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EXPERIMENTAL INVESTIGATION ON R.C. BEAMS RETROFITTED IN FLEXURE AND SHEAR BY PRETENSIONED STEEL RIBBONS

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ABSTRACT

The experimental results of three and four point loading tests on reinforced concrete beams strengthened by angles and/or pre-stressed stainless steel ribbons (CAM system) are presented. The stainless steel prestressed ribbons play the role of adjunctive transversal reinforcement as well as confine the structural element. Six flexural critical beams and nine shear critical beams were tested. In the first group four beams were retrofitted with bottom stainless steel angles and transversal ribbons, with two different spacing. In the second group three specimens were retrofitted by wrapping the beam with the ribbons, while three specimens were strengthned by perforation of the beam beneath the slab height, and by partially wrapping the beam by inserting the ribbons through the hole, in order to simulate a strengthening performed without drilling of the slab. The test results prove the effectiveness of the retrofitting system for both the flexural and shear critical beams.

INTRODUCTION

Upgrading of reinforced concrete structures in seismic areas is often required due to material deterioration or the need for higher performance and safety level. There is a need for techniques that can provide cost effective solutions to both the design and implementation of strengthening measures.
The choice of upgrading strategies is usually conditioned by the level of safety that the existing structure is able to ensure. In upgrading structure with poor capacity to withstand seismic action, the use of dissipative devices or base isolation is gaining popularity since they are able to limit the intervention on existing structure, reducing the time during which the structure is not usable. It is noteworthy that in both cases, if strength of structure brittle mechanisms, such as beam to column joints or beam and column shear strength, is really poor a light retrofitting of the structure is still mandatory.
Another retrofitting approach is based on the increment of structure ductility, pursued by ensuring that a highly dissipative collapse mechanism is activated under strong intensity seismic action. To this aim column strength must often be increased to avoid soft story mechanisms, and the plastic hinge region of beams and columns have to be provided of high flexural ductility (Campione *et al.* 2007, Colajanni,

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Papia, Spinella (2013), Colajanni *et al.*, 2013, 2014); at the same time again element brittle mechanism need to be prevented .

In the last decades, many techniques have been developed to increase the flexural and shear strength of reinforced concrete beams designed with poor longitudinal and transversal reinforcements. Beam strengthening by reinforcement plates externally bonded onto the surface of concrete members is utilized as a cheap, structurally capable tool for rehabilitating and/or retrofitting the structurally inadequate load capacity of concrete members. The bonding technology of plates has been deeply l. investigated (Swamy *et al.* (1996), Subedi and Baglin (1998)) and codified (Eurocode 8, 1998). In order to obtain a more durable and effective techniques, recently, fibre reinforced polymer (FRP) material has been used as an attractive alternative to steel plate in reinforcing concrete structures (Trantafillou (1998), Pellegrino and Modena (2002), Yeh and Mo (2005)). The use of synthetic materials is a very appealing option for reinforcing because of their light weight, high durability and high tensile strength. However, their application requires specialized artworks and their effectiveness in presence of heat and fire is still under investigation.

Recently, a new technology, namely the Active Confinement of Manufacts (or Masonry) CAM originally used for seismic retrofitting of masonry structures (Dolce et al. 2002) has been applied to the reinforcing and confinement of reinforced concrete elements (Dolce et al. 2003). The technology is based on the use of pre-stressed stainless steel ribbons that wrap the element constituting adjunctive transversal reinforcement, ensuring confinement to the concrete and, when steel angles are used as longitudinal reinforcements, help the bond to the concrete both by friction and inducing compressive stress in the concrete.

Recently, a large research project focused on the assessment of increasing of both strength and ductility of reinforced concrete beams and poor masonry panels and arch by the CAM system was performed. Here, the results of experimental tests on fifteen reinforced concrete beams critical in flexure or in shear are reported. Four beams critical in flexure were strengthened by bottom angles and wrapped with CAM, six critical in shear where strengthening with high strength stainless prestressed ribbons acting as transversal reinforcement; three of them were retrofitted by perforation of the beam beneath the slab height, and by placing the ribbons around the beam through the hole, in order to simulate a retrofitting performed without drilling of slab. The test results prove the effectiveness of the retrofitting system for both the flexural and shear critical beams.

ACTIVE CONFINEMENT OF REINFORCED CONCRETE BEAM

The application of CAM system for retrofitting concrete elements is based on the use of stainless steel angles and ribbons to wrap reinforced beams and columns (Figure 1). The loops are closed with a special tool, which is able to apply a calibrated prestress to the ribbon. In current applications, the ribbon is 0.9-1 mm thick and 19 mm wide, with yielding and failure strengths in the range 750-850 MPa and 850-1000 MPa respectively, and elongation at failure than often is greater than 10% to 25%. Different connection devices are used, with failure strength higher than the 70% of the strength of the corresponding ribbon.

The system includes drawpieces also, which play the role of connection and force transmission elements between adjacent ribbon loops as well as stress distribution elements on concrete. Stainless steel angles with smoothed edges have common sizes are 40x40x4 mm and steel S235 in usually used. The drawpieces have usually dimensions of 125×125 mm, 4 mm thick [Figure 2a]. Special drawpieces are also utilized when the ribbons do not wrap entirely the beam that have to be reinforced, but the beam is perforated beneath the slab height, and the ribbon is placed through the hole, in order to have a strengthening performed without drilling of the slab. The steel angles placed longitudinally at the bottom of the beam and bonded by mortar act as longitudinal reinforcements. The pre-stressed steel ribbons constituting adjunctive transversal reinforcement, also ensuring confinement to the concrete, and helping the steel angle bond to the concrete both by friction and by generate compressive stress in the concrete. Lengths of angles are also placed under the ribbons at the corner of the face of the beam in order to spread out the confining action and to reduce loss of prestressing due to friction. The small thickness and flexural stiffness of the ribbons facilitate their positioning in narrow space, minimizing the demolition of non structural elements as infill panels and partitions

Figure 1. CAM system applied to reinforced concrete beams and columns

Figure 2. Stainless steel elements of CAM system: a) ribbons; b) angles; and connector

EXPERIMENTAL PROGRAM

The experimental program aims at evaluating the increases in strength and ductility of flexural and shear critical beams strengthened by the CAM system,. Fifteen tests were performed, analysing the behaviour of two types of large-sized RC beams under static loading up to failure. To this purpose, four point bend test for bending strength and three point bend test for shear strength evaluation were performed.

In Figure 3 the geometrical configurations of the tested beams and the load scheme are reported, while in Table 1 the identifying labels and the reinforcing system characteristics are reported. Six Flexural Critical (FC) and nine Shear Critical (SC) beams were casted from the same batch with a low strength concrete and steel rebar grade B450C. The SC beams (Figure 3a, 3c, and 3e) have cross section dimensions of 150x250 mm and span length of 3250 mm, with 2+2 ϕ12 longitudinal rebar and stirrups ϕ6 with 100m spacing, while SC beams (Figure 3b, 3d, and 3f) have cross section dimensions of 150x350 mm and span length of 1450 mm, with 3+3 ϕ18 longitudinal rebar and stirrups ϕ6 with 200 mm spacing. The shear span was set equal to 1000 mm for FC beams, and 650 mm for SC beams, obtaining a corresponding shear span-depth ratio of 4 and 1.86 respectively.

The flexural critical beams were reinforced with two bottom angles of steel grade S235 and dimensions 40x40x4 mm, bonded by mortar EMACO R955M, and 3overlapped ribbons 9 mm thick and 19 mm wide of stainless steel having nominal yielding and ultimate strength of 850 MPa and 1000 MPa respectively. The CAM system ribbon details are summarized in Table1. Two specimens had ribbon spacing of 200 mm (FC-3R-S20), and two ribbon spacing of 100 mm (FC-3R-S10). Two flexural critical beams were tested unreinforced (FC-UR).

The shear critical beams were reinforced with the same ribbons with spacing of 200 mm. Three of them were reinforced by external wrapping of the entire section (SC-3R-S20), while three specimens were retrofitted by perforation of the beam beneath the slab height, and by placing the

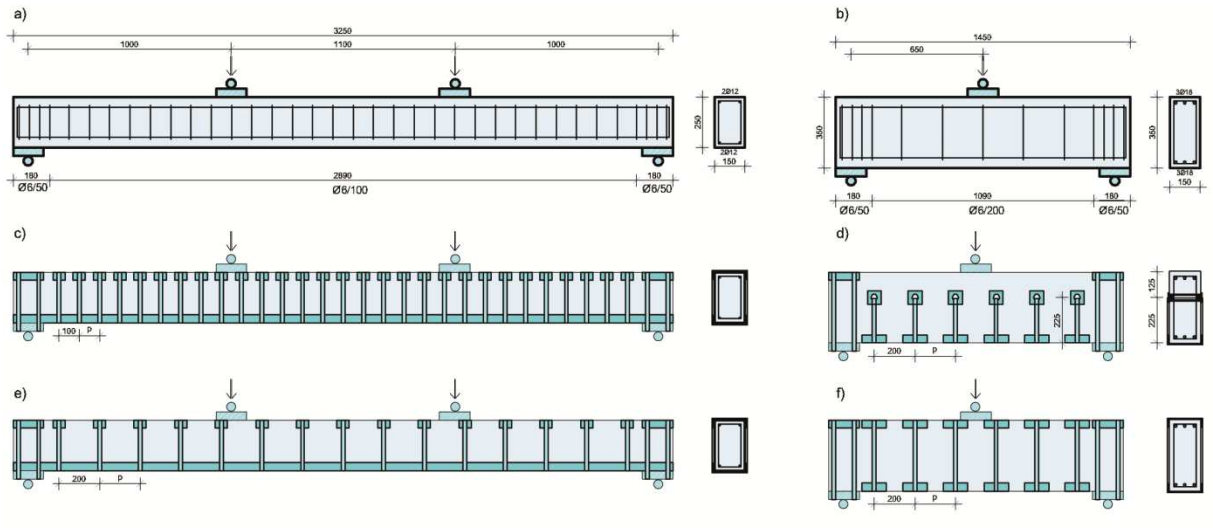

Figure 3. Geometrical scheme and load configuration of tested beam critical in: a) flexure; b) shear

Table 1. Experimental program

Label	#°specimens	#° ribon	ribbon spacing (mm)
FC-UR	2	0	0
FC-3R-S20	2	3	200
FC-3R-S10	2	3	100
SC-UR	3	0	0
SC-US-3R-S20	3	3	200
SC-3R-S20	3	3	200

ribbons around the beam through the hole, in order to simulate a retrofitting performed without drilling of the slab (see Figure 1) (SC-US-3R-S20). Three shear critical beams were tested unreinforced (SC-UR).

BEAM STRENGTH ASSESMENT

In order to elucidate the expected effect of the different configuration of the retrofitting system, simple models available in literature are used to compare the anticipated performance of the different configurations. The response are obtained by modelling the constitutive behaviour of the concrete confined with the B450C steel stirrup by the Mander at al. (1988) model, and evaluating the ultimate strain of the concrete according to Scott et al (1982). Steel behaviour is modelled by a simple bilinear curve with strain hardening. Nominal material strengths were used, and perfect bond is assumed.

For the high strength stainless steel the behaviour of the ribbons is assumed to be elastic up to the collapse of the junction, that is cautionary assumed corresponding to the attainment of 70% of the ribbon steel strength (according to the limit ensured for the junction by the producers). Thus, the confinement model for the concrete is derived as done in the Spoelstra and Monti (1999) model for FRP confined concrete, where the variable confining lateral pressure is taken into account. Thus the lateral confining pressure is obtained by superposition of those ensured by the stirrup and the ribbons.

Beam shear strength is predicted by the simple truss analogy with variable inclination of the compressed concrete strut, applied according the Italian code (Italian Ministry of Infrastructure, 2008). To this aim, in order to take into account the presence of high strength steel ribbons, the inclination of the concrete compressed strut θ and the shear force V_{Rs} that the beam can withstand at the yielding of shear reinforcement are evaluated as follows:

$$1 \le ctg\,\theta = \sqrt{\frac{b_w\, f_c'}{\frac{A_{sw} f_y}{s} + \frac{d_r t_r b_r f_{y,r}}{d\, s_r}} - 1} \le 2.5 \quad : \; V_{Rs} = 0.9d\left(\frac{A_{sw} f_y}{s} + \frac{d_r t_r b_r f_{y,r}}{d\, s_r}\right) ctg\,\theta \qquad (1,2)$$

where A_{sw} is the cross-sectional area of the shear reinforcement, d and b_w the height and the smallest width in the tensile area of the cross-section, $d_r \le d$ the height of the section wrapped with the ribbons, b_r, t_r and s_r width, thickness and spacing of the ribbons respectively, s the spacing of stirrup, f_y, $f_{y,r}$ and f'_c are the yielding strength of stirrup steel, the conventional limit value of the high strength stainless steel (70% of the ultimate strength) and the concrete compressive strength reduced for multi-axial stress state. In Table 2 the ultimate loads that produce the flexural and shear collapse for the six different configurations of the beam are shown together with the maximum strain of the concrete ε_c (c) and of the tensile rebars ε_s (c) and the curvature ϕ at the collapse.

Table 2. Assessment of beam load capacity (kN), strain and curvature at the collapse.

Configuration	FC-UR	FC-3R-S20	FC-3R-S10	SC-UR	SC--US-3R-S20	SC3R-S20
Flexure (y)	44.2	111.4	111.4	328.6	328.6	329.2
Flexure (c)	59.4	124.5	136.3	387.1	387.1	415.3
Shear	167.9	188.9	189.1	184.3	241.9	256
ε_c (c) %	7.24	3.42	6.01	0.70	0.70	1.08
ε_s (c) %	1.10	1.77	3.68	3.94	3.94	5.56
ϕ_c d	8.34	5.19	9.69	4.64	4.64	6.64

The results prove that the beams of the group FC are critical in flexure, while those of the group SC are critical in shear. The effectiveness of the angles in increasing load capacity assessed of the model is obviously conditioned by the assumption of perfect bond between angles and concrete. It is noteworthy the role played of the confining action exerted by the ribbons on the compressed concrete that, for flexural critical beam, allows the ultimate concrete strain to be increased from the value 1.1% for the un-strengthened beam to 1.77 % for ribbons spacing of 220 mm, and to 3.68% for ribbon spacing of 100 mm; this circumstance enable to moderate the reduction of curvature ductility caused by the introduction of the angles. If the angles are introduced without the ribbons, the steel rebar strain at the collapse would drop to ε_s (c)=2.04 %, with a reduction of the collapse curvature of 62% with respect to the un-strengthened beam. When ribbons with 200 mm spacing are introduced the ultimate curvature drop is limited to 37.7%, while when the spacing is reduced to 100 mm, the collapse curvature is increased of 16%. It has to be emphasized that the ultimate concrete strain predicted by the Scott *et al.*(1982) model when applied to concrete confined by the ribbons is likely to be over-estimated.

TESTS ON MATERIALS

The materials used for preparation of the specimens were subjected to test. Two concrete cylinders of dimensions 150x300 mm and three cubes of side 150 mm were subjected to uniaxial compression; the stress strain curve for the cylinders are shown in Figure 4a; an average strength f_{co}=12.45 MPa was detected. In figure 5b the curves of the three direct tensile tests on the angles are reported, that provide an average ultimate stress f_t of about 433 MPa. Finally, direct tensile test on the stainless steel ribbons, with and without connection devices were performed, since the mechanical property of the junction strongly affect the efficiency of the CAM system. In details, six specimens were tested: three with and three without junctions. A servo-hydraulic machine, with a 4000 kN load-carrying capacity, was used for the tensile tests of stainless ribbons, adopting a gauge length of 80 mm. In Figure 5c the six stress strain curves are depicted. All the specimens with and without junction exhibited ultimate strength larger that the nominal one (1000 MPa), proving that a full strength junction is obtained. However, two of the three ribbons without junction exhibited an ultimate strain smaller than 0.1, while the third one ultimate strain was 0.1075. The curves show that the presence of the junction produces a stiffness loss for the ribbon, requiring the activation of larger deformation to turn on high stress level. By.

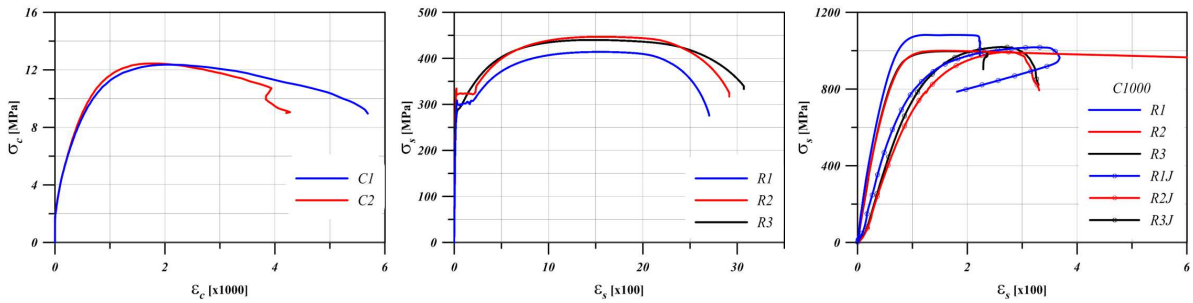

Figure 4. Stress-strain curve for: a) concrete; b) angles; c) steel ribbons without (R#) and with junction (RJ#)

contrast, the junction enlarges the ultimate deformation of the ribbon, ensuring ultimate deformation greater than 3% in all the examined cases

TEST SETUP

The contrast steel frame for load application and the test setup for FC beams are shown in Figure 5. A rigid steel HEA 240 beam was collocated under the plate of the hydraulic jack to obtain a four point load application. Steel cylinders with 40 mm of diameter and plates of 150x150x40 mm were placed between concrete beam and rigid steel beam and the support in order to avoid premature collapse due to stress concentration. Load was applied by hydraulic jacks, having load and displacement capacity of 1000 kN and 200 mm respectively. A load cell was placed between the hydraulic jack and the steel beam, to measure the intensity of the applied force. Two LVDTs were placed on both side of the midspan section of the beam, and at the loading sections, and two at the sections where the support are placed. For the beams strengthened in flexure, eight strain gauges where placed on the angles at the half and at the end of the shear span length in order to check the efficiency of the bond between angles and concrete. Load, displacements and strain values were recorded by a computer data acquisition system, consisting into an HBM MGCPlus unit, able to recording 64 strain gauges, 8 volt signals (for supplementary load cells) and 8 full bridge signals (for supplementary inductive transducers) . The unit control was interfaced by the CATMAN software provided by HBM. The test was performed with load control until the maximum strength of the specimen was reached, then the specimen was loaded depending on the value of the recorded displacement.

In SC beams the distance between the supports was changed according the design specifications for beam and shear span lengths. Load was applied through the steel plate cylinder directly to the midspan of the beam, and the displacements at the midspan only, together with those of the beam at the support sections were measured. Eight strain gauges were placed on different ribbons in order to check their stress at the maximum load and at the collapse.

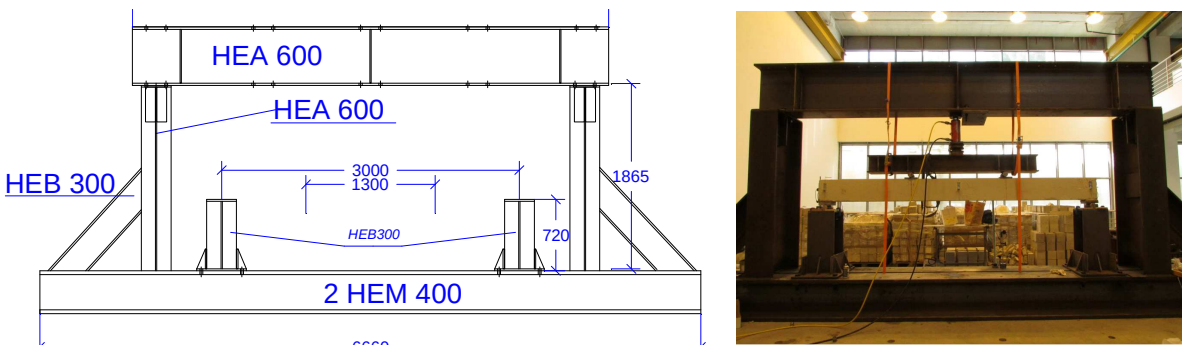

Figure 5. Contrast steel frame and test setup for FC beams.

TEST RESULTS

In Table 3 for each specimen are reported the maximum value of the load and the maximum value of the vertical displacement at the midspan imposed during the test. The latter is obtained by averaging the results of the two LVDTs placed on the two sides of the beam. It has to be emphasized that for the flexural critical beams, due to the extremely large curvature ductility ensured by the beam sections, the testing setup was not able to produce the collapse of the beams; therefore, in these cases, the maximum displacements have to be interpreted as a displacements that the beams are able to withstand without any strength degradation. In Figure 7 the load-displacement curves for the flexural critical beam specimens are reported, showing the results obtained for each of the two LVDTs (s1 and s2) placed at the two sides of the beam midspan section.

The two specimens of the un-strengthened flexural critical beams withstand a maximum load of 45.7 kN and 44.4 kN, very close to the yielding load assessed by the model (44.2 kN), while the assessed ultimate load was not reached since the small plastic hinge rotation attained was not able to activate the hardening of the rebars.

In Figures 8a, the cracking pattern at the end of the test is shown for the specimen FC-UR-b, characterized by vertical cracks starting at the bottom of the beam in the middle region with constant bending moment.

The specimens that were retrofitted with two angles at the bottom tensile chord and 3 superimposed ribbons with 200 mm spacing showed a larger load capacity, increasing the average ultimate load from 45.0 kN to 118 kN. The yielding load assessed by the model was overcome, and about half of the load increment ensured by the hardening was activated, due to the larger values of the attained displacements, that reached the values of 85.4 mm and 92.6 mm for the two specimens.

It has to be emphasized that the increment of the tensile reinforcement by the angles alone would have generated a collapse due to a premature attainment of the ultimate strain in the concrete; the concrete confinement effect ensured by the ribbons allowed a larger deformation of the concrete. Moreover, the measure of the strain gauges placed on the angles showed that they were yielded, proving the effectiveness of the bond with the concrete. The increment of flexural capacity moved the collapse mechanism from pure flexure to a shear-flexure interaction, with the contemporary presence of vertical cracking in the central zone of the beam with constant bending moment, and diagonal cracking along the shear span (Figure 8b).

Thus, the reduction of the ribbon spacing in the FC-3R-S10 specimens does not significantly increased the beam load capacity, but it avoided the appearing of diagonal cracking and increased the rotation capacity of the plastic hinge at midspan (Figures 7c, 7f and 8c), as it is proved by the maximum displacement reached by the specimens FC-3R-S10-b that was 148.9 mm.

The load-displacement curves of three point bend tests on shear critical beams are reported in Figure 9. As expected, all the specimens collapsed in shear. In the unreinforced beams, all the three specimens exhibit a very similar behaviour and load capacity; an average value of the collapse load 188.2 kN was obtained, very close to the value of 184.3 kN predicted by the model. The specimens showed a “ductile” behaviour, whit collapse for diagonal tension due to the small amount of transversal reinforcement. In specimen b and c, where midspan displacements of about 18.4 mm were imposed, the reduction of the beam load capacity was smaller than the 7%. The cracking pattern at the collapse condition is characterized by a principal diagonal cracks that extends from the loading point to the support (Figure 8d).

Table 3. Experimental values of maximum load (P_{max}) and maximum displacement(δ_{max})

Specimen	a		b		c		Media	
	P_{max}	δ_{max}	P_{max}	δ_{max}	P_{max}	δ_{max}	P_{max}	δ_{max}
	kN	*mm*	*kN*	*mm*	*kN*	*mm*	*kN*	*mm*
FC-UR	45.7	44.7	44.4	75.0			45.0	59.9
FC-3R-S20	117.6	85.4	118.5	92.6			118.1	89.0
FC-3R-S10	117.2	110.6	123.2	148.9			120.2	129.7
SC-UR	188.7	21.9	187.6	18.4	188.5	18.3	188.2	19.5
SC-US-3R-S20	230.2	20.0	228.7	20.0	226.7	31.1	228.5	23.7
SC-3R-S20	271.7	14.6	290.8	22.2	268.3	19.2	276.9	18.7

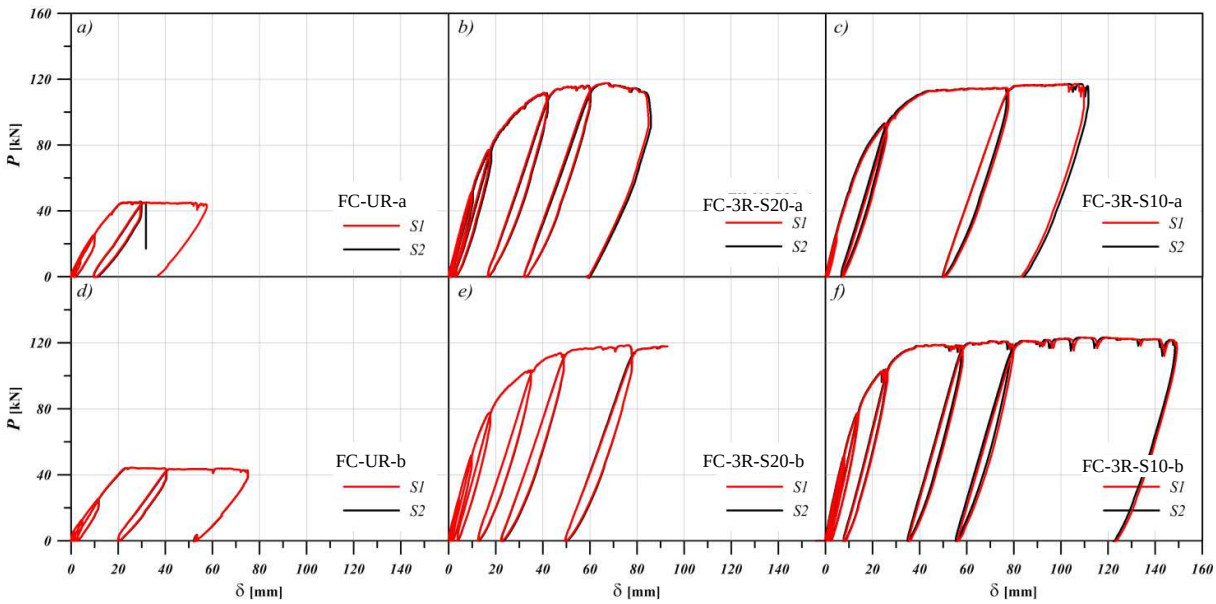

Figure 7. Load displacement curves for flexure critical beams.

The beams strengthened with ribbons that wrap the entire height of the beam, namely series SC-3R-S20, reached the collapse for load values ranging from 270 kN to 291 kN, with an average load capacity of 276.9 kN, that is 47 % larger than that of the un-strengthened specimens and larger of the strength predicted by the model also, proving the efficiency of the system in increase the shear strength of the beam. The average value of the imposed maximum displacement was similar to that of the unreinforced specimens (average value of δ_{max}=18.7 mm) but in these cases the behaviour were more brittle (Fig 9c, 9f, 9i) since the increment of transversal reinforcement turned the failure mode from diagonal tension to shear compression. Only few ribbons reached the yielding stress and no one of the ribbon junctions exhibit any damage. A more spreaded cracking patterns were detected, with many diagonal cracks that starting from the loading point reached the bottom chord and the support (Figure 8f), where large cracking proved that the compression strength of the concrete was reached.

The SC beams designed with ribbon strengthening that wrapped only partially the beam (by perforation of the beam beneath the slab height, and by placing the ribbon around the beam through the hole, in order to simulate a retrofitting performed without drilling of the slab (Figure 8c)) reached the collapse for a mean load values for the three specimens of 228.5 kN, with a very small difference

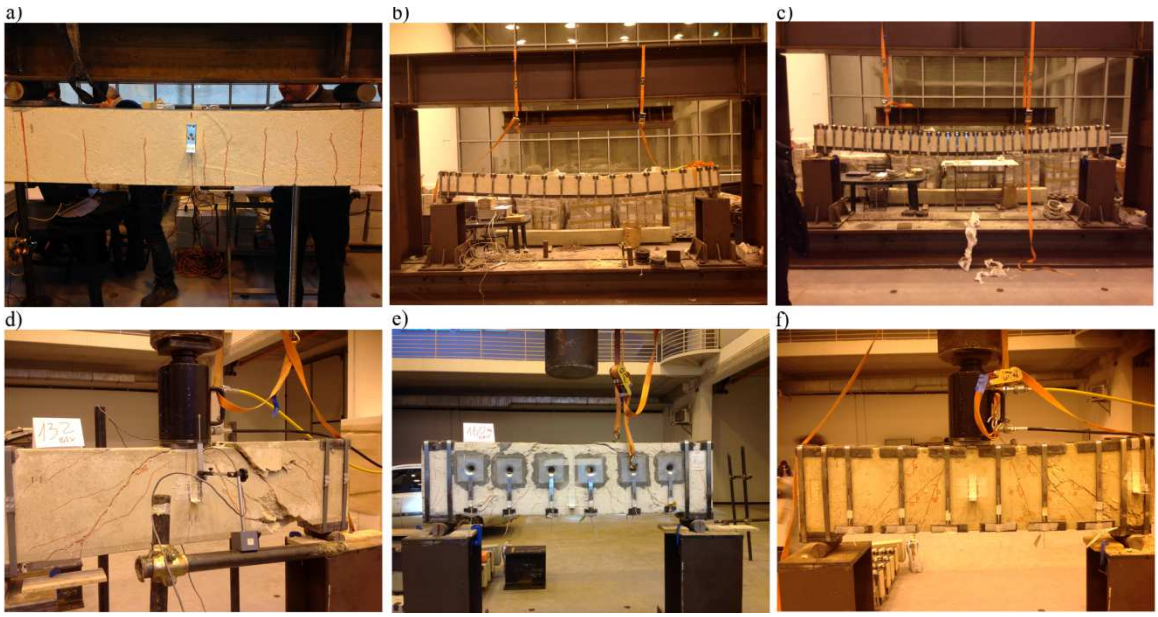

Figure 8. Beam configurations at the end of the tests: a) FC-UR-b, b) FC-3R-S20-b; c) FC-3R-S10-4; d) SC-UR-a; e) SC-US-R3-S20a; f) SC-R3-S10b

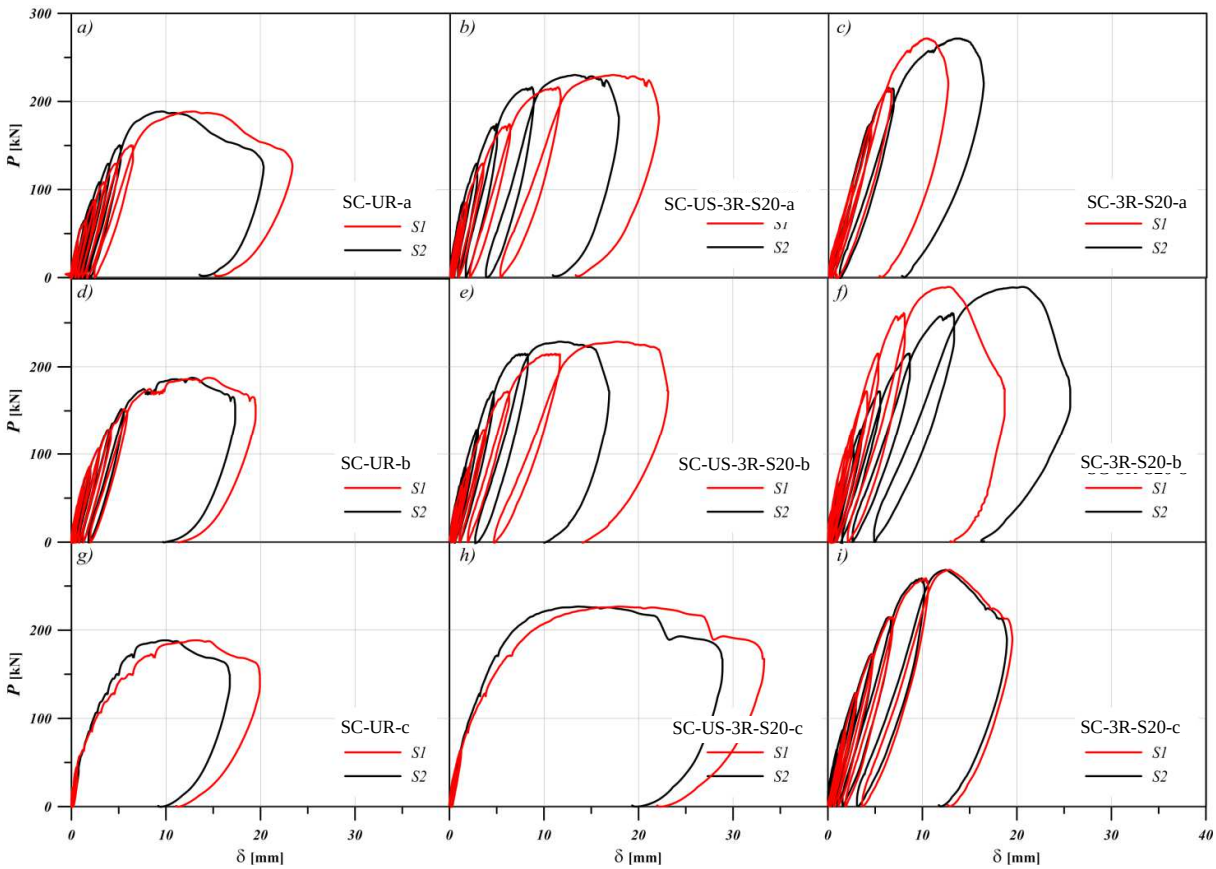

Figure 9. Load displacement curves for shear critical beams.

between the different specimens; the value is 21.4% greater than the mean load capacity of the un-strengthened specimens, proving the effectiveness of the ribbons; however the experimental value is smaller than that predicted by the model, stressing that the reduction of the capacity due to the partial wrapping is not adequately modelled by Equations 1 and 2. The small strength decay showed in the curves in Figure 9b, 9e, and 9h and the large maximum displacement reached by the specimens SC-US-3r-S20-c, equal to 31.1 mm, prove that the behaviour was "ductile", similar to that of the un-strengthened specimens. The crack pattern at the collapse for specimen SC-US-R3-S20a is shown in Figure 10, and is characterized by a widespread cracking, whit larger diagonal cracks that starting from the holed plates reached the bottom tensile chord in the vicinity of the support. The steel ribbons, delaying the enlargement of the principal cracking, have induced the formation of new cracks parallel to the previous one. Let us stress that, due to the ribbon spacing dimensions, many cracks goes from the plate to the bottom chord without intersect any ribbon. Therefore, a more reduced spacing of the ribbons, that actually was larger than $0.8\ d_s$=180 mm, would have improved the performance of strengthening system. Noteworthy, from Figure 10 it can be recognized that the compressed concrete

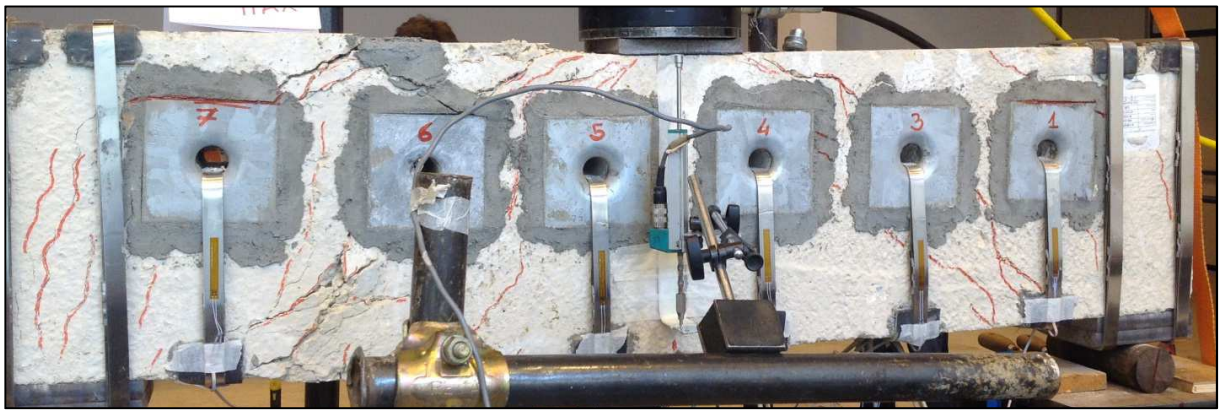

Figure 10. Crack pattern at the collapse for specimen SC-US-R3-S20a

of the top chord in the vicinity of the load point is crushed due to the absence of the confining action exerted by the ribbons. It has to be emphasized that, in an actual beam, the compressed cord would have a T shape with an increased capacity of withstand compression actions.

CONCLUSIONS

The main results of a research aiming at emphasized the effectiveness of the use of angles and prestressed stainless high strength steel ribbons for flexural and shear strengthening of poor detailed reinforced concrete beams were presented. The experimental results of three point and four point bend tests carried out at the laboratory of the "DICIEAMA" Department of the Messina University were reported and discussed. Fifteen beams were tested, divided into two groups. To the first one pertain six flexural critical beams, to the second one nine shear critical beams.

Two un-strengthned flexural critical beams were tested as benchmarks; the other four were strengthened by angles and by the prestressed steel ribbons with two different spacing. The experimental results proved the efficiency of the strengthening system and the ability of the ribbons in increasing the bond between the angles and the substrate and confining the compressed concrete, enhancing the element ductility. The values of the load carrying capacity of the unreinforced elements were increased of the 160% , as well as the displacement capacity that was increased by the ribbons.

The results of the test on the shear critical beams were successful too. The shear capacity of the beams was increased by the strengthening ribbons of 47% when the they wrapped the entire beam, while an increment of 21% was found for beams wrapped only partially by making a perforation of the beam beneath the slab height, and by placing the ribbon around the beam through the hole, in order to simulate a strengthening performed without drilling of the slab. The ribbons induced a spreading of the cracks along the shear span, involving a larger zone of the concrete in the energy dissipation process.

The obtained results proves the efficiency of the investigated procedure for strengthening reinforced concrete elements, and claim for a development of research activities on this topic. The natural development of this research will be the derivation of analytical models able to represent with larger efficiency the behaviour of shear critical partially wrapped beams according models based on the plasticity theory recently proposed (Colajanni, Recupero, Spinella (2008), (2010), Spinella, Colajanni, Recupero (2010)). Moreover, the response will be investigated by finite element method based on well consolidated theory (Spinella, Colajanni. La Mendola, 2012)

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