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Shear behaviour of steel fibre reinforced concrete beams

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ABSTRACT

The possibility of substituting traditional transverse reinforcement (stirrups) for steel fibres in precast elements can significantly reduce production costs. In the present paper, the shear behaviour of prestressed elements has been investigated by means of experimental tests on full scale beams. Tests concern beams with conventional as well as steel fibre reinforcement. Experimental results show that the shear behaviour of fibre reinforced concrete beams without conventional reinforcement is similar to, or even better than that of beams with stirrups. When used in beams with stirrups, steel fibres significantly improve their shear strength. A discussion on the contribution of steel fibres on the shear strength is also presented, with reference to the latest RILEM provisions.

RÉSUMÉ

La possibilité de remplacer des armatures transversales traditionnelles par des fibres d'acier dans des éléments pré-tendus peut apporter des améliorations considérables sur les performances structurales. Dans ce rapport, le comportement en cisaillement des éléments précontraints a été étudié à l'aide de tests expérimentaux sur des poutres à échelle grandeur nature. Les essais portent aussi bien sur des poutres avec armatures traditionnelles que sur des poutres renforcées à l'aide de fibres. Les résultats expérimentaux montrent que les performances des poutres en béton de fibres sans armatures traditionnelles sont similaires, sinon meilleures, à celles des poutres avec armature ordinaire de cisaillement. Lorsqu'elles sont utilisées dans des poutres avec armatures traditionnelles, les fibres d'acier améliorent considérablement leur résistance en cisaillement.

1. INTRODUCTION

The interest on fibre-reinforced concrete (FRC) structures is continuously growing [1].

Steel Fibre Reinforced Concrete (SFRC) is already widely used in structures where fibre reinforcement is not essential for integrity and safety (*i.e.* slabs on grade). In the last few years, fibres have been also used as main reinforcement in structures under bending [2]; early examples are the square slabs of the Heathrow Airport car park in London [3] or the foundation slab of Postdamer Platz in Berlin [4].

Moreover, the addition of steel fibres to high strength concrete (HSC) results in an optimal composite material, since their presence reduces the well known brittleness of HSC and increases fatigue strength and resistance to impact loading [5].

In heavy precast industry, where HSC is commonly adopted, diffused fibre reinforcement could be utilized to reduce or substitute conventional transverse reinforcement, with advantages in the production process and reduction of labour costs [6].

Some studies have been already undertaken to better investigate the shear behaviour of structural elements with fibre reinforcement.

Noghabai [7] conducted several experimental tests on beams of different dimensions and shear spans, and composed of different types of fibres. His results show that the presence of fibres is essential, particularly when web traction or tension flange collapse occurs. Moreover, the presence of fibres can increase the load requirement for tension-flange collapse, thus increasing the likelihood that compressive crushing might be the ultimate collapse mechanism. Finally, it was found that the performance of beams with hybrid fibres (with varying aspect ratio and shape) was superior to that of beams with only one type of fibre.

Casanova, Rossi and Schaller [8] showed that the substitution of stirrups in HSC structures with a volume fraction of fibres higher than 1.25% improves the overall ductility, and better control crack growth as compared to conventional shear reinforcement.

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The results obtained by Williamson [9] emphasized that, for specimens with straight fibres ($V_f = 1.5\%$) instead of stirrups, the shear strength increased up to 45% as compared to similar specimens with conventional shear reinforcement. When using hooked fibres the shear strength increased up to 45-70% so that bending collapse occurred prior to shear collapse.

Di Prisco and Ferrara [10] experimentally verified that, in thin-web elements made of fibre reinforced concrete without transverse reinforcement, it is possible to increase the ductility and to limit crack opening.

Recently, an extensive research on fibre reinforced beams was carried out (Brite Euram Program) [11]. The research showed that steel fibres can substitute the minimum shear reinforcement necessary in ordinary r.c. members to ensure a ductile failure.

In this paper, the results of an experimental research program on fibre reinforced prestressed beams are presented. Six full scale beams, two made of plain concrete and four made of fibre reinforced concrete, were tested to evaluate the possibility of substituting the minimum conventional transverse reinforcement required by Eurocode 2 (EC2) [12] with steel fibres. The shear behaviour of the beams is analysed both in the prestressing transfer zone (TZ where, usually, reinforcement is required to provide shear strength) as well as in a zone where the prestressing force is completely diffused (DZ where, usually, the minimum shear reinforcement is sufficient). Particular attention was paid to the shear contribution offered by fibres, either in addition to stirrups or in substitution of conventional shear reinforcement.

Shear behaviour is analyzed both at Service Limit State, with particular attention to fibre effects on crack pattern, and at Ultimate Limit State, where steel fibres may influence the shear failure mechanism.

2. MATERIAL AND SPECIMEN GEOMETRY

In order to investigate the effect of fibres on shear strength, six prestressed concrete beams were cast by adopting a concrete having a compressive strength (target characteristic value from cubes) $f_{ck,cube} = 75$ MPa (C60/75) and the composition reported in Table 1.

Table 2 shows the average compressive strength $f_{c,cube}$ (measured from 100x100x100 mm cubes), the tensile strength f_{ct} (measured from cylinders with diameter equal to 80 mm, and height equal to 240 mm) and the Young's modulus of concrete E_c (measured from cylinders). All these quantities were measured on 3 specimens immediately before testing the beams. It can be noticed that the materials adopted for five out of six beams (Beams 1, 2, 3, 5 and 6) have almost the same mechanical properties, while the concrete of Beam 4 has a compressive strength approximately 10% higher.

Table 3 shows the yielding (f_{sy}) and tensile (f_{st}) strength of the traditional steel reinforcement (cold formed welded mesh fabric and Tempcore rebars). The mechanical properties of the hooked steel fibres adopted in the present research are presented in Table 4.

Table 1 - Concrete composition

CEMENT TYPE	CEM I - 52.5R
CEMENT CONTENT	380 kg/m^3
MAXIMUM AGGREGATE SIZE	15 mm
WATER-CEMENT RATIO	0.31
PLASTICIZER	2.75 l/m^3 in beams without fibre; 3.67 l/m^3 in beams with fibre
FIBRE CONTENT	50 kg/m^3 (V_f=0.64%) only in Beams 3 and 4

Table 2 - Mechanical properties of concrete

SPECIMEN	$f_{c,cube}$ [MPa]	f_{ct} [MPa]	E_c [MPa]
BEAM 1	84.9	4.43	41383
BEAM 2	86.1	3.69	41556
BEAM 3	82.3	4.65	44191
BEAM 4	92.8	5.18	43053
BEAM 5	83.6	4.43	42293
BEAM 6	84.2	4.72	43527

Table 3 - Mechanical properties of transverse reinforcement steel

TRANSVERSE REINFORCEMENT	f_{sy} [MPa]	f_{st} [MPa]
STIRRUPS Φ6	481	614
MESH Φ5	575	658

Table 4 - Characteristics of the two types of fibres adopted

STEEL FIBRES	RC-45/30-CN (Beam 3/5)	RC-80/30-BP (Beam 4/6)
TYPE OF STEEL	Low carbon	High carbon
SHAPE	Hooked	
SECTION	Circular	
LENGTH [mm]	30	
DIAMETER [mm]	0.62	0.38
ASPECT RATIO [l/ϕ]	48	79
MINIMUM TENSILE STRENGTH [MPa]	1250	2300

Two beams (Beams 1 and 2) were cast with plain concrete, while 50 kg/m^3 of steel fibres ($V_f = 0.64$ %) were added to the remaining beams. Two types of fibres were used: a normal strength fibre (45/30) having a length of 30 mm and an aspect ratio (length/diameter) equal to 45 and a high strength fibre (80/30) having a length of 30 mm and an aspect ratio of 80. Fibres 45/30 were used in Beams 3 and 5 while fibres 80/30 were used in Beams 4 and 6.

As mentioned above, the shear behaviour in both transfer and diffused zones was investigated. Firstly, four beams (Beams 1-4) were cast and tested in each zone. Secondly, in order to confirm the results obtained from the first four beams, two other beams (Beams 5 and 6) were cast, but tested only in diffused zones to better verify the possibility of substituting the minimum conventional reinforcement with steel fibres.

All beams were designed according to EC2 [12] provisions in order to have a shear failure in either of the two testing areas. For this reason, the flanges and web out of the experimental zones (the collapse location) were stiffened: the section of the beam along the zones chosen for the shear collapse has therefore the typical geometry of a widespread precast beam with a panel-web thickness equal to 120 mm (see Fig. 2).

The test area was 1.75 m long (5 times the web depth) while the overall beam length was 9 m for Beams 1-4 and 9.9 m for Beams 5-6 (Fig. 2). The beams were simply supported with a span of 5.65 m and a point load was applied 2.2 m away from support "B" (Fig. 1), in order to have a constant shear force along the testing zone (neglecting dead load).

Two shear tests were performed on each beam, adopting the configuration shown in Fig. 1.

For Beams 1-4, the first test concerned zone TZ, while the second was devoted to zone DZ. Concerning Beams 5-6, both tests were carried out in a zone of diffused prestressing (DZ). The tests in zone TZ are representative of the beam behaviour near supports, where transverse reinforcement is necessary to improve both the strand bond behaviour through confinement and the shear strength of the beam, while those in zone DZ represent the behaviour in an internal part of the beam, where usually a minimum reinforcement amount is required and the prestressing action is completely diffused.

Fig. 2 shows the beam geometry and the strand position. Strands having a diameter of 0.6'' were used with an initial tension of 1400 MPa (before prestress losses). The initial bending moment due to prestressing, referred to the centroid of the gross cross-section, was about 670 kNm.

The load set-up allowed the simulation of the actual behaviour of a full scale precast beam where secondary beams usually transfer point loads on the main girder (Fig. 3).

In TZ and DZ zones different types of transverse reinforcement were adopted:

- Beam 1 was made of prestressed concrete without any transverse reinforcement (neither in TZ nor in DZ zones);
- Beam 2 was made of prestressed concrete with transverse reinforcement designed according to EC2 [12]. In TZ, the stirrups (ϕ6@100 mm) were designed on the basis of truss mechanism while in DZ, transverse reinforcement was the minimum required by the code (mesh fabric ϕ5@200x200 mm);
- the TZ zone of Beam 3 and Beam 4 was made with the same transverse reinforcement of Beam 2. In the same beams, the DZ zone was made without traditional rebars, so that steel fibres were the only reinforcement present in the beam web;
- Beams 5 and 6, being copies of the DZ experimental models of Beams 3 and 4, respectively, were cast with only steel fibres as shear reinforcement;

In the remaining parts of the beams, stirrups ϕ10@100 mm were adopted.

Table 5 summarizes the shear reinforcement adopted in both zones of all the beams tested.

3. SET-UP DESCRIPTION

It is well known that shear strength in a beam is initially provided by concrete. After cracking, if transverse reinforcement is present, the shear strength can be ensured by a truss mechanism. Before the truss mechanism

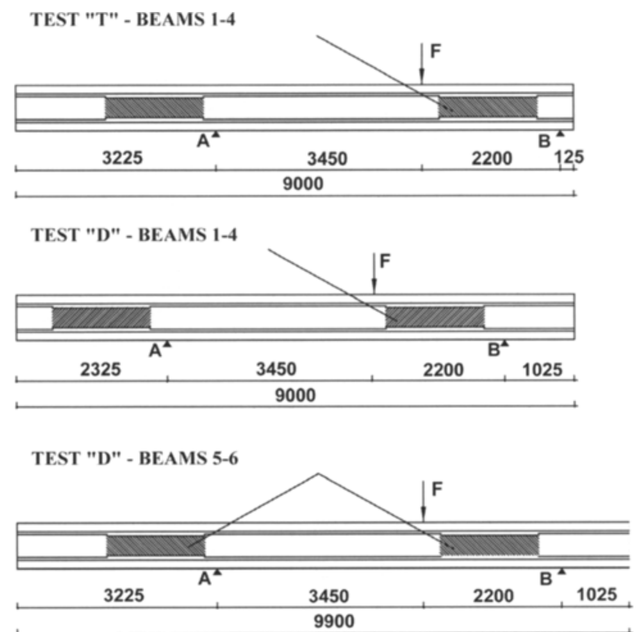

Fig. 1 - Scheme of the loading phases.

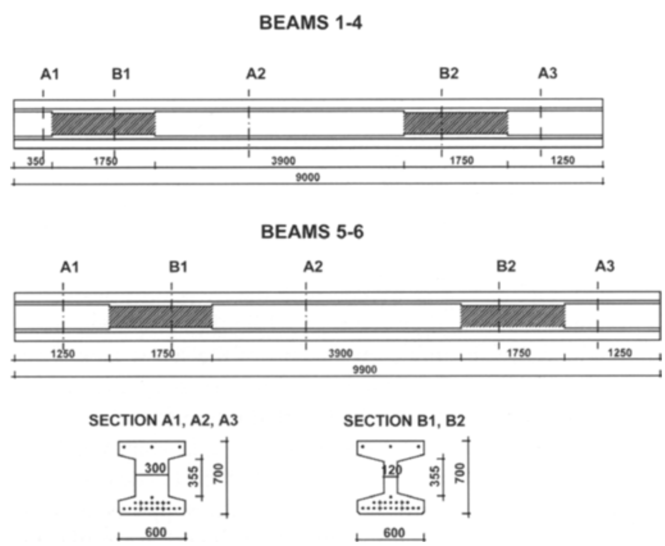

Fig. 2 - Geometry of the full scale beams.

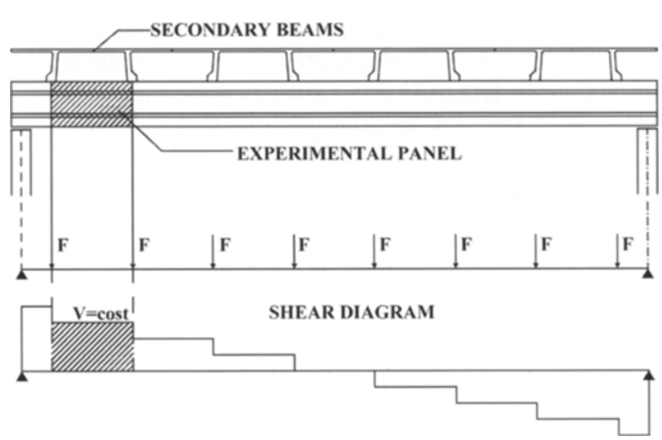

Fig. 3 - Load scheme for a beam subjected to pointed loads from secondary beams.

Table 5 - Shear reinforcement in each experimental test

	TEST	SHEAR REINFORCEMENT (experimental panel)
BEAM 1	TZ	*no shear reinforcement*
	DZ	*no shear reinforcement*
BEAM 2	TZ	*stirrups (design reinf.)*
	DZ	*mesh (minimum reinf.)*
BEAM 3	TZ	*stirrups +45/30 fibres*
	DZ	*45/30 fibres*
BEAM 4	TZ	*stirrups +80/30 fibres*
	DZ	*80/30 fibres*
BEAM 5	DZ I	*45/30 fibres*
	DZ II	*45/30 fibres*
BEAM 6	DZ I	*80/30 fibres*
	DZ II	*80/30 fibres*

activates, an unstable branch may occur in the beam response. In order to capture this unstable branch, a displacement controlled test had to be carried out.

Displacement control was obtained by adopting an electro-mechanical jack having a maximum loading capacity of 1000 kN and a stroke of 350 mm. By imposing the engine speed it was possible to control the displacement rate.

The jack was placed below the beam and the load (F) was applied by means of a steel frame (Fig. 4).

For the heavily reinforced beams, with a strength higher than 1000 kN (maximum load applicable by the screw jack), two 600 kN hydraulic jacks were used in parallel with the screw jack. To ensure a displacement controlled test in the range of interest, an initial force smaller than the first cracking load, was applied with the hydraulic jacks and kept constant throughout the test, while the remaining load up to collapse, was applied by means of the screw jack. The applied loads (both from the screw and the hydraulic jacks) were measured with load cells inserted in the frame.

Fig. 5 shows the measuring devices: Linear Variable Differential Transformers (LVDTs) allowed to measure the vertical displacements of the beam and the supports as well as the strand slips; potentiometric transducers were placed on both sides (back and front) of TZ and DZ zones (Fig. 6) to measure crack openings and strut deformations. The latter transducers were placed with an inclination variable from 20° (for the strut deformation) to 70° (for the crack opening) with respect to the horizontal line (Fig. 6). These inclinations were selected based on the observation that, at collapse, the arch action would have prevailed, with a strut inclination approximately equal to the shear panel diagonal. This assumption was confirmed during the tests.

4. EXPERIMENTAL RESULTS AND DISCUSSION

The tests were performed by imposing a monotonically increasing displacement until the ultimate load was reached. The tests started with two loading and unloading cycles in the elastic branch.

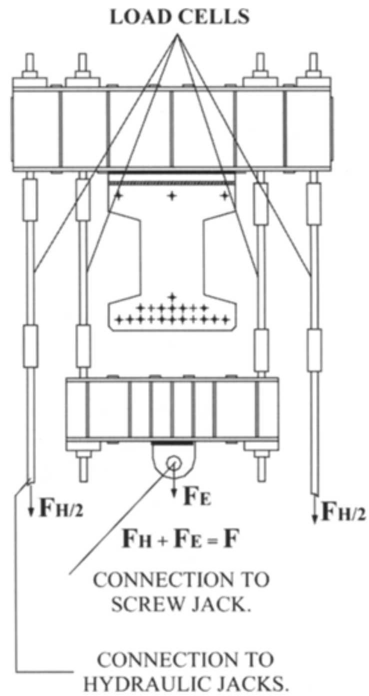

Fig. 4 - Scheme of the steel loading frame adopted.

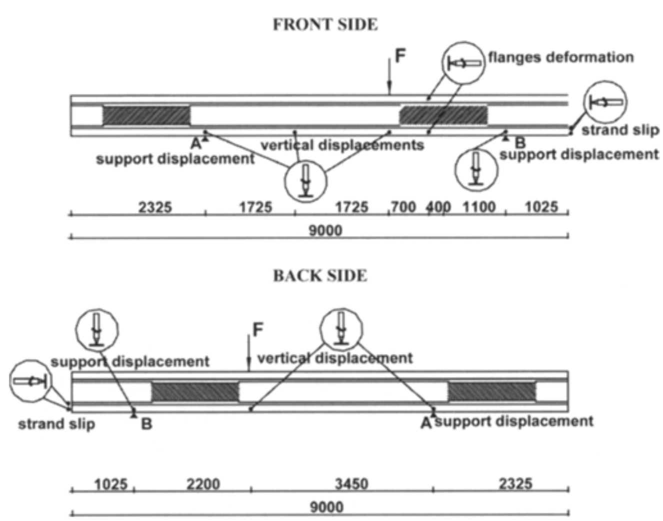

Fig. 5 - Measuring devices on the front and back sides.

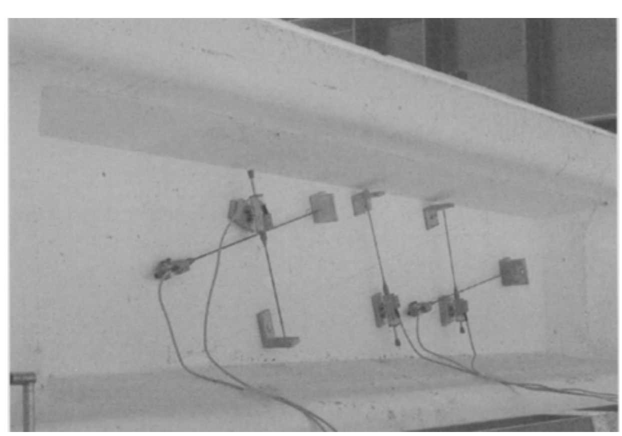

Fig. 6 - Instrumentation in the experimental model.

Fig. 7 shows the shear force versus displacement curve for the tested beams both for TZ (Beams 1, 2, 3 and 4; Fig. 7a) and DZ zones (Beams 1, 2, 3 and 6; Fig. 7b). In all tests, cracking appeared at the end of the elastic stage, with a shear force approximately equal to 450 kN. Only Beam 4 shows a higher elastic limit due to a greater concrete strength (Table 2).

In the post cracking stage, some differences are observed in the response of the six beams. Beam 1, without transverse reinforcement, shows the smallest post cracking strength. In the TZ zone, a brittle behaviour occurred after cracking (Fig. 7a). The post cracking load, related to the strength offered by concrete alone, was not able to increase beyond the value reached at the elastic limit. In the DZ zone, the post cracking load exceeded the first cracking load after large deformations and significant damage (Fig. 7b).

The results of Beam 2, with traditional reinforcement, and of the fibre reinforced Beams 3, 4 and 6, are similar. In the TZ zone, where a constant amount of transverse reinforcement is provided (regardless of the fibre content) fibres allow to increase the ultimate shear force (up to 20% more than the ultimate load of Beam 2). Moreover, the shear-displacement curve shows a more stable, resistant, and ductile post-peak behaviour for fibre reinforced beams (Fig. 7a).

As far as the DZ zone is concerned, the observed behaviour of Beam 2 (with minimum reinforcement) and Beam 3 (with fibre reinforcement only) was similar. Beam 6 showed the best performance: at the crack onset, a small load decrease was observed and the post cracking behaviour was stable until the ultimate load was reached.

Fig. 8 shows a comparison between the shear-displacement curve of the DZ zones with either 45/30 fibres (Fig. 8a) and 80/30 fibres (Fig. 8b). The figures show that the test results present a remarkable consistency. These results confirm that 45/30 fibres ensure at least the same shear resistance offered by the minimum shear reinforcement.

By using 80/30 fibres (Beam 4 and Beam 6), the structural response is more stable and a higher strength is observed. A greater bearing capacity was observed for Beam 4, due to its better concrete characteristics (Table 2).

The use of 80/30 fibres results in a slight improvement of strength and ductility. However, the considerable cost of these fibres (approximately twice as much as 45/30 fibres) and the reduced concrete workability does not seem to justify their use for practical applications.

Figs. 9 and 10 show the width of the main cracks versus the shear force for the TZ and DZ zones, respectively, for Beams 2, 3 and 4. In order to compare the shear contribution of the two fibre types adopted, a reference opening value for the main crack equal to 1.5 mm, (significant for ultimate conditions) is adopted. At this crack opening value in the TZ zone (Fig. 9), Beam 2 (shear reinforcement according to EC2) carries a shear force equal to 560 kN, Beam 3 (same shear reinforcement and 45/30 fibres) carries a shear force of 620 kN, and Beam 4 (same shear reinforcement and 80/30 fibres) a shear load equal to 730 kN. The shear strength increase

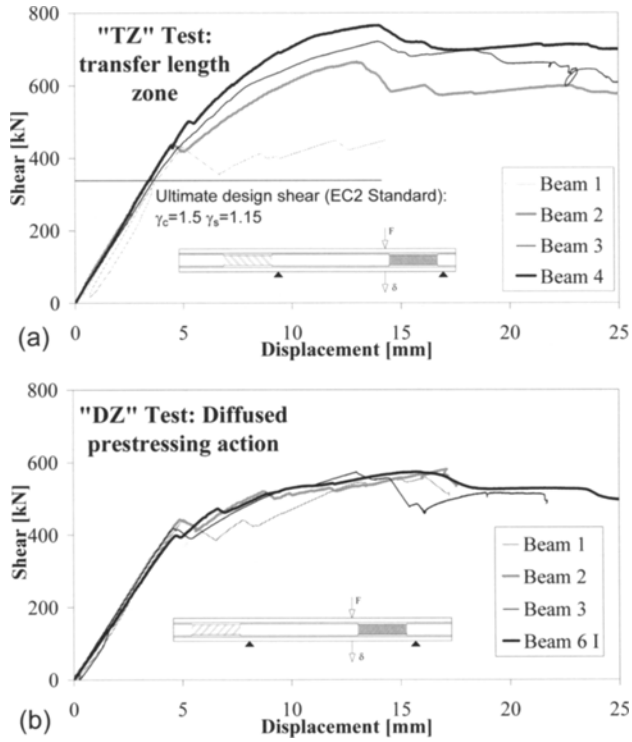

Fig. 7 - Shear-displacement curves both for the TZ Test (a), and the DZ Test (b).

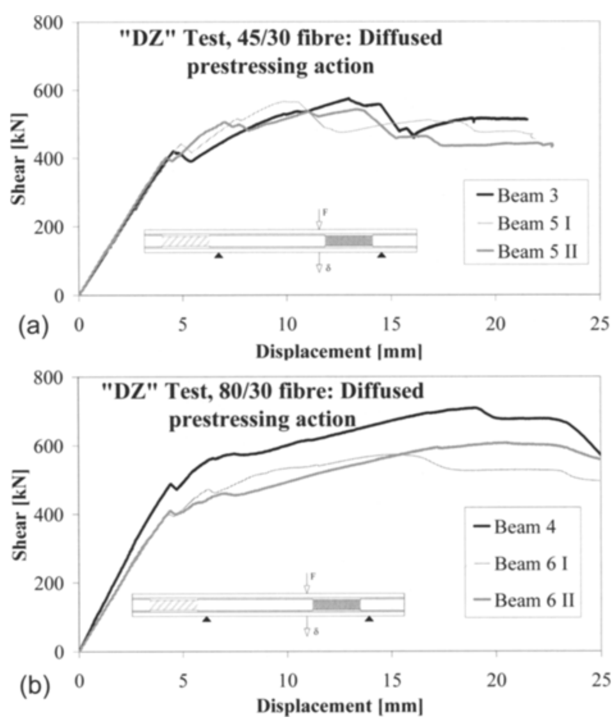

Fig. 8 - Repeatability of results: shear-displacement curves for 45/30 fibres (Beam 3 and Beam 5), DZ test (a), and for 80/30 fibres (beam 4 and Beam 6), DZ test (b).

due to the use of either 45/30 or 80/30 fibres is approximately equal to 12 % and 32 %, respectively. In the latter case, the higher concrete strength of Beam 4 should also be accounted for.

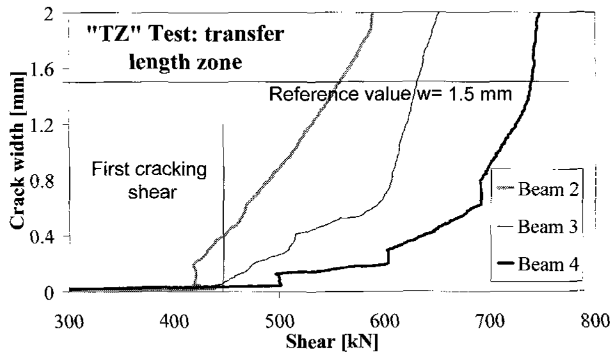

Fig. 9 - Main crack width-shear curves for the TZ Test.

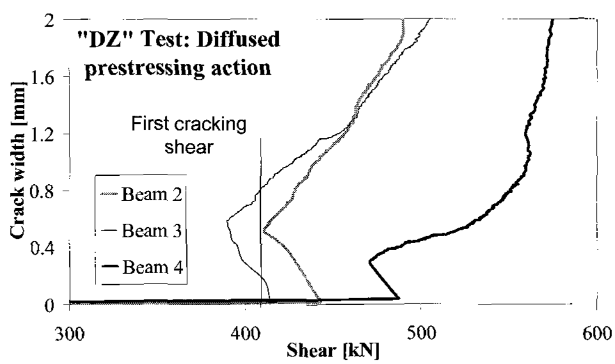

Fig. 10 - Main crack width-shear curves for the DZ Test.

Concerning DZ zones (Fig. 10), the behaviour of Beams 2 and 3 is similar. The good performance of Beam 4 (with high carbon steel fibres) is again confirmed: the crack width increases more slowly, particularly just after first cracking, till a shear force value equal to 480 kN is reached, after which the crack propagation showed a behaviour similar to that of the other beams.

Figs. 11 and 12 show the crack pattern evolution for TZ and DZ zones respectively. The photographs concern three different load levels: the first corresponds to the first crack onset, the second to the maximum load, while the third concerns the final crack pattern. It can be noticed that: in TZ zone of Beam 1 a single wide crack occurs; in Beam 2 cracks do not merge into a single main crack and several cracks are present according to the classical truss mechanism; in Beams 3 and 4 cracking is more diffused and, consequently, the crack width is smaller.

As far as the DZ zones are concerned, Beams 1 and 2 show a similar collapse mechanism, influenced by the panel geometry: the presence of a mesh fabric (Beam 2) induces a widening of the resistant compression strut and no evident truss mechanism is observed. Beams 3 and 4 show more diffused cracks than Beam 2, especially next to the shear failure load: at this point, a wider compression strut appears involving a greater bearing capacity.

It should be noticed that the resisting mechanism for Beams 2, 3, and 4, initially showed a distributed crack pattern in the panel, and eventually developed with a progressive rotation during which the cracks merged, leading to a single macro crack oriented as the diagonal of the panel. This progressive phenomenon is the cause of the several small unloadings observed in the shear-displacement curve (Fig. 7).

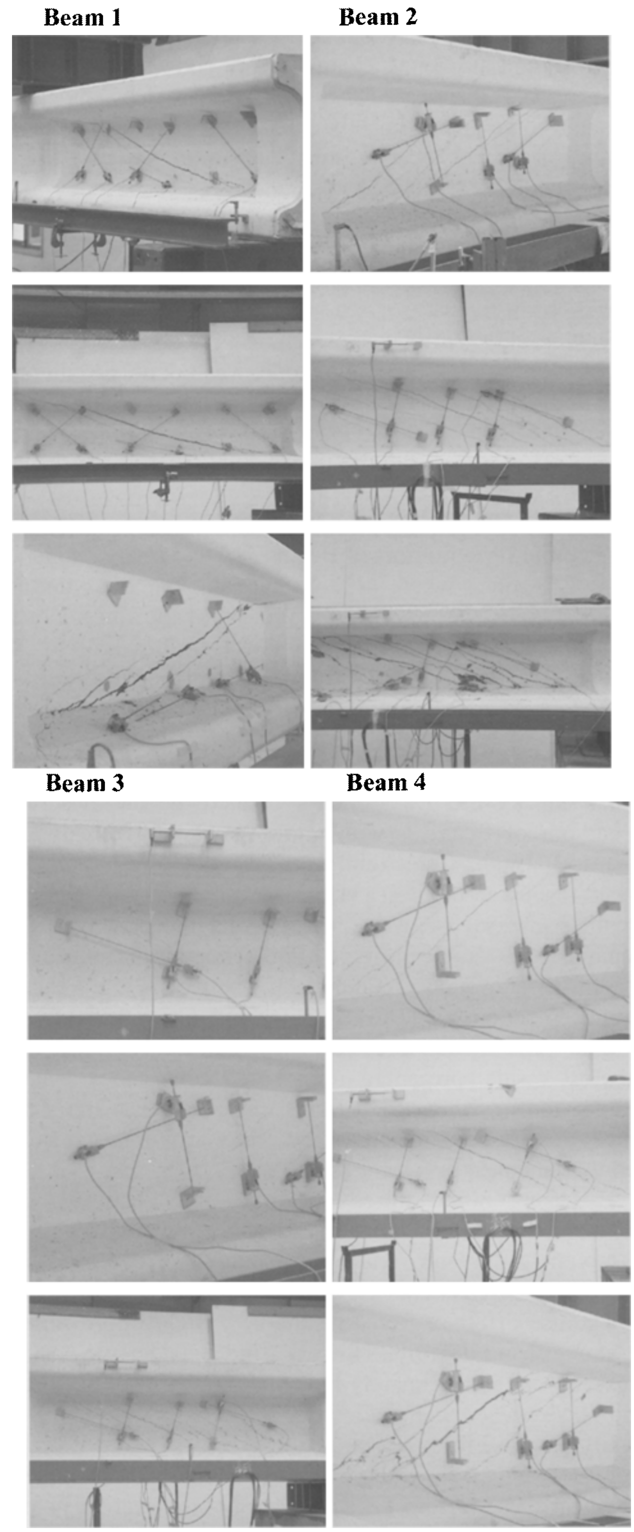

Fig. 11 - Crack pattern evolution in TZ zone of the four beams.

5. STANDARD OVERVIEW

A discussion of the experimental results in light of the provisions given by RILEM Recommendations on fibre

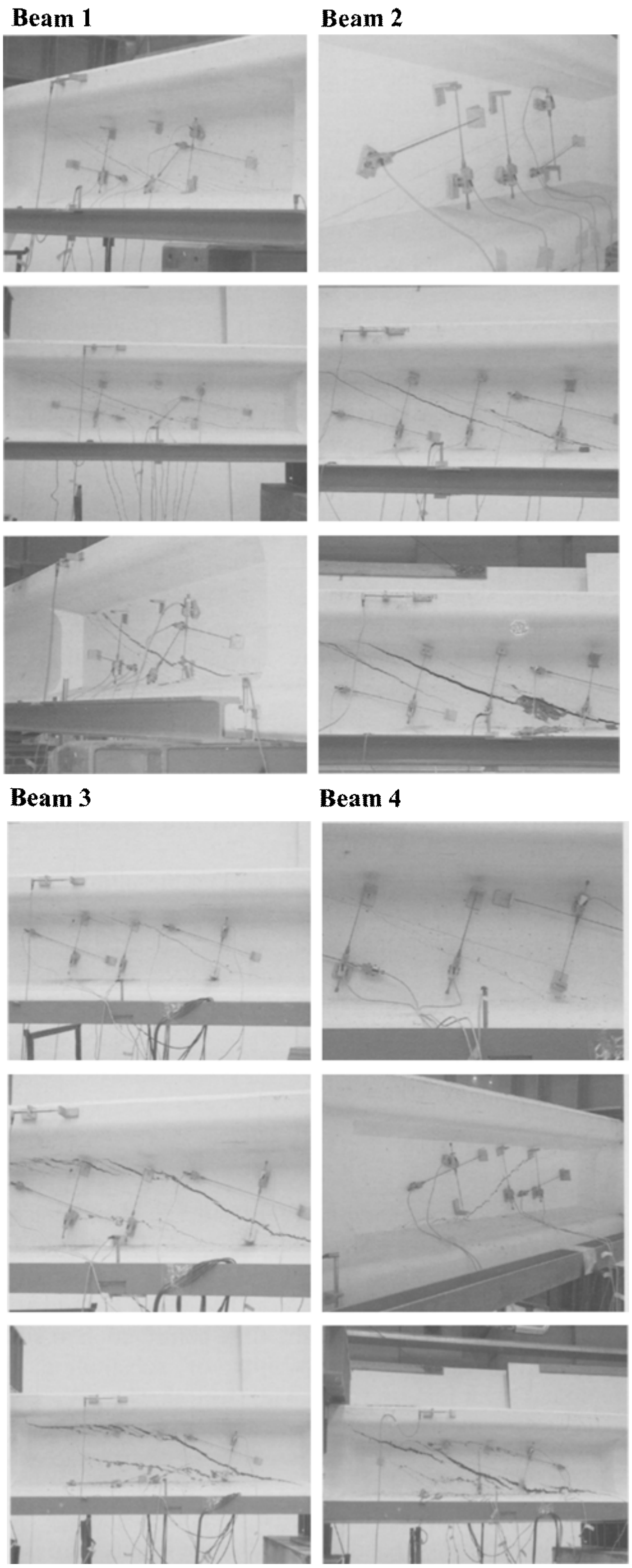

Fig. 12 - Crack pattern evolution in DZ zone of the four beams.

reinforced concrete design [13], and EC2 [12] (which is limited to reinforced concrete structures without fibres) is herein presented.

Table 6 shows the value of each shear strength contribution (concrete, stirrups, mesh and fibres) calculated according to EC2 Standard Method, EC2 Variable-Angle Truss Model and RILEM Provision. The shear strength is also determined according to a Variable-Angle Truss Model assuming $\lambda = \cot\theta = 4.0$ ($\theta = 14°$), where θ is the inclination of the compression strut on the beam axis (which is the maximum possible value of λ for the given geometry of the panel tested).

Table 6 – Resistance shear contributions

Contribution	Experiments	EC2 Standard [14]	EC2 Var. Inc. $\lambda = 2.5$	RILEM [13]	Var. Inc. $\lambda = 4.0$
Transfer Length zone "TZ"					
Shear reinforcement (stirups ϕ6/100)					
Concrete	443	331	0	274	0
Stirrups	219	152	398	152	636
Total	**662**	**483**	**398**	**426**	**636**
Shear reinforcement (stirups ϕ6/100)+ 45/30 fibres					
Concrete	443	331	0	274	0
Stirrups	285	152	398	152	636
Fibre		0	0	35	0
Total	**728**	**483**	**398**	**461**	**636**
Shear reinforcement (stirups ϕ6/100)+ 80/30 fibres					
Concrete	443	331	0	274	0
Stirrups	327	152	398	152	636
Fibre		0	0	163	0
Total	**770**	**483**	**398**	**589**	**636**
Diffused prestressing action "DZ"					
Minimum shear reinforcement (mesh ϕ5/200)					
Concrete	443	331	0	274	0
Mesh	138	64	178	64	286
Total	**581**	**395**	**178**	**338**	**286**
45/30 fibres only					
Concrete	443	331	0	274	0
Fibre	135	0	0	35	0
Total	**578**	**331**	**0**	**309**	**0**
80/30 fibres only					
Concrete	443	331	0	274	0
Fibre	196	0	0	163	0
Total	**639**	**331**	**0**	**437**	**0**

NB. All values are in kN.

The experimental shear strength assumed for the comparison represents the mean value of the measured maximum loads of each beam tested. The concrete strength contribution for the experimental tests was conventionally assumed as equal (for all beams) to the first cracking load of Beam 1 (Fig. 7) without transverse reinforcement. Note that, since no significant differences were observed between TZ and DZ zones of Beam 1 (Fig. 1), the concrete strength contribution was assumed to be equal for both zones. An explanation of this could be given by observing that the prestressing action, even in the transfer length zone (TZ test), may be totally diffused in the middle of the experimental zone, where the first crack appeared.

The shear strength contribution due to stirrups, mesh fabric, or fibres, was conventionally assumed, for the other beams, to be equal to the difference between the observed shear strength and the shear strength of Beam 1.

In order to better compare the experimental results and the code provisions, the partial safety factors for the materials (γ_c and γ_s) in the latter formulations were assumed equal to 1.

For all the beams tested, the concrete strength contribution, if estimated according to the "old" edition of EC2 [12] provisions, is almost equal to 75% of the experimental value while, if computed according both to RILEM and the "new" edition of EC2 [14], it is approximately equal to 62% of the experimental value.

Moreover, EC2 [12] and RILEM underestimate the strength contribution offered by steel reinforcement (mesh fabric or stirrups) when the "Standard Method" is adopted. Assuming the EC2 variable angle truss model, the upper limit for the truss provided by EC2 ($\lambda_{max} = \cot\theta = 2.5$) seems too restrictive; in fact, by assuming $\lambda = 4.0$, a more accurate strength prediction is obtained in the DZ zone.

It should be observed that, when the minimum shear reinforcement required by EC2 [12] is provided by mesh fabric alone, the variable angle truss model significantly underestimates the experimental value due the "arc action" that developed for wide opened cracks.

As far as fibre reinforced concrete is concerned, the most recent RILEM provisions [13] account for the fibre strength contribution as an addendum that depends on the post-cracking residual tensile strength of the material that is related to the ultimate limit state ($f_{R,4}$) [13].

In the present research work, SFRC characterization was carried out according to the Italian Standard [15], which is based on four point bending tests on a notched beam having a span of 450 mm and a cross section of 150x150mm. On the contrary, RILEM provisions [13] are based on three point bending tests on beams having a similar span (500 mm instead of 450 mm) and the same cross section with a shorter notch (25 mm instead of 45 mm). Both test methods refer to residual strengths (named as an equivalent post-cracking strength in the Italian Standard) that, although different, are related to serviceability or ultimate limit states. The equivalent strength significant for the ultimate limit state from the Italian Standard is ($f_{eq(0.6\text{-}3.0)}$), whereas the RILEM test refers to $f_{R,4}$.

Fig. 13 shows the experimental load-displacement curves for the four point bending beams tested, 4 cast with 45/30 fibres, and 4 cast with 80/30 fibres. The equivalent strength ($f_{eq(0.6\text{-}3.0)}$) [15] was found to be equal to 2.18 MPa and 8.61 MPa for 45/30 fibres and 80/30 fibres, respectively. An approximate value of the residual tensile strength required by RILEM ($f_{R,4}$) [13] was determined from the results of the four-point bending test by using the procedure suggested by RILEM for the three-point bending test. In doing so, the computed $f_{R,4}$ was equal to 2.36 MPa for 45/30 fibres and to 11.07 MPa for 80/30 fibres. The considerable differences between the two types of fibres (according to both Recommendations) is not reflected in the full-scale beams because of the presence of other shear resistant mechanisms [16], which have a greater impact on the beam response (Table 6).

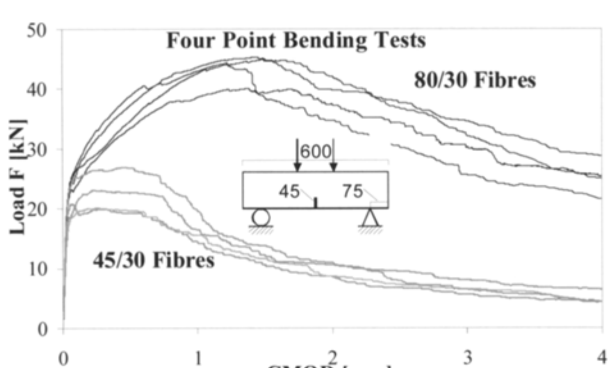

Fig. 13 - Load-CMOD curves from four-point bending tests using 45/30 and 80/30 fibres.

By considering the above mentioned approximation, DZ tests show that the fibre resistance contribution calculated according to RILEM [13], is always smaller than the experimental one, particularly for 45/30 fibres (35 kN versus 135 kN, equal to 26% of the experimental value, Table 6). A better estimation was instead obtained when considering 80/30 fibres (163 kN versus 196 kN, equal to 83% of the experimental value).

The experimental results presented in Table 6 clearly demonstrate that fibres may effectively substitute the required minimum shear reinforcement; in fact, the strength contribution provided by the mesh fabric was found to be equal to 138 kN while that of the fibres was equal to 135 kN for 45/30 fibres and 196 kN for 80/30 fibres. In addition, as already discussed, a better structural behaviour was observed in SFRC beams (Fig. 7b)

Finally, it should be emphasized that the ultimate shear strength for the TZ zone, if calculated according to EC2 [12] by assuming the material partial safety factors ($\gamma_c = 1.5$, $\gamma_s = 1.15$), is equal to 370 kN. This value is considerably smaller than the first cracking load, similar for all beams and it is equal to 56 % of the experimental maximum load (Fig. 7a).

6. CONCLUDING REMARKS

In this paper, the experimental results of shear tests on full scale prestressed beams are presented. The aim of the research is to compare beams with transverse reinforcement and/or a low volume fraction ($V_f = 0.64$ %) of hooked steel fibres, in order to investigate the fibre contribution to shear reinforcement and the possibility of substituting the minimum shear reinforcement with steel fibres.

The experiments simulate both the beam behaviour close to the supports, where stirrups are usually designed to resist shear, and at midspan, where only a minimum shear reinforcement is usually required by building codes.

The experimental results show that the beams reinforced only with steel fibres show a similar, or even better, post cracking behaviour than the beams with the minimum amount of transverse reinforcement: results from six tests (three specimens, DZ tests, for each of the two types of fibres adopted) confirm this important result.

In prefabrication, this is particularly appealing for facilitating the industrialization of the production and introducing an improvement in the overall characteristics and durability of the products.

When fibres are used in addition to conventional transverse reinforcement the shear strength significantly increases. Steel fibres also reduce the width of shear cracks, thus improving also durability.

The shear strength evaluated according to the EC 2 provisions (which do not take into account the contribution of fibres) under the hypothesis of a variable truss inclination model, is lower than the experimental shear force at crack onset.

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