EXPERIMENTAL STUDIES OF CONCRETE BEAMS REINFORCED LONGITUDINALLY WITH STEEL AND GFRP BARS

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Summary:
The presence article is part of research work aiming to contribute to the development of hybrid reinforcement system what combines GFRP (Glass Fiber Reinforced Polymer) and steel bars in an optimized arrangement of the bars, using potentialities that each material con provide. The special experimental studies of concrete beams reinforced with steel–GFRP bars were carried out. Results of the experimental studies indicate that presence of the steel bars contributes considerably to ductility and stiffness, reduces crack width and crack spacing values.

Keywords: flexural steel and GFRP reinforcement; deflection; crack width.

Introduction

The present article is part of research work aiming to contribute to the development of reinforcement system what combines GFRP (Glass Fiber Reinforced Polymer) and steel bars in an optimized arrangement of the bars, using potentialities that each material can provide.

The replacement of conventional steel reinforcement by GFRP bars has been investigated (Toutanji, Saafi 2000; Abdalla 2002) to prevent the corrosion problem and to improve the durability of concrete structures near marine environments, near the ground, in chemical and other industrial plants, in places where good quality concrete cannot be achieved, and in the thin structural elements. Was shown (Taheri et al 2009) in comparison with steel, GFRP materials have higher resistance to corrosion, and higher strength-to-weight ratio. Furthermore GFRP materials are non-electrical conductive and non-magnetic material (ACI440R-07). However, how it was shown in (Aiello, Ombres 2002) the major obstacles of the application of GFRP bars as a reinforcing material for concrete structures are relatively high initial costs, low modulus of elasticity, lack of ductility (linear stress-strain relationship up to rupture with no discernible yield point), and absence of well-consolidated design guidelines. Concrete members reinforced with GFRP and subjected to bending moments behave linearly up to cracking, and almost linearly after cracking with significant lower stiffness (Taheri et al 2009).

Deflections of concrete members reinforced with GFRP bars are generally large than the members reinforced exclusively with steel reinforcement. This is due to the low modulus of elasticity and different bound characteristics of GFRP reinforcement (Taheri et al 2009). In addition, as a result of a larger crack width and smaller compressive stress blocks when using GFRP bar for the flexural reinforcement, the shear capacity of GFRP-reinforced concrete beams is smaller than in the case of steel-reinforced concrete beams of the same reinforcement ratio (Taheri et al 2009).

In an attempt to overcome these drawback, actual studies proposed a combination of GFRP and steel reinforcement for concrete beams. As was shown by Aiello and Ombres (Aiello, Ombres 2002) with this combination of the reinforcement materials, and considering the minor (minimum required for bond) concrete cover required for GFRP, an effective solution in term of durability is obtained by placing GFRP bars near the outer surface of the tensile zone and steel bars at the inner level of the tensile zone. The presence of the steel bars in the above mentoned hybrid reinforcement systems contributes significantly to ductility and stiffnes.

Experimental studies of hybrid reinforced beams

The special experimental study of concrete beams reinforced hybrid (steel and GFRP bars) reinforcement were carried out. Two series of concrete beams of rectangular cross-section, of width b = 120 mm, height h = 190 mm and length l = 2070 mm, reinforced with different percentage of tensile longitudinal steel and GFRP bars were tested in four point loading configuration (see fig. 1). This figure also represent the geometry of the beams, the reinforcement arrangement, as well ass the loading and support condition. In table 1 are indicated the values of geometrical and material parameters of beams. Figure 2 shown a perspective view of the test setup.

The test program has been drawn up in such a way that the total amount of reinforcement, what is estimated mechanical reinforcement index ω $\left(\omega = \dfrac{f_{yk} \cdot \rho_l + f_{yk(f)} \cdot \rho_{l(f)}}{f_{cm}} \right)$ that achieve approximately equal values of the ultimate bending moment (M_u) and tensile reinforcement failure mode.

Up to near the failure of the control beam I-B1 reinforced exclusively with steel bars tensile strains in reinforcement exceeded the yielding value e_{sy} = 2,5 ‰ and tensile strain $\varepsilon_{s,exp}$ = 9,8 ‰ nave bin attained. Concrete compressive strain was near ultimate value εcu ≅ 3,5 ‰. Beam I-B1 failure occurred as a result of the achievement of yield strains of the steel reinforcement.

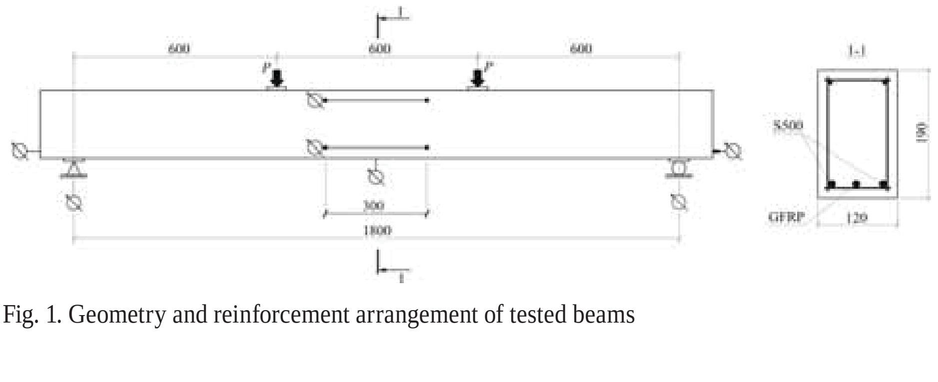


Fig. 1. Geometry and reinforcement arrangement of tested beams

Tab. 1. Program of experimental research

Serie	Beam	Cross-section size, mm	Mean concrete compressive strength f_{cm}, MPa[2)]	Reinforcement					
				number	area, mm^2		reinforcement ratio, %		
					A_s	$A_{s(f)}$	ρ_{st}	ρ_p	w
I	I-B1(t)[1)]	120×190	37,6	2∅12S500	226,2	–	0,99	–	19,8
	I-B2(t)			2∅10S500 + ∅8GFRP	157,1	50,3	0,69	0,22	22,6
	I-B3(t)			2∅8S500 + ∅10 GFRP	100,5	78,5	0,44	0,34	22,6
	I-B4(t)			2∅6S500 + ∅12 GFRP	56,5	113,1	0,25	0,50	24,8
	I-B5(t)			2∅4S500 + 2∅10 GFRP	25,1	157,1	0,11	0,69	29,8
II	II-B2		29,2	2∅10S500 + ∅8 GFRP	157,1	50,3	0,69	0,22	22,6
	II-B3			2∅8S500 + ∅10 GFRP	100,5	78,5	0,44	0,34	22,6
	II-B4			2∅6S500 + ∅12 GFRP	56,5	113,1	0,25	0,50	24,8
	II-B5			2∅4S500 + 2∅10 GFRP	25,1	157,1	0,11	0,69	29,8

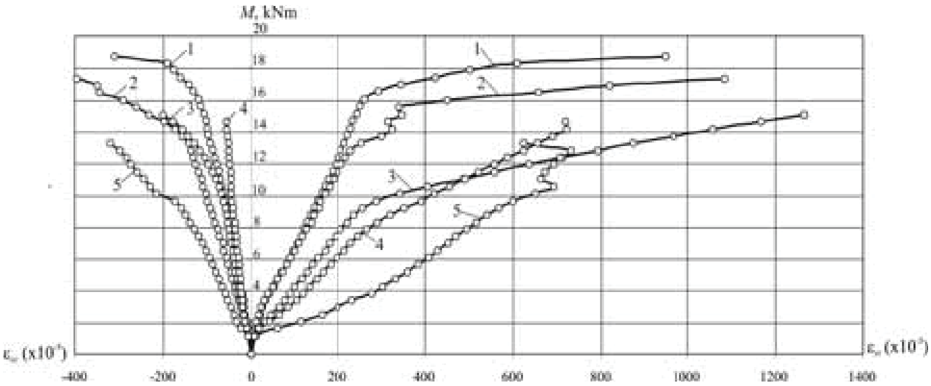
Notes:
1) index (t) means that the manufacture of the tested beams heat treatment was applied (e.g. by system);
2) the mean compressive strength of concrete was tested on cube speciment.

Fig. 2. General view of the test setup

Figure 3 represent the “load – strain” and “moment – deflection” diagrams respectively.

The ductility ratio which represents a ratio between the ultimate and yielding deflections of the beam I-B1 at midspan point ($\eta = a_u / a_y$), was equal to η = 2,44 (see table 2). The same ductility behavior showed concrete beams I-B2 and I-B3 reinforced with steel and GFRP bars (2Æ10 S500 + Æ8 GFRP, ω = 0,226).

The failure of the beams I-B2 and I-B3 occurred as a result of reaching the yield strains in the steel reinforcement and then the crushing of concrete in compression zone of the section (loading process ended due to the failing of concrete in compression zone of section, where the ultimate compression strains was attained).



1 – I-B1; 2 – I-B2; 3 – I-B3; 4 – I-B4; 5 – I-B5

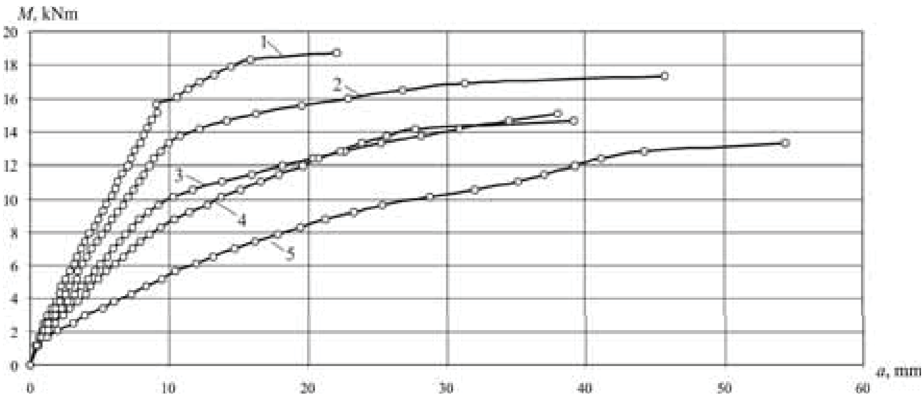
Fig. 3. Load-strains response for tested beams

Tab. 2. Results of experimental studies

Serie	Beam	Cracking parameters			Deflection, mm				M_{Ru}, kN×m
		M_{crc}, kN×m	w_{max}, mm	w_m, mm	$a_{0,6}$	a_y	a_u	$\eta = a_u / a_y$	
I	I-B1	6,53	0,20	0,17	6,47	9,05	22,05	2,44	18,76
	I-B2	4,27	0,25	0,20	7,42	9,93	45,76	4,61	17,35
	I-B3	2,05	0,75	0,63	8,19	8,51	38,06	4,47	15,05
	I-B4	2,05	0,70	0,62	10,40	7,90	39,23	4,97	14,60
	I-B5	1,60	1,00	0,79	17,81	6,09	54,42	8,94	13,25

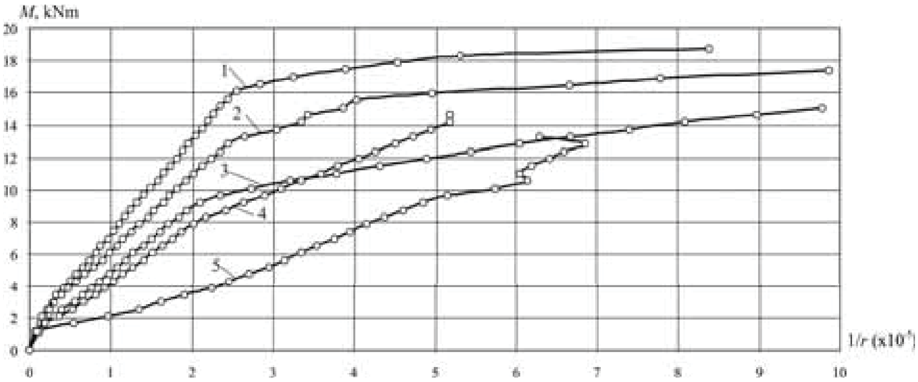
w_{max}, w_m – maximum and mean (average) crack width at loading level M / M_{Ru} = 0,6;
$a_{0,6}$, a_y, a_u – midspan deflection for different loading level: 0,6 M_u, M_y and M_u, respectively;
M_{crc} – cracking moment; M_{Ru} – ultimate moment.

For beams I-B2 and I-B3 value of ductility ratio was η = 4,61 (see figures 4, 5, table 2) and ultimate deflections were twice more that the deflection of the control beam I-B1. Up to near the failure tensile strain in steel reinforcement was equal to $\varepsilon_{s,exp}$ = 11 ‰, and strain in compression concrete was $\varepsilon_{cu} \cong$ 4 %.



1 – I-B1; 2 – I-B2; 3 – I-B3; 4 – I-B4; 5 – I-B5

Fig. 4 – Moment-deflection response for the tested beams



1 – I-B1; 2 – I-B2; 3 – I-B3; 4 – I-B4; 5 – I-B5

Fig. 5. Moment-curvature response for the tested beams

As shown in figure 6 the bulk of the total tensile force in reinforcement is perceived by steel bars (78 kN) and only 28 % of the total force takes GFRP bars.

Ultimate bending moment (M_u = 15,05 kN×m) was smaller for the concrete beams I-B3 (2Æ8 S500 + Æ10 GFRP) and I-B4 (2Æ6 S500 + Æ12 GFRP) than for the reference beam I-B1, but the ductility ratio was almost the same (see figures 4, 5) (experimental value of tensile strain in steel reinforcement was equal to $\varepsilon_{su,exp}$ = 12 ‰ and $\varepsilon_{su,exp}$ = 9 ‰ for beams I-B3 and I-B4 respectively).

In table 2 are indicated the values of cracking and ultimate bending moments, deflections and ductility ratios for tested beams.

As figures 4, 5 illustrates, the yield strains in steel reinforcement was achieved at comparatively low loading rate (M_y / M_u) ≅ 0,3. After yielding of steel reinforcement as the bending moment increases, as the tensile force in GFRP bars is increased practically linearly (see figure 7). The tensile force in the steel bars and GFRP bars at ultimate stage was almost equal for beam I-B3, but for beam I-B4 tensile force in GFRP bars was near 45 % greater than tensile force in steel bars. For beam I-B2 yield strain in steel reinforcement was achieved at loading rate (M_y / M_u) ≅ 0,6.

Figures 4, 5 is illustrated relationships "*M* – 1/*r*" and "*M* – *a*" for tested beams. As shown in figures 4, 5 relationships "*M* – 1/*r*" and "*M* – *a*" can be idealized trilinear diagram. The first range of curves is characterized elastic behavior of concrete beams before cracking. The second branch with different slope is characterized post-cracking behavior, and third branch with smallest slope is characterized behavior of concrete element after yielding of steel reinforcement.

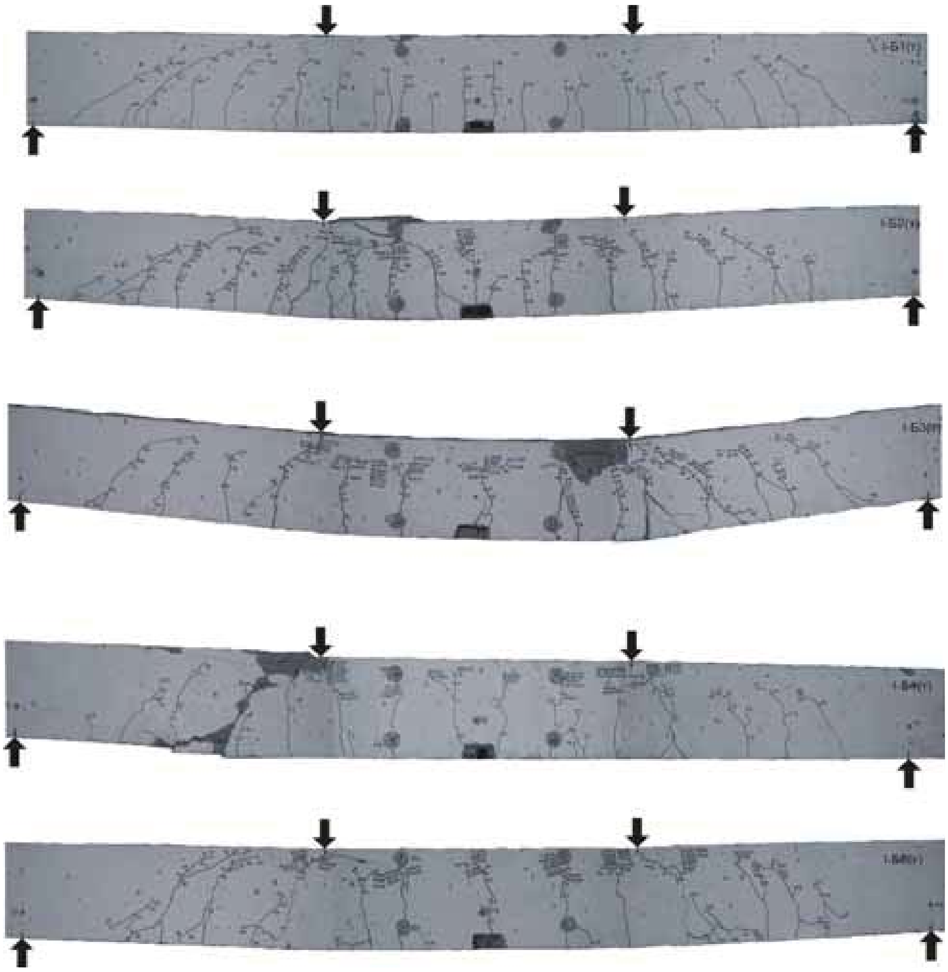
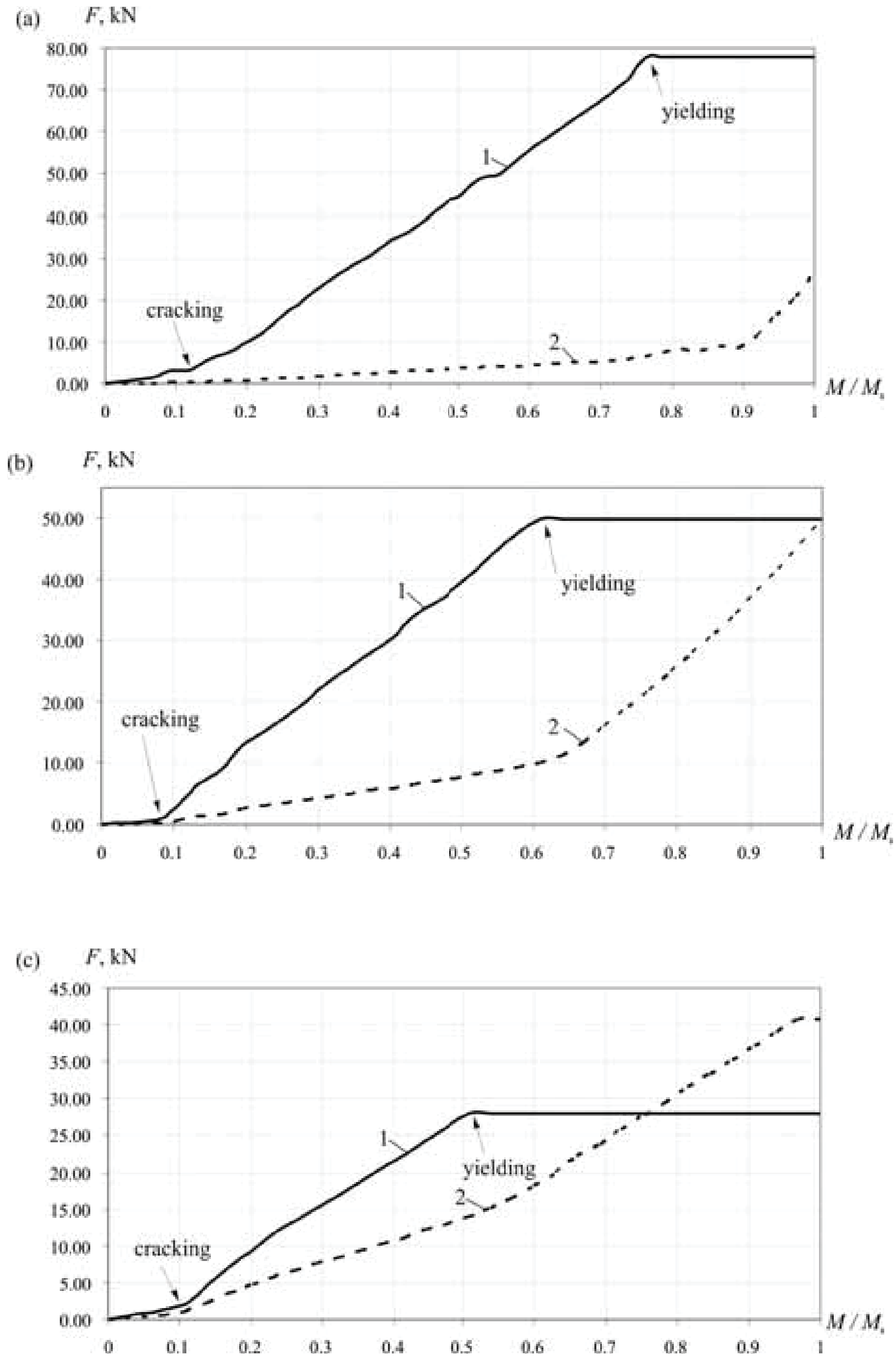
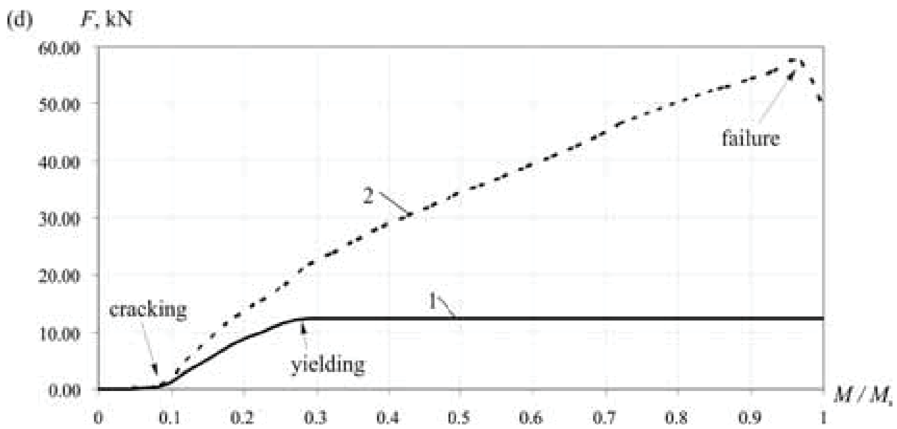


Fig. 6. Crack pattern for the tested beams

The different slopes are explained by crack width grown (see figure 10), increasing of deflections and as a result decreasing of bending (flexural) stiffness (see table 2). Figure 10 represent the crack distribution on the length of concrete beams after failure. As shown in diagram (figure 8) which represents ratios between mean and maximum cracking width, for all tested beams, except of beam I-B2, maximum crack width exceeded a limit value ω_{lim} = 0,3 mm in accordance with EN 1992–1. For tested beams I-B3 and I-B4 it can be explained by early achievement of yielding strains in steel bars. But in comparison with the beam I-B4 reinforced mainly with GFRP bars, maximum crack width for beams I-B3 and I-B4 was from 24 % to 26 % lower at the same loading condition.


(a)
F, kN
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
M / Ms
yielding
cracking
1
2
(b)
F, kN
50.00
40.00
30.00
20.00
10.00
0.00
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
M / Ms
yielding
cracking
1
2
(c)
F, kN
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
M / Ms
yielding
cracking
1
2



(a) I-B2; (b) I-B3; (c) I-B4; (d) I-B5

Fig. 7. Redistribution of the tensile force between steel (1) and GFRP (2) bars as the load increases

In fact, it can be concluded that the deflection of concrete beams reinforced with GFRP and steel bars was smaller than that pf beams reinforced exclusively with GFRP. In comparison with beam exclusively reinforced with GFRP bars, the presence of steel bars reduces crack width and crack spacing value (see figures 7, 8).

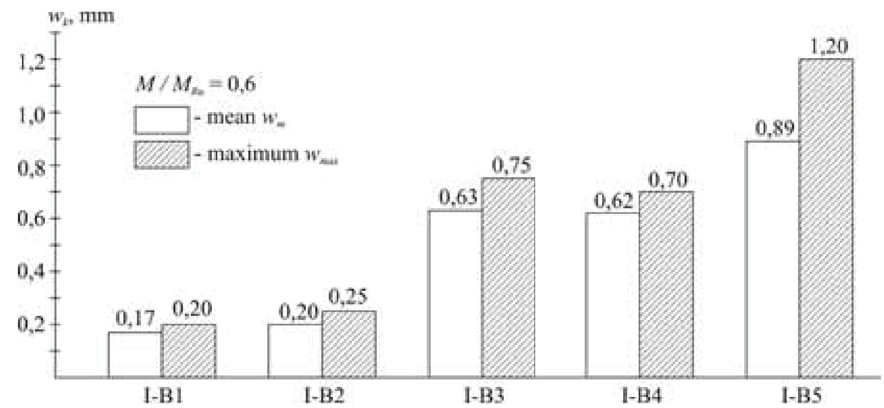


Fig. 8. The experimental values of the mean (average) and maximum crack width for the tested beams (loading level M / M_{ru} = 0,6)

It should be noted that all the tested beams reinforcement bars are located equidistant one from the top fiber. In-fact non-corrodible GFRP bars with height tensile strength, but with brittle behavior and low modulus of elasticity are near bottom fiber, while ductile steel bars have a concrete cover thickness that assures a high protection from the effects of corrosive agents (Taheri et al 2009). Reinforcement ratio of steel reinforcement and its location in the cross-section should be designed in such a way that requirements were provided restrict deflection, width of cracks and structural safety, even in case of a fire occurrence and the consequent loss of GFRP reinforcing capacity. For the optimization of the reinforcing systems for given type of beams (the location of the steel and GFRP bars in the cross-section) a currently running a special parametric studies, based on analytical model (Aiello, Ombers 2002).

Conclusions

Results of experimental studies indicate that presence of the steel bars in the above mentioned hybrid reinforcement system (steel bars + GFRP bars) contributes significantly to ductility and stiffness. It was shown that the deflection of concrete beams reinforced with steel and GFRP bars was smaller than that of beams reinforced with GFRP, and the presence of steel bars reduces crack width and crack spacing values.

The hybrid reinforcement system (steel + GFRP bars) is a competitive solution whn the long term costs with repairing activities are also taken into account (Taheri et al 2009).

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