined with this criterion, the flexural strength of each SRP-strengthened specimen was calculated assuming that ACI 440.2R-02 guidelines (Prota et al. 2004).

Table 1 shows the test matrix of the research program, summarizing the area of internal tensile steel, the type and matrix of the externally bonded reinforcement, the number of plies, and the values of both the $S$ and $\rho_{eq}$ ratios. Considering the adopted test setup, the ultimate load $F_{\text{ult}}$, corresponding to the predicted flexural capacity of each beam, was computed and reported in the next column of Table 1. Figure 4 shows geometric details for all strengthened beams. Seven beams were bonded with steel tapes impregnated with epoxy resin or cementitious grout (Beams A and B); the remaining two beams (C-1 and C-2) were strengthened with CFRP laminates using epoxy resin. Table 1 shows that the design flexural capacity of Beam C-1 was approximately twice that of Beam D, whereas for Beam C-2 it was approximately 90% of the design flexural capacity of Beam U. Two of the beams strengthened with steel tape and cementitious grout were mechanically anchored with nail anchors (B-3 and B-4). The nail anchor selected for this application was a wide-ringed-head nylon anchor with zinc-plated hammer screw (Fig. 5). The anchor is 6 mm in diameter and 60 mm long. A 24 mm-diameter washer was used to enlarge the ringed head of the anchor to obtain a better hold to the SRP.

### SPECIMEN PREPARATION

The bottom face of all beams was sandblasted and cleaned to ensure proper bond before strengthening. No primer was used for bonding SRP tapes with either epoxy or cementitious grout. When a uniform and complete mixing of the epoxy was observed, it was spread to areas where the steel tape had contact. The steel tape was cut to design length and pressed onto the wet epoxy gel with a hard roller. Where two plies of steel tape were used, an additional layer of epoxy was spread and the previously mentioned steps were repeated. The second ply started 100 mm from the cut-off point of the first ply.

For beams bonded with cementitious grout, the same installation procedure was followed. For beams anchored with nail anchors, a total of 31 holes, 60 mm deep and 6 mm in diameter, were drilled in a staggered pattern along two parallel lines, with a center-to-center distance of 200 mm (Fig. 4) before strengthening. After bonding the steel tape with cementitious grout, the anchors were hammered into the holes and locked in with 24 mm-diameter washers.

The procedure for applying the CFRP laminates was as recommended by the manufacturer (Mapei 2000); suggestions provided by ACI 440.2R-02 (ACI Committee 440 2002) guidelines for externally bonded FRP systems were also considered. The surface preparation started with a layer of primer followed by a layer of putty. After the putty had hardened, the carbon-fiber sheet was adhered to the surface with the epoxy; steps similar to those used for the installation of SRP were then followed.
All beams were instrumented to record global and local parameters. The midspan deflection was measured by a vertical linear variable displacement transducer (LVDT). Three horizontal LVDTs were placed on one side of the specimen to record displacements over a length of 0.35 m across the midspan at depths of 5, 55, and 175 mm from the compressive fiber, respectively. On the opposite side, crack width and concrete shortening were measured using demec targets placed 50 mm center-to-center on a total length of 0.55 m at the same depth of the LVDTs on the other side of the beam. Readings were taken at selected load levels. A total of 20 strain gauges were used during each test to measure strains on the externally bonded reinforcement. Depending on width and number of plies, the strain gauge arrangement slightly changed for each beam. In general, some gauges were placed within the constant moment region and some at the cut-off points; longitudinal and transverse strain profiles were obtained.

**TEST RESULTS**

Before testing the beam specimens, characteristics of the traditional materials were verified and found to be consistent with the design assumption. Concrete cubes (with side of 150 mm) showed an average compressive strength of approximately 40.1 MPa. For the reinforcing steel bars (three samples per diameter), average values of 500 MPa, 600 MPa, and 12% were found for the yield strength, the ultimate strength, and the ultimate strain, respectively.

The load-midspan deflection curves of tested beams are depicted in Fig. 6 to 8, which show the trends of each group of beams strengthened with same material systems compared with the response of the two unstrengthened beams. Values of loads and midspan deflections at first cracking ($F_{cr}$ and $\delta_{cr}$), yielding of tensile steel bars ($F_y$ and $\delta_y$) and ultimate ($F_u$ and $\delta_u$) are summarized in Table 1. The first cracking of Beam U occurred at a load of 13.6 kN, while Beam D showed the first crack at a load of approximately 9.2 kN. After first cracking, a loss of stiffness occurred for both beams; curves highlight a change in slope that is more significant for Beam D than for U (Fig. 6). The shapes of the load-deflection curves indicate another loss of stiffness at loads of 141.4 and 43.3 kN for Beams U and D, respectively. This is due to yielding of the tensile reinforcement that occurred at midspan deflections of 35.7 and 25.1 mm, respectively. After these thresholds, the behavior of both beams was characterized by large flexural cracks and then
collapse due to concrete crushing in the constant moment region. Failure loads were equal to 147.6 and 49.3 kN for Specimens U and D; their ultimate behavior was characterized by a ductility factor $\delta_u/\delta_y$ of 1.6 and 4.0, respectively.

The installation of the 3X2 steel tape at the bottom of a Type D beam was beneficial in terms of first cracking (Fig. 6). Regardless of width and number of plies, first cracking of Beams A-1, A-2, and A-3 occurred at a load of approximately 20 kN. A loss of stiffness was then observed; curves show a similar slope for Beams A-1 and A-3, which are less stiff than A-2. Further loss of stiffness was a consequence of yielding of the steel bars; A-1 yielded at 60.3 kN, while A-2 and A-3 reached the yielding at loads of 79.7 and 76.5 kN, respectively. After yielding the slope of each curve reflects the different amount of external reinforcement: A-2 and A-3, having the same amount of external steel tape, provide the same slope and are stiffer than A-1. The mode of failure was similar for the three beams: it was concrete cover separation (Fig. 9 and 10) that initiated at one of the loading points as described in literature (Teng et al. 2001). The minimum ultimate load within Group A beams was provided by A-1 whose failure occurred at approximately 86.3 kN; the maximum load of 121.1 kN was attained by A-2. Tape layout based on the same area as Beam A-2, but arranged on two plies, limited the ultimate capacity of Beam A-3 to 100.4 kN. This specimen exhibited the lower ultimate deflection (that is, 54.5 mm); despite different ultimate strength, A-1 and A-2 showed similar ultimate deflections of 75.7 and 72.4 mm, respectively. The installation of 12X steel tape did not significantly affect the first cracking of Group B beams (Fig. 7), whose cracking loads were in the range of 9.2 to 11.5 kN. Deflections, however, were slightly lower than those of Beam D at corresponding loads (Table 1). The loss of stiffness due to cracking was very similar for Beams B-1, B-2, and B-3; such similarity is also confirmed by very close values of yielding loads ranging between 57.1 and 60.4 kN (Table 1). Beam B-4, having twice the tape area, was stiffer than the other three and yielded at a load of 75.2 kN. The ultimate behavior highlights that Beams B-2 and B-3 failed at loads of 72.7 and 71.5 kN, respectively. This points out that the nails were unable to improve the ultimate performance of Beam B-3, whose ultimate deflection (60.4 mm) was slightly larger than that of B-2 (56.8 mm). The epoxy resin allowed Beam B-1, whose tape area was the same as for B-2 and B-3, to attain its failure at ultimate load and deflection equal to 88.6 kN and 89.2 mm, respectively. Similar strength performance was attained by Beam B-4, whose failure occurred at 86.7 kN. Doubling the tape area enabled B-4 to reach an ultimate strength very close to that of an epoxy bonded beam with half the tape area (Beam B1), but reduced its ultimate deflection to 46.5 mm. The failure of Beams B-1 and B-2 was due to interfacial debonding that initiated at one of the loading points, as previously discussed in the literature.
The epoxy allowed Beam B1 a better engagement of the concrete substrate than that provided by the cementitious grout on Beam B2; this can be observed by comparing Fig. 11 and 12. The failure of Beams B-3 and B-4 was also due to interfacial debonding after nail-bearing failure (Fig. 13).

CFRP laminates increased cracking loads of Beams C-1 and C-2 (13.8 and 15.6 kN, respectively) when compared with reference Beam D (Fig. 8). The loss of stiffness due to cracking was more significant for Beam C-1 than that of C-2, having twice the external FRP area. This determined also that its yield load (108.6 kN) was higher than that of C-1 (75.7 kN). After the yield point, curves of both specimens show further loss of stiffness that is again more significant for Beam C-1 than that of C-2. Both collapsed due to FRP debonding initiated at one of the loading points and is characterized by separation of the concrete cover. Even though Beam C-2 failed at a load approximately 40% higher than C-1, their ultimate deflections were almost identical (55.8 mm versus 55.7 mm).

Important information is also provided by the analysis of strain gauge readings at both midspan and termination of the externally bonded reinforcement of each beam. In this paper, the discussion of local strains is limited to their average values. Average strains of the SRP tape in the constant moment region of Beams A-1 and A-2 were all close to 0.010, whereas an average value of 0.007 was recorded when two plies of 3X2 tape were used (Beam A-3). Beam B-1, whose 12X tape was bonded with epoxy, provided an average strain at midspan equal to 0.012; the average strain recorded on the same tape bonded with cementitious mortar with and without nail anchors (Beams B-3 and B-2, respectively) was between 0.0051 and 0.006. A similar average value (0.005) was recorded when two plies of 12X tape were used (Beam B-4). Ultimate average strains at midspan provided by beams strengthened with two plies (Beam C-1) and three plies (Beam C-2) of CFRP laminate were equal to 0.007 and 0.006, respectively.

Readings provided at beam failure by strain gauges installed at the termination of the externally bonded reinforcement were less homogeneous than those obtained at midspan due to the well-known effect of stress concentration at the termination of the plates (Cosenza and Pecce 2001). In general, at the end of beams bonded with the 3X2 tape (Type A beams), average ultimate strains were on the order of 0.0001 with peak values up to 0.0005. Average strains on the order of 0.00007 were recorded at the end of 12X tapes (Type B beams) with peaks up to 0.0003. Average strains on the order of 0.0002 were given by CFRP laminates with peak values up to 0.002.

**DISCUSSION**

The analysis of the test results is conducted first with respect to beams strengthened with the same external reinforcement (3X2 tape, 12X tape, or CFRP laminate); and then, beams using similar $\rho_{eq}$ with different materials are compared. Remarks on the influence of different reinforcement type and layout on crack widths are also presented while experimental-theoretical comparison in terms of strength, crack widths, and deflections are reported elsewhere (Prota et al. 2004; Ceroni et al. 2004).

For each group of beams strengthened with the same system, the following can be highlighted:

1. Up to the yielding of the internal steel reinforcing bars, the slope of the load-deflection curve of Beam A-3 was very similar to that of A-1, which had an external tape area of only half of that in A-3. Beam A-2, equivalent to A-3 in terms of tape area, exhibited a stiffer behavior before steel yielding. This is also evidenced by the fact that average crack widths were almost identical for Beams A-1 and A-3, but were wider than those exhibited by Beam A-2. Considering that crack spacing was similar for all tested beams and approximately equal to the stirrup spacing (100 mm), outcomes provided by Group A beams suggest that the capability of the externally bonded SRP to reduce crack width and to stiffen the member...
in the preyielding phase is strongly dependent on the width of the external reinforcement rather than on its sectional area (Ceroni et al. 2004):

2. By doubling the width of the 3X2 steel tape, the ultimate strength increased by approximately 40% (A-2 versus A-1), while the ultimate deflection was quite similar. When the same area increase was achieved by doubling the number of plies rather than width (Beam A-3), the strength increased only by approximately 16% compared with Beam A-1 due to a high concentration of interfacial stress. The ultimate deflection was approximately 28% lower due to a lower stiffening effect already observed prior to yielding. Overall, if compared with Beam D, the 3X2 steel tape provided increases of ultimate strength ranging between 75% (A-1) and 145% (A-2), even though the ultimate deflection had reductions ranging between 25% (A-1) and 46% (A-3);

3. No significant stiffening was provided by the 12X steel tape installed on Beams B-1, B-2, and B-3 with epoxy and cementitious grout compared with Beam D prior to yielding. The load-deflection behavior of Beam B-4 was slightly stiffer than Beam D after a load of approximately 25 kN. Such a result suggests that the structure of the 12X tape makes it less stiff than 3X2, and its effectiveness in reducing crack width (Ceroni et al. 2004) and increasing flexural stiffness is negligible;

4. Epoxy resin impregnation allowed Beam B-1 to withstand ultimate load and deflection values that were approximately 23 and 53% larger than those seen in equivalent Beams B-2 and B-3 bonded with cementitious grout, respectively. To attain the strength provided by epoxy resin with cementitious grout, it was necessary to double the area of 12X tape (Beam B-1 versus B-4); however, the ultimate deflection of Beam B-4 was 48% smaller than that of B-1. The use of nail anchors to improve bond of the 12X tape to the concrete surface did not affect strength significantly, but ultimate deflection of Beam B-3 was approximately 6.5% larger than that of B-2. When compared with the strength of Beam D, schemes based on 12X tape increased beam strength between 46 and 79% (B-3 and B-1, respectively), with reductions of ultimate deflection ranging between 13 and 55% (B-1 and B-4, respectively); and

5. The installation of CFRP affected the stiffness of strengthened beams and this was confirmed also by crack width trends (Ceroni et al. 2004). By doubling the area of CFRP, the strength of Beam C-2 was approximately 39% higher than that of C-1; the ultimate deflections were almost identical. If compared with Beam D, the CFRP reinforcement allowed boosting the strength by percentage ranging between 95 and 173%; a reduction of ultimate deflections of 23% smaller than those attained by Beam A-1. A higher stiffness of CFRP laminate compared with that of SRP laminate (resulting in a postyield slope of C-1 steeper than that for A-1, as depicted in Fig. 6 and 8) allowed Beam C-1 to attain an ultimate strength approximately 12% larger than A-1, even though its ultimate deflection was 26% smaller;

2. By comparing the slopes of load-deflection curves of Beams A-2, A-3, B-4, and C-2 (characterized by $\rho_{eq}$ ranging between 0.72 and 0.79) it is observed that Tape 3X2 impregnated with epoxy (A-2 and A-3) was very effective in delaying the first cracking; the CFRP reinforcement had some influence on cracking initiation (C-2), which was not affected by the installation of Tape 12X impregnated with cementitious grout and anchored with nails (B-4) (Table 1). Slopes of branches between first cracking and steel yielding highlight a stiffening effect that was maximum for Beams A-2 and C-2 was lower for Beam A-3, and was negligible in the case of Beam B-4. Such a trend was confirmed also by a comparison in terms of capacity of the externally bonded system to reduce crack widths (Ceroni et al. 2004). Yielding of steel bars for Beams A-2, A-3, and B-4 occurred at similar loads and deflections (Table 1). The yielding of Beam C-2 occurred at load and deflection approximately 41 and 15% higher than these other three beams, respectively. Branches of load-deflection curves after steel yielding are approximately parallel for Beams A-2, A-3, and B-4. Beam C-2 provides a stiffer trend that could be partially due to the slight difference of $\rho_{eq}$ with others (Table 1). The lower bond performance of the cementitious grout affected the strength of Beam B-4, which was 71 and 86% that of Beams A-2 and A-3 bonded with epoxy resin, respectively. Its ultimate deflection was 65 and 85% that of A-2 and A-3, respectively. The influence of stress concentration that limited the ultimate performance of A-3 (two plies) when compared with A-2 (one ply) was already discussed. Beam C-2 provided a strength 11% higher than A-2 with a ultimate deflection 23% smaller. Beams A-2 and C-2 exhibited ultimate strength on the order of 82 and 91% that of Beam U, even though their $\rho_{eq}$ was approximately equal to 49 and 52% that of Beam U, respectively. These data have particular relevance if one considers that for both A-2 and C-2, the full capacity of the cross section was not exploited due to debonding of the externally bonded reinforcement. In terms of ultimate deflections, Beams A-2 and C-2 attained values equal to 1.26 and 0.98 times that provided by Beam U, respectively (Table 1).

CONCLUSIONS

The paper presents an experimental study aimed at assessing the potential of SRP to provide a strengthening system alternative to traditional techniques and to FRP
laminates. SRP-based solutions use improved traditional materials (steel and cementitious grout). This could be advantageous over FRP and overcome its problem areas such as high cost of constituents (fibers and epoxy matrix), fire resistance, low confidence and experience with nontraditional materials, and incompatibility with mechanical anchorages due to stress concentration.

Experimental tests were conducted to assess the structural effectiveness of SRP and evaluate the influence of epoxy versus cementitious matrix; the possibility of using nail anchors to improve the bond of steel tapes impregnated with cementitious grout was also verified. The performance of seven SRP-reinforced beams were compared with that of unstrengthened and FRP-reinforced beams. This preliminary analysis of test results underlined that:

1. Strength increases provided by SRP bonded with cementitious grout were smaller than those obtained using epoxy. CFRP was more effective than epoxy-bonded SRP in terms of strength; the trend was inverted in terms of ultimate deflections. Compared with the unstrengthened beam, SRP allowed attaining strength increases ranging between 46 and 145%, while reductions of ultimate deflections ranged between 13 and 55%. A comparison between beams with equivalent reinforcement ratio highlights that epoxy-bonded SRP tapes provided ultimate strength approximately 10% smaller than CFRP with deflections approximately 24% larger;

2. The epoxy resin was more effective than the cementitious grout in engaging the concrete substrate. Regardless of the type of matrix (epoxy or cementitious), the behavior of equivalent (same area of external reinforcement) SRP-strengthened beams was similar up to yielding of the internal steel. At ultimate, the epoxy SRP ultimate strength and midspan deflection were approximately 23 and 53% larger than those corresponding to the SRP impregnated with cementitious grout;

3. The nail anchors did not improve the performance of the SRP impregnated with cementitious grout. The lack of transverse link in the steel tape did not allow distributing the local stress concentration at anchor location; this determined the local bearing failure of nails that were unable to improve the bond and delay tape debonding;

4. The 3X2 tape affected the global stiffness of strengthened beams and this effect was dependent on the width rather than on the area of the bonded tape. The different macrostructure made the 12X tape unable to provide any stiffening effect. Such trends were confirmed by recorded widths of cracks, whose spacing was very similar for all tested beams; and

5. Strains recorded at failure on the externally bonded reinforcement in the constant moment region indicated that interfacial issues and their influence on failure modes are mainly dependent on the matrix (that is, epoxy versus cementitious) rather than on the type of fiber (steel versus carbon). Strain values were consistent when epoxy was used to bond the 3X2 tape (Type A beams), the 12X tape (Beam B-1), and CFRP (Type C beams). Average values of approximately 0.010, 0.007, and 0.006 were found for one, two, and three plies, respectively. When the SRP was bonded with the cementitious mortar (Beams B-2, B-3, and B-4), those values were on the order of 0.006 and 0.005 for one and two plies, respectively. These trends confirm that when the cementitious mortar was used, the debonding occurred earlier compared with the epoxy resin, as it was highlighted by the different engagement on the concrete substrate after failure. These data will provide important background for the extension of design criteria developed for FRP laminates to the case of SRP tapes bonded with either epoxy resin or cementitious mortar.

Laboratory outcomes confirmed the effectiveness of SRP for the flexural strengthening of RC members. Even though smaller than CFRP, strength increases provided by SRP were significant if compared with upper limits that the strengthening design needs to respect in compliance with ACI 440.2R-02 (ACI Committee 440 2002) guidelines. Epoxy-bonded SRP performed better than FRP in terms of ultimate deflection—this could be very important, especially for structures that require a high displacement capacity. Overall, SRP strengthening systems appeared to be a promising technique that could be alternative to FRP when durability is not a critical requirement, even though more research is needed on this aspect. The system could be further optimized by improving the bond of the cementitious grout and by developing effective mechanical anchorages able to prevent or delay delamination. The experimental results presented in the paper could represent a first step for the development of code recommendations for the design of flexural strengthening of RC structures using SRP.

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