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Behaviour of GFRP-steel reinforced I shape beams with steel fibers as shear reinforcement

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Abstract
This paper evaluates the possibility of developing prefabricated beams without stirrups by using fiber reinforcement for increasing the concrete shear capacity, and a hybrid flexural reinforcement system composed of glass fiber reinforced polymer (GFPR) and steel rebars. A high compressive strength and high post-cracking tensile capacity steel fiber reinforced self-compacting concrete (SFRSCC) was developed, aiming at supressing the need of steel stirrups in this type of beams while providing sufficient ductility for structural applications. The experimental results were analysed in terms of failure mode, deformational and cracking behaviour, as well as load carrying capacity. A constitutive model, capable of simulating three types of material nonlinearities simultaneously in an integration point (IP), was used and its predictive performance was assessed by simulating the experimental tests. The numerical approach was then used to assess the potentialities of this material system and structural concept when applied to relatively large span beams.

Keywords: Steel fiber; Hybrid steel-GFRP reinforcement; Mechanical testing, FEM analysis
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Introduction
Corrosion of steel stirrups is one of the most common causes that limits the long-term performance of reinforced concrete (RC) structures, since these conventional shear reinforcements are generally placed with the closest proximity to the exterior surface of the elements. Available research suggests that steel fibers can substitute partially, or even totally, the conventional shear reinforcements. Corrosion of the steel flexural reinforcement is also responsible for deterioration and damage process in RC structures. Fiber reinforced polymers (FRPs) can be an alternative flexural reinforcement for the development of more durable RC structures, due to their immunity to corrosion and high strength-to-weight ratio. However, FRPs have a relatively low modulus of elasticity in comparison with the one of steel reinforcements. FRP reinforced concrete beams have larger deflection and wider cracks compared to that of steel reinforced concrete elements. Moreover, the FRP reinforced concrete structures exhibit a brittle failure. To address these problems, application of steel bars as an additional reinforcement is suggested resulting a hybrid reinforcement system [1]. The present study aims to propose a design methodology for prefabricated concrete beams flexurally reinforced with hybrid systems, and using steel fibres as exclusive transverse reinforcement for a concrete with self-compacting requisites.

Experimental program
Tailoring and characterization of developed concretes
Based on a mix design methodology [2], SFRSCC composition with 90 kg/m^3 hooked ends steel fibers, and a reference self-compacting concrete (0% fibers), designated as SCC, were developed. Both compositions were designed to have nominal slump flow of about 660 mm and to pertain to the C50 strength class. For the SCC concrete, an average Young's modulus (E_{cm}) of 32.10 GPa and average compressive strength (f_{cm}) of 66.45 MPa were obtained, whereas values of E_{cm} = 33.23 GPa and f_{cm} = 67.05 MPa for the SFRSCC specimens were determined. The flexural behaviour of these two concretes at 28 days age was obtained by testing three notched beams per concrete type. By taking the characteristic

values of the residual flexural strength parameter at 0.5 mm (f_{R1k}=14.95 MPa) and at 2.5 mm (f_{R3k}=14.08 MPa), the toughness of the SFRSCC concrete composition is classified as "13c". Additional details regarding the SFRSCC and the SCC compositions can be consulted elsewhere [3].

Test specimens and design methodology

Four quasi-real scale *I*-cross section beams were fabricated and studied in terms of load carrying capacity, deformational and cracking behaviour. Three of the beams, namely F1.1-NW-2G12 (Sec. 1 of Fig. 1), F0-NW-2G12 (Sec. 1), and F0-W-2G12 (Sec. 2) have the same flexural reinforcement: one steel strand (with cross section of 140 mm^2, yield strength of f_{sy} = 1740MPa and elasticity modulus of E_s = 200 GPa), and two GFRP rebars of 12 mm diameter with ribbed surface, tensile strength (f_u) of 1350 MPa and elasticity modulus (E_f) of 56 GPa. For each of these three beams, total prestressing force of 238 kN were applied to the flexural reinforcement. This prestressing force corresponds to 30% of f_u and 56% of f_{sy}. The beam F1.1-NW-2G12 was designed by adopting a flexural reinforcement ratio (the ratio calculated for the hybrid GFRP-steel longitudinal reinforcement, ρ_{flex}=0.4%) higher than the hybrid balanced reinforcement ratio of this beam (ρ_{fd}=0.3%), which can be calculated according to the formula proposed by [4].

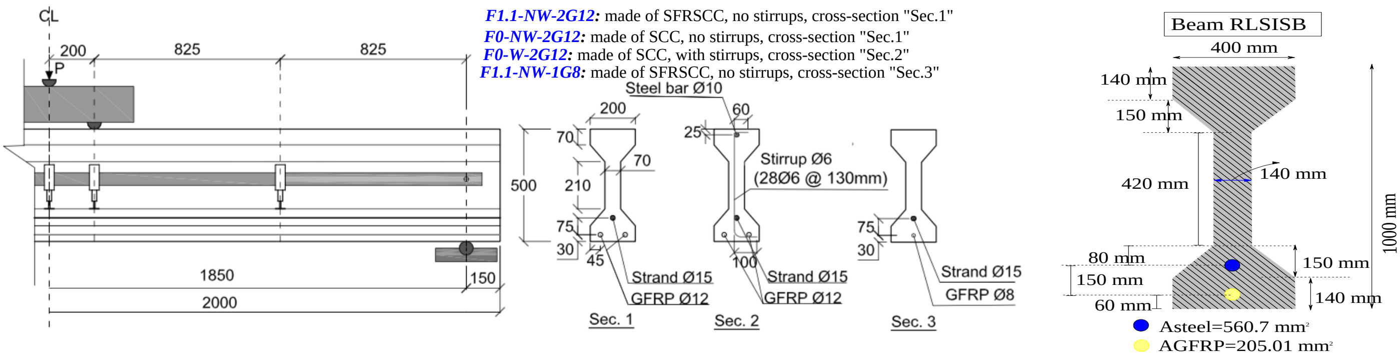

Figure 1: Configuration and test setup of the beams (dimensions in mm).

The same ρ_{flex} is adopted for the beams F0-NW-2G12 and F0-W-2G12 as well. The F1.1-NW-1G8 beam was reinforced with a lower reinforcement ratio, by adopting a GFRP rebar of 8 mm diameter, and the same steel strand, applied for fabricating the rest of the beams. This GFRP bar had f_u =1500MPa and E_f = 50 GPa. This beam was prestressed at 80% f_{sy} for the steel strand and 30% f_u for the GFRP bar, providing a total prestressing force of 218 kN (almost the same force as the total prestressing force applied for the other three beams).

Experimental results and discussions

The F0-NW-2G12 and F0-W-2G12 failed by diagonal tension failure mode (Fig. 2). By loading F0-NW-2G12, initially the flexural and diagonal cracks developed, but the diagonal cracks propagated and grown more rapidly due to the absence of shear reinforcement for resisting to the quick degeneration of these shear cracks in the critical one, which was followed by an abrupt load decay. The cracking behaviour of the F0-W-2G12, shear reinforced with steel stirrups, was characterized by the development of several inclined cracks, which caused the yielding of the stirrups crossed by the shear failure crack. The SFRSCC beam F1.1-NW-2G12 developed a more diffuse crack pattern composed initially by flexural cracks, and by diagonal cracks in later stages of the loading process, and finally failed with the propagation of in-plane shear crack at the transition between the bottom flange and the web. This shear-tension failure, was accompanied by the formation of horizontal splitting cracks along the steel strand towards the supports of the beams. Comparing the F0-W-2G12, reinforced with steel stirrups, with F1.1-NW-2G12 made by

SFRSCC, it is verified that in the former beam a smaller number of cracks with larger distance were formed, while in the SFRSCC beams, the reinforcement provided by steel fibers is the responsible for the development of larger number of cracks of smaller spacing and width, providing to this beam a higher ductility and energy dissipation in the fracture process. The F1.1-NW-1G8 failed in bending with a very low number of cracks compared to the rest of the beams. All the beams presented a relatively high deflection at failure, which was more than twice the deflection of these beams at serviceability limit state (SLS). TheF0-NW-2G12 presented abrupt load decay just after the peak load. An almost similar load at SLS, F_{SLS}, was obtained by testing the beam "F0-W-2G12" with conventional shear reinforcement. The stirrups made the beam to be capable of continuing supporting load/deflection higher than those values registered at critical shear crack formation (about 37.7 mm). However, it still exhibited a brittle failure that occurred due to the rupture of stirrups. F1.1-NW-2G12 16% increase in the F_{SLS} compared to the F0-NW-2G12. The F_{SLS} of 223 kN obtained in the case of F1.1-NW-2G12 indicates that this type of beams can be adopted in pre-fabrication for constituting structural systems.

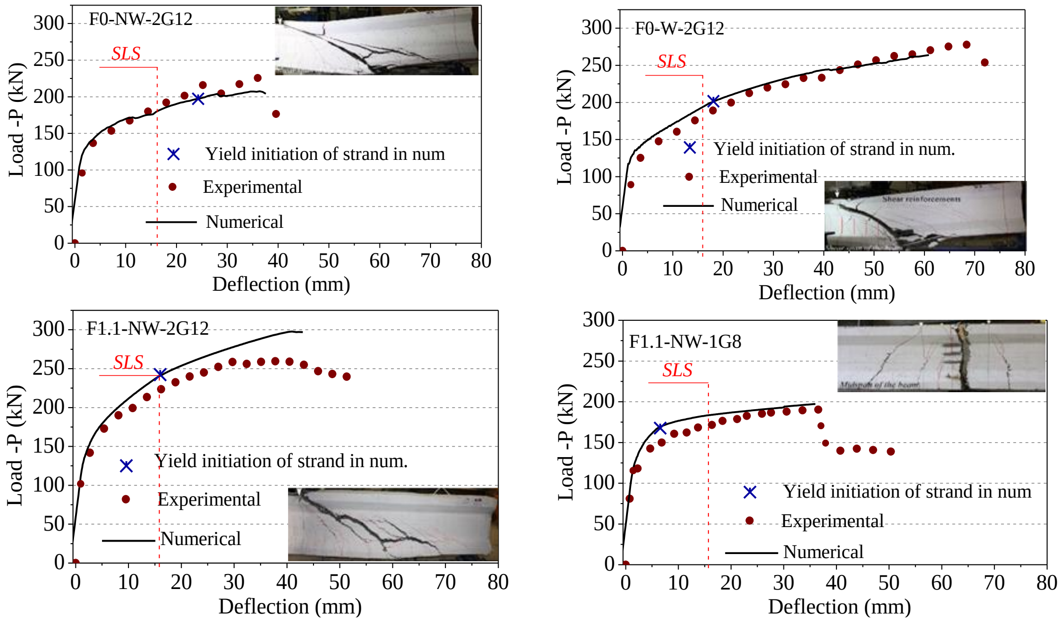

Figure 2: Deflection and crack pattern (at failure stage) of the beam specimens.

The increase of maximum load capacity was 17% in the case of the beam reinforced with stirrups, F0-W-2G12, when compared to F0-NW-2G12. The F1.1-NW-2G1 presented an increase of 20% in the F_{SLS}, when compared to that of F0-W-2G12. The results also show a small difference (7%) in maximum load capacity between F1.1-NW-2G12 and F0-W-2G12 (with conventional stirrups). The F1.1-NW-1G8 presented a relatively high load carrying capacity at SLS, which was close to the F_{SLS} of F0-W-2G12 (beam with steel stirrup). For this beam, the sudden drop of load at around 40 mm deflection (failure stage) can be attributed to the rupture of GFRP bar. Results of the strain gauges installed on the GFRP bars of the other three beams indicated that these bars did not rupture once beam failure occurred.

Numerical simulations

The plastic damage smeared crack (PDSC) model [5,6], capable of simulating material nonlinearities due to fracture mode I-II, and compression, was used to simulate the behaviour of the developed beams. Detailed approach for obtaining values of the model parameters is described elsewhere [3,7]. For all these beams except F1.1-NW-1G8, the analyses were interrupted when the crack pattern demonstrates the eminence of structural collapse, which is in general followed by difficulties in assuring convergence due to the formation of failure mechanisms. The simulation ofIn F1.1-NW-1G8 was terminated when the tensile strength in GFRP cable was attained, since the failure of this beam was governed by failure of GFRP bar. Fig. 2 shows that the model is able of capturing with good accuracy the deformational response of all the beams. The mean value of measured-to-predicted maximum load carrying capacity ratios was μ=1.03, with a standard deviation of of S=0.098. Based on an assumed normal distribution function, there is a direct relationship between μ, S, the global safety factor (φ), and a percentage of design strength (Fractile level ,F), that is expected to be less than φ × ultimate load capacity [8]. Considering F=5% (Fib Bulletin 45 [8]) and taking μ=1.03, S=0.098, the value of φ was calculated as 0.84. The numerical tool was applied to assess potentialities of the proposed material system and structural concept when applied to relatively large span beams. For this analysis a beam (called as RLSISB) of 12 m length with 11 m length between the supports, and a cross section according to Fig. 1, was modelled using the PDSC model. The reinforcement ratio for this beam was the same as that of the beam F1.1-NW-1G8. The beam RLSISB is subjected to its dead load plus a live load (applied uniformly on top surface of the beam). For this analysis, values of parameters of constitutive models of concrete/reinforcement were the same as those used for the beam F1.1-NW-1G8. The maximum load capacity (vertical reactions in the supports) for the beam RLSISB was predicted as 900.23 kN. Considering the already obtained φ (0.84), the design maximum load capacity for the beam RLSISB is: 0.84×900.23=756.2 kN. This design loads corresponds to application of 68.7 Kn/m uniformly distributed load.

Conclusions

This study presents a new generation of RC beams without stirrups by using fiber reinforcement for increasing the concrete shear capacity, and a hybrid flexural reinforcement system of GFPR-steel rebars. Several RC beam were developed and their behaviour were studied experimentally and numerically. This study indicates this type of SFRSCC beams can be adopted in pre-fabrication for buildings with industrial or commercial activities.

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