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DURABILITY AND SUSTAINABILITY OF FIBRE REINFORCED POLYMER (FRP) COMPOSITES FOR CONSTRUCTION AND REHABILITATION

DURABILITÉ ET CYCLE DE VIE DES MATÉRIAUX COMPOSITES EN POLYMÈRES RENFORCÉS DE FIBRES (PRF) EN CONSTRUCTION ET RÉHABILITATION

EFFICIENCY OF DIFFERENT FRP-BASED FLEXURAL STRENGTHENING TECHNIQUES IN BEAMS SUBMITTED TO FATIGUE LOADING

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ABSTRACT

Flexural strengthening of reinforced concrete (RC) structures with fiber reinforced polymers (FRP) has been used mostly by two main techniques: Externally Bonded Reinforcement (EBR) and Near-Surface Mounted (NSM). In both strengthening techniques the FRP systems are applied on the cover concrete, which is normally the weakest region of the element to be strengthened. Consequently, the most common problem is the premature failure of the system, which occurs more frequently when using the EBR technique. In an attempt of overcoming this weakness, another flexural strengthening technique, named MF-EBR – Mechanically Fastened and Externally Bonded Reinforcement, is analyzed in the present paper. This technique uses multidirectional carbon fiber laminates that are simultaneously glued and anchored to concrete. To compare the efficiency of NSM, EBR and MF-EBR techniques, four-point bending tests with RC beams were carried out under monotonic and fatigue loading. In this work the tests are described and the obtained results are presented and discussed.

1. INTRODUCTION

Fiber reinforced polymers (FRP) are gaining acceptance in the rehabilitation and/or strengthening of existent structures since they are good alternatives to traditional strengthening systems, due to their high stiffness and tensile strength, low weight, good fatigue behavior, immunity to corrosion, and geometric versatility. The FRP's are used, mainly, by two strengthening techniques (ACI 2008): the Externally Bonded Reinforcement (EBR) and Near-Surface Mounted (NSM). The EBR technique has been used to increase the flexural and the shear resistance of reinforced concrete (RC) elements, as well as the concrete confinement, and to control the cracking process. Despite the advantages of this technique, the bond between FRP and concrete surface can be susceptible of degradation, particularly due to environmental conditions such as fire, high temperatures, UV radiation, humidity and even vandalism acts. The NSM technique was proposed as an alternative strengthening technique and, when compared to the EBR, several advantages can be pointed out, mainly: the amount of site installation work and the surface preparation may be reduced; the NSM technique is less prone to the debonding from the concrete substrate; the FRP reinforcements can be more easily anchored into adjacent members (preventing debonding failures); the FRP reinforcements are protected by the concrete cover and, consequently, less exposed to mechanical damage and impact loading, and less exposed to the fire and vandalism acts. Furthermore, the aesthetic of the strengthened structure is virtually unchanged (De Lorenzis and Teng 2007). Several studies, however, have shown the frequent occurrence of FRP debonding when the EBR (CNR 2004) is used, and concrete cover rip-off failure mode when NSM technique is applied (Barros and Fortes 2005). Besides being both fragile failure modes, they do not allow the adequate exploitation of the tensile potentialities of FRP systems (the maximum stress installed in the FRP at failure of the strengthened element is much lower than its tensile strength).

In alternative to EBR and NSM, a quite new technique, called Mechanically Fastened *FRP* (MF-FRP), has been proposed based on the use of steel fasteners applied along the laminate's length. The application of the MF-FRP technique in the flexural strengthening of RC elements improves the flexural capacity with little or no loss in ductility (Martin and Lamanna 2008). In the last decade the MF-FRP technique has been investigated by several researchers and some benefits have been pointed out, namely, quick installation with simple hand tools, no special labor skills are needed, no surface preparation is required, and the strengthened structure can be used immediately used after installation of the FRP (Elsayed et al. 2009, Lee et al. 2009, Bank and Arora 2007, Quattlebaum et al. 2005, Lamanna et al. 2004). Nevertheless, some notable disadvantages of this system have been observed, including scale effects, cracking induced by the impact of fasteners in high-strength concrete, and less-effective stress transfer between the FRP and concrete due to the discrete attachment points (Ray et al. 2000).

Based on the MF-FRP technique, the Mechanically Fastened and Externally Bonded Reinforcement (MF-EBR) technique has been proposed. The MF-EBR combines the fasteners from the MF-FRP technique with the externally glued properties from the EBR. In addition, the fasteners are pre-stressed and multidirectional laminates exclusively made with carbon fiber reinforced polymers (MDL-CFRP) with high longitudinal tensile strength, modulus of elasticity and bearing strength are used in order to mobilize high levels of efficiency (Sena-Cruz et al. 2010a). To assess the efficiency of EBR, NSM and MF-EBR techniques, four-point bending tests with RC beams were carried out under monotonic and fatigue loading. The tests are described and the results are presented and discussed in detail.

2. EXPERIMENTAL PROGRAM

2.1 Specimens and Test Configuration

The experimental program was composed of two series of four beams each, being the distinction between the series associated to the loading configuration: one subjected to monotonic loading and the other to fatigue loading. Each series was composed of a reference beam (REF), and a beam for each investigated strengthening technique. The RC beams have a cross section of 200 mm wide and 300 mm height, being 2000 mm the distance between supports. All the beams have three longitudinal steel bars of 10 mm diameter (3Ø10) at the bottom, and 2Ø10 at the top (see Fig. 1). The transverse reinforcement is composed of steel stirrups of 6 mm diameter (Ø6) with a constant spacing of 100 mm in order to avoid shear failure. Fig. 1 includes the cross section of the beams.

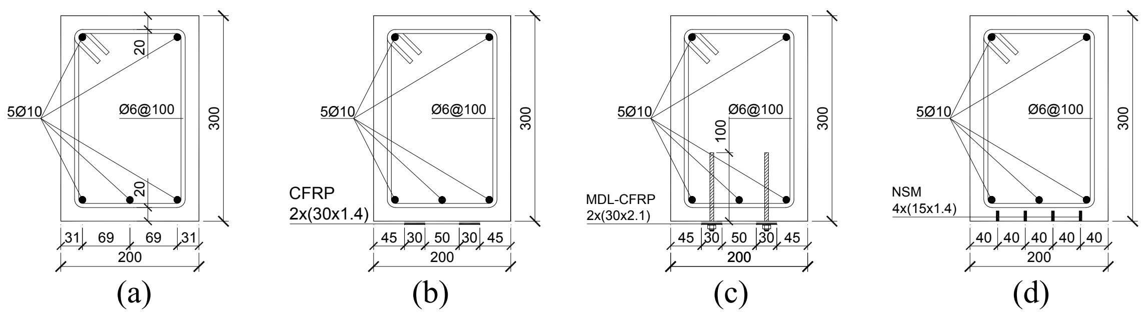


Fig. 1. Cross section of the all beams: (a) REF; (b) EBR; (c) MF-EBR; (d) NSM. Note: all dimensions are in [mm].

Table 1 presents the main properties of the beams. In this table t_f, L_f and w_f are the thickness, the length and the width of the laminates, respectively, and $\rho_{s,eq}$ is the equivalent longitudinal steel reinforcement ratio defined by Eq. 1:

$$\rho_{\mathrm{s,eq}} = \frac{A_s}{bd_s} + \frac{E_f}{E_s} \cdot \frac{A_f}{bd_f} \tag{1}$$

In this equation b is the width of the beam, A_s and A_f are the cross sectional area of the tensile longitudinal steel bars and FRP systems, respectively, E_s and E_f are the modulus of elasticity of steel and FRP, respectively, and d_s and d_f are the distance from the top concrete compression fiber to the centroid of the steel bars and FRP systems, respectively. In all the strengthened beams similar $\rho_{s,eq}$ was applied.

Table 1. Properties of the beams.

Beam	Laminate type	Quantity	t_f [mm]	L_f [mm]	w_f [mm]	$\rho_{s,eq}$ [%]
REF	-	-	-	-	-	0.439
EBR	Unidirectional	2	1.41	1400	30	0.550
MF-EBR	Multidirectional	2	2.07	1400	30	0.553
NSM	Unidirectional	4	1.41	1400	15	0.561

In this experimental study, a four-point loading test configuration was adopted for the monotonic and fatigue tests (see Fig. 2a). A servo-controlled hydraulic system was used to perform the monotonic tests under displacement control, with a deflection rate of 20 µm/s, using the linear variable differential transducer (LVDT) located at the mid-span of the beam (LVDT3 in Fig. 2). The load was applied through an actuator equipped with a load cell of 500 kN of maximum capacity. The fatigue tests were performed between a minimum fatigue level of $S_{\min}$=25% and maximum fatigue level of $S_{\max}$=55%, where the S is the ratio between the applied load and the monotonic beam's load carrying capacity, F_{m}. The fatigue tests were composed by three main steps: initially, a monotonic loading was applied, under force control and at a load rate of 100 N/s up to the maximum level (S_{max}), in order to register the initial response of the specimen; then, 1 million cycles was imposed at 2 Hz of frequency between $S_{min}\times F_m$ and $S_{max}\times F_m$; finally, a monotonic loading up to the failure, with the same configuration of the monotonic tests, was applied to the specimens. In addition to the LVDT3, others four LVDTs were used to record the deflections in the loaded sections (LVDT2 and LVDT4) and at the sections coinciding with the free ends of the FRP systems (LVDT1 and LVDT5), see Fig. 2a. Strain gauges were glued on both the longitudinal steel reinforcement and FRP's to measure the strains during the tests, see Fig. 2b-e.

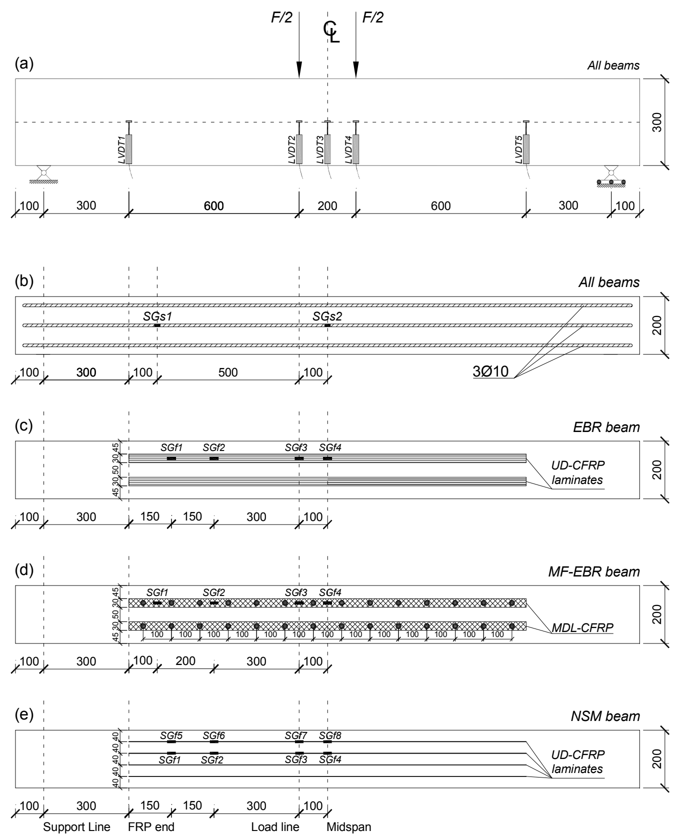


Fig. 2. Instrumentation adopted: (a) vertical deflection; (b) strains on the steel bars; (c) strains on the laminate of the EBR beam; (d) strains on the laminate of the MF-EBR beam; (e) strains on the laminates of the NSM beam. Note: all dimensions are in [mm].

2.2 Material Characterization

The mechanical characterization of concrete was assessed by means of compression tests. For this purpose six cylindrical concrete specimens were tested at the age of the tested beams to evaluate the compressive strength and the modulus of elasticity according to the recommendations NP EN 12390-3:2009 and LNEC E397:1993, respectively. From the compression tests, an average compressive strength value of 53.08 MPa, with a coefficient of variation (CoV) of 4.0%, and an average value of 31.17 GPa (CoV=4.4%) for the modulus of elasticity, were obtained. The age of the concrete beams at the date of experimental program was about two years. The steel of the longitudinal bars and stirrups has a denomination of A400 NR SD according to NP EN 1992-1-1:2010. Additional information related with the experimental characterization of the steel bars can be found elsewhere (Bonaldo 2008).

In this work two different types of CFRP laminates were used: unidirectional (UD-CFRP) for the case of EBR and NSM techniques, and multidirectional (MDL-CFRP) for the case of the MF-EBR technique. Tensile tests were performed according to the ISO 527-4:1997 for both laminates (UD-CFRP and MDL-CFRP) to assess their tensile properties. From these tests it was obtained a tensile strength, a modulus of elasticity and an ultimate strain of 1866 MPa (CoV=5.1%), 118 GPa (CoV=2.8%) and 1.58 % (CoV=5.1%) for MDL-CFRP, and 2435 MPa (CoV=5.8%), 158 GPa (CoV=3.9%) and 1.50 % (CoV=4.7%) for UD-CFRP, respectively (Sena-Cruz et al. 2010a). From the bearing tests with MDL-CFRP specimens performed according to the ASTM D5961/D5961M–05 standard, a bearing strength of 316.4 MPa (CoV=11.8%) and 604.4 MPa (CoV=5.8%) was obtained for the case of unclamped and clamped with a torque of 20 N×m, respectively (Sena-Cruz et al. 2010a). To bond the laminates to concrete the S&P Resin 220® epoxy adhesive was used. According to the supplier, this adhesive has a flexural tensile strength, a compressive strength and a bond concrete/laminate strength of 30 MPa, 90 MPa e 3 MPa, respectively. A Hilti® chemical anchors system was adopted to fix mechanically the laminate to concrete for the case of the MF-EBR beam. This system is composed by the resin HIT-HY 150 max and the HIT-V M8 8.8 threaded anchors. The preparation of the strengthened beams required several steps that are described elsewhere (Sena-Cruz et al. 2010b).

3. RESULTS

Table 2 resumes the main results obtained in the performed tests, while Fig. 3 depicts the relationship between force and displacement at mid-span during the tests. In this table the meaning of the symbols are: δ_{cr}, deflection at concrete crack initiation; F_{cr}, load at concrete crack initiation; $\delta_{y,}$ deflection at the yield initiation of the longitudinal steel bars; F_y, load at the yield initiation of the longitudinal steel bars; δ_{max}, deflection at the maximum load; F_{max}, maximum load; ε_{fu}, ultimate strain in the FRP according to the results obtained in tensile tests; ε_{fy}, maximum strain in the FRP at F_y; ε_{fmax}, maximum strain in the FRP at F_{max}.

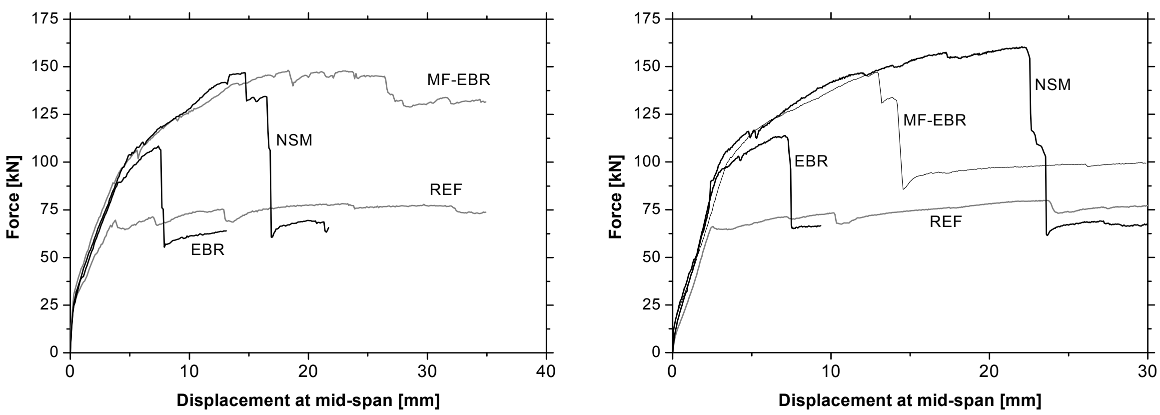
In terms of monotonic testes, it can be concluded that the most effective strengthening technique was the MF-EBR, not only due to the maximum load reached (F_{max}=148.2 kN), but also in terms of deflection at failure and maximum/failure strain ratio in the FRP. When compared with the EBR, the MF-EBR system had an increase of the load carrying capacity of about 37%. This superior behavior cannot be explained by the higher axial stiffness, E_fA_f, of the laminate, since the ratio between the E_fA_f of the MDL-CFRP and E_fA_f of the UD-CFRP (used in the EBR beam) is only 1.08. The pre-stressed anchors have contributed for this higher strengthening effectiveness of MF-EBR technique. In fact, while EBR FRP systems failed by FRP peeling (Fig. 4a), and NSM FRP systems by concrete cover rip-off (detachment of the concrete cover that includes the CFRP strips, Fig. 4b), the MF-EBR FRP laminates failed by bearing (Fig. 4c-d). The presence of the anchors avoided the premature debonding (peeling) of the laminates, as well as the detachment of the concrete cover (rip-off). Defining the level of ductility as the ratio between the deflection at the maximum load and the deflection at the yielding of the longitudinal steel bars (δ_{max}/δ_y), in the MF-EBR beam the δ_{max}/δ_y was equal to 4.35, which was much higher than the values registered in the EBR (1.80) and NSM (2.98) beams. Apparently, in the MF-EBR beam the force corresponding to the crack initiation, F_{cr}, is higher than the F_{cr} of the other beams. This behavior can be explained by the existence of pre-stress. In fact, the pre-stress provided by the anchors may have induced a compressive stress state on the concrete cover which has delayed the concrete crack initiation. This phenomenon could also explain the higher stiffness in the phase between the concrete crack initiation and the steel yield initiation of the MF-EBR beam. After the longitudinal steel bars have yielded, a slight higher stiffness can be observed in the NSM beam, when compared with the MF-EBR beam. This behavior can be justified by the confinement that surrounding concrete provides to the NSM CFRP laminates.

Table 2. Main results obtained in the tests.

Beam	Crack initiation		Yielding		Ultimate		δ_{max}/δ_y	$\varepsilon_{fy}/\varepsilon_{fu}$ [%]	$\varepsilon_{fmax}/\varepsilon_{fu}$ [%]	Failure mode
	δ_{cr} [mm]	F_{cr} [kN]	δ_y [mm]	F_y [kN]	F_{max} [kN]	δ_{max} [mm]				
MONOTONIC										
REF	0.36	29	3.8	70	79.3	22.6	5.95	-	-	-
EBR	0.27	25	4.1	90	108.4 (37%)*	7.4	1.80	24.0	36.6	Peeling
MF-EBR	0.38	32	4.2	96	148.2 (87%)*	18.3	4.35	15.8	69.3	Bearing
NSM	0.40	29	4.9	104	147.3 (86%)*	14.6	2.98	23.4	63.3	Rip-off
FATIGUE										
REF	0.26	20	2.5	66	79.9	23.3	9.32	-	-	-
EBR	0.32	27	3.0	94	114.2 (43%)*	7.1	2.37	14.6	29.6	Peeling
MF-EBR	0.35	31	3.7	101	147.2 (84%)*	12.9	3.49	15.0	63.4	Bearing
NSM	N/A	N/A	3.3	105	160.7 (101%)*	22.2	6.73	15.4	55.7	Rip-off

*$(F_{max}-F_{max,REF})/F_{max,REF}$ where $F_{max,REF}$ is the maximum load of the reference beam in the series.

In terms of fatigue tests, the behavior at crack initiation, was similar to the one observed in the monotonic tests, i.e., the best one was registered in the MF-EBR beam. After the fatigue loading, the NSM was the most effective strengthening technique for both the maximum load (F_{max}=160.7 kN) and ultimate deflection capacity (δ_{max}/δ_y=6.73). When compared with the corresponding monotonic tests, marginal variation in terms of maximum load was obtained for the case of the REF, EBR and MF-EBR beams, whereas an increment of 9% was attained in the NSM beam. No rational explanation can be pointed out for this behavior. The inferior performance of the MF-EBR beam, when compared with the monotonic one, can be attributed to a possible loss of efficiency of the pre-stressed anchorages along the fatigue cycles, and due to a bearing strength degradation of the MDL-CFRP during the cycles. The EBR and NSM beams exhibited the same failure modes occurred in the monotonic tests. Despite the performance in the monotonic tests, the MF-EBR beam presented a more fragile failure mode with bearing and inter-laminar failure of the FRP. Stiffness degradation was also evaluated for the fatigue cycles. Marginal variations were observed. In fact, a decrease of 8.3%, 3.0%, 0.3% and 12.1% in terms of stiffness was observed for the REF, EBR, MF-EBR and NSM beams, respectively.



(a) (b)

Fig. 3. Force *vs*. displacement relationship of the tested beams: (a) monotonic loading; (b) fatigue loading.


(a) (b)

(c) (d)

Fig. 4. Failure modes: (a) EBR beam; (b) NSM beam; (c) and (d) the MF-EBR beam.

4. CONCLUSIONS

In this paper a flexural strengthening technique, called MF-EBR – Mechanically fastened and externally bonded reinforcement, is studied. This technique combines the fasteners from the MF-FRP technique and the epoxy bond-based performance from the EBR technique. In addition, all the fasteners are pre-stressed. Multidirectional laminates exclusively made with carbon fibers reinforced polymers (CFRP) were used. To compare the efficiency of the MF-EBR, EBR and NSM strengthening techniques, an experimental program was carried out, composed of two series of four beams, one submitted to a monotonic loading, and the other to a fatigue loading. For both series the beams were subjected to a four-point bending loading configuration. In the fatigue tests 1 million cycles at a frequency of 2 Hz was applied to each beam. Each series is composed of a reference beam (REF) and one beam for each of the strengthening techniques analyzed: EBR, MF-EBR and NSM. When compared to the reference beam, an increase on the loading carrying capacity of 37%, 87% and 86% was obtained for the EBR, MF-EBR and NSM strengthened beams, respectively. When compared to the EBR beam, an increase of about 37% on the load carrying capacity was obtained for MF-EBR technique. The most favorable aspect of the MF-EBR technique was, however, the normalized deflection capacity at maximum load (4.35), which was much higher than that registered in the EBR (1.80) and NSM (2.98) beams. In addition, more ductile failure mode was observed for MF-EBR technique. In the fatigue tests, after having been subjected to 1 million cycles, the NSM beam has supported the highest ultimate load, corresponding to an increase of 101%, while the MF-EBR and EBR beams presented an increase of 84% and 43% in the load capacity, respectively, when compared with the maximum load of the control beam. In the fatigue tests the NSM beam presented the highest normalized deflection capacity at maximum load (6.7), while a value of 3.5 and 2.4 was registered in the MF-EBR and EBR beams, respectively.

5. ACKNOWLEDGEMENTS

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