Composite Structures 167 (2017) 166–177

Contents lists available at ScienceDirect

Composite Structures

journal homepage: www.elsevier.com/locate/compstruct

Evaluation of the effectiveness of current guidelines in determining the strength of RC beams retrofitted by means of NSM reinforcement

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ARTICLE INFO

Article history:
Received 18 August 2016
Revised 10 January 2017
Accepted 28 January 2017
Available online 2 February 2017

Keywords:
NSM
RC beams
Strengthening
Analytical formulation
Assessment

ABSTRACT

Near surface mounted (NSM) fiber reinforced polymer (FRP) reinforcement represents a valid alternative to externally bonded (EB) FRP reinforcement for strengthening existing reinforced concrete (RC) elements. NSM composites are usually comprised of glass, carbon, and aramid fiber reinforcement with circular (bars) or prismatic (strips) cross-section. When the NSM technique is employed for flexural strengthening, FRP composites are embedded into grooves cut in the concrete cover and filled with an inorganic (e.g. cement grout) or organic (e.g. epoxy resin) binding agent. Although many studies on the bond behavior of NSM FRP composites can be found in the literature, limited work is available regarding analytical models for designing NSM strengthening of RC members. In this paper, a database comprised of 155 experimental tests on RC beams strengthened in flexure using NSM reinforcement is collected from the scientific literature. The experimental database is employed to assess the accuracy of the analytical provisions obtained following the American, English, Canadian, and Italian guidelines. The assessment shows that, although conservative, the analytical models considered are (relatively) poorly accurate and further studies are needed to provide a reliable tool for designing RC flexural strengthening with NSM reinforcement.

1. Introduction

In the last decades, fiber reinforced composite materials have gained increasing popularity in the civil engineering industry. Among them, fiber reinforced polymer (FRP) composites have been employed to strengthen existing reinforced concrete (RC) and masonry elements. FRP composite strips are largely applied to existing elements as externally bonded (EB) reinforcement (see e.g. [1]). A valid alternative to EB FRP systems is represented by near surface mounted (NSM) FRP systems. In the NSM method, grooves are first cut into the concrete cover of an RC element and the added reinforcement is bonded therein with an appropriate groove filler (typically epoxy paste or cement grout) [2].

The first applications of the EBR technique, made with steel plates, are half a century old [3–5]. Before that, strengthening made by means of near-surface mounted (NSM) steel rebar had already been adopted in practice [6]. However, the NSM method had a limited success with respect to externally bonded reinforcement applications. One of the main reasons why externally bonded reinforcement was preferred to near-surface mounted lies in the possibility to increase the steel plate-to-concrete bond by means of bolts (adequately anchored into the concrete substrate) placed along the length of the plate – usually along the anchorage length – to avoid debonding failures. This solution guarantees the effectiveness of the external reinforcement, which in turn allows for attaining concrete crushing failure.

EBR systems made with steel plates is an effective solution but suffers of some disadvantages, among which the problem of corrosion of the steel plates and the difficulty in manipulating heavy and long plates in construction sites are the most important. Replacing the steel plates with fiber reinforced polymer (FRP) sheets provides a satisfactory solution to the problems described above and, because of this, has been increasingly studied an adopted starting from the eighties of the last century [7]. Nevertheless, the adoption of FRP (mainly carbon fiber reinforced polymer, or CFRP) sheets does not generally allow for using anchor bolts because of the mechanical properties of the composite sheet. This fact implies that the efficiency of an externally bonded FRP reinforcement is limited by the delamination of the composite, which can be related either to debonding of the sheet from the substrate with a relatively thin layer of concrete attached to it or to splitting of the entire concrete cover at the level of the tensile reinforcement [1].

More recently, near-surface mounted (NSM) FRP reinforcement has attracted an increasing amount of research as well as practical

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http://dx.doi.org/10.1016/j.compstruct.2017.01.070
0263-8223/

application. NSM systems were proved to be more efficient than EBR systems because the NSM strips are less sensitive to debonding phenomena (see for instance [2,8,9]). Although NSM reinforcement cannot be used in concrete elements for which the cover depth is low, NSM bars are protected by the concrete cover and are less exposed to accidental impacts, mechanical damage, fire, and vandalism and the aesthetic aspect of the strengthened structure is virtually unchanged [10].

Extensive research has been carried out to investigate the NSM-concrete bond behavior (see for instance [11–15]). However, limited work is available regarding analytical models for designing NSM strengthening of RC members.

In this paper, three analytical models for the estimation of the bending capacity of RC beams strengthened with NSM FRP composites (i.e. American ACI 440.2R-08 [2], English TR 55 [16], and Canadian CSA S806-12 guidelines [17]) are analyzed. Furthermore, the procedure for EB reinforcement proposed by the Italian CNR-DT 200 R1/2013 [1] is extended to the case of NSM reinforcement. An experimental database comprised of 155 RC beams strengthened in flexure with NSM reinforcement was collected from the literature. Comparisons between the maximum bending moment computed by the analytical models and the corresponding experimentally measured value allowed to assess the accuracy of each model.

2. NSM flexural strengthening analytical models

The accuracy of the analytical methods proposed by the American ACI 440.2R-08 [2], English TR 55 [16], and Canadian CSA S806-12 [17] guidelines for the evaluation of the flexural capacity of NSM strengthened RC members was assessed in this paper. In addition, the procedure included in the Italian guideline CNR-DT 200 R1/2013 [1] for the evaluation of the flexural capacity of EB FRP strengthened RC members was extended to the case of NSM reinforcement and its accuracy was assessed using the experimental database collected.

All analytical models considered in this paper have a solid fracture mechanics framework. The maximum strain of a near surface mounted FRP bar should be computed by evaluating the fracture energy of the specific interface where fracture is assumed to occur. However, since the evaluation of the fracture energy associated with each possible fracture interface is a difficult task, simplified formulations are adopted by design codes/recommendations.

All procedures considered are based on the assumptions that plane sections remain plane, there is no relative slip between FRP reinforcement and concrete, and FRP reinforcement has a linear-elastic stress-strain behavior up to failure. These assumptions, although questionable, appear to be reasonable for design purposes and their validity can be confirmed by comparison between experimental results and corresponding analytical provisions. Failure of the strengthened member is assumed when one of the following scenarios occurs:

i. failure (rupture or debonding) of the NSM reinforcement, which is assumed to occur when the strain in the NSM reinforcement attains the maximum value ε_{fd};
ii. concrete crushing, which is assumed to occur when the concrete strain is equal to 0.003 in the case of American guideline, and to 0.0035 in the remaining guidelines considered;
iii. simultaneous rupture of NSM reinforcement and crushing of concrete, referred to as *balanced failure* in the Canadian guideline [17].

The analytical formulations provided by each guideline for computing the maximum NSM strain ε_{fd} are here briefly recalled for the sake of clarity. Wherever possible, the same notation employed in the original documents was adopted.

2.1. ACI 440.2R-08 [2]

According to the American guideline ACI 440.2R-08 [2] the maximum strain in the FRP reinforcement is affected by many factors (e.g. the member dimensions, reinforcement ratio, and treatment of the FRP reinforcement surface) and is included within the range $0.6\varepsilon_{fu}$ and $0.9\varepsilon_{fu}$, where ε_{fu} is the design rupture strain of the FRP reinforcement:

$$\varepsilon_{fu} = C_E \cdot \varepsilon^*_{fu} \tag{1}$$

where $\varepsilon^*{}_{fu}$ is the ultimate rupture strain of the FRP reinforcement and C_E is an environmental reduction factor, which was assumed equal to 1.0 in this study. C_E accounts for the possible reduction of the composite properties due to the exposition of the strengthening system to certain environmental conditions for a relatively long time. The environmental reduction factor C_E was assumed equal to 1.0 in this paper because the environmental conditions during laboratory tests are controlled and no reduction of the composite properties due to environmental exposure should occur. The maximum value of the strain in the FRP reinforcement is recommended to be:

$$\varepsilon_{fd} = 0.7\varepsilon_{fu} \tag{2}$$

In order to achieve ε_{fd} the FRP reinforcement should have a bonded length greater than the development length l_{db}, which is defined as the length needed to achieve the design strength [2]:

$$l_{db} = \frac{d_b}{4\tau_b} f_{fd} \quad \text{for circular bars} \tag{3}$$

$$l_{db} = \frac{a_b \cdot b_b}{2(a_b + b_b)\tau_b} f_{fd} \quad \text{for rectangular bars} \tag{4}$$

where d_b is the diameter of the NSM bar, τ_b is the average bond strength, which is assumed equal to 6.9 MPa, f_{fd} is the design strength of the NSM reinforcement, and a_b and b_b are the width and thickness of the NSM plate, respectively.

2.2. TR 55 [16]

According to the English guideline TR 55 [16], NSM strengthening of RC beams should be designed against three different failure modes, namely concrete cover separation, concrete splitting failure, and failure in the adhesive layer. To avoid concrete cover separation, the strain in the FRP should be limited to:

$$\varepsilon_{lim} = 38\sqrt{\frac{b}{n} \cdot \frac{f_{ctk}}{E_f A_f}} \leqslant \varepsilon_{fu} \tag{5}$$

where b is the width of the concrete cross-section, n is the number of NSM bars employed, f_{ctk} is the concrete characteristic tensile strength, E_f is the FRP modulus of elasticity, and A_f is the area of the single NSM bar.

To avoid concrete splitting failure the strain in the FRP reinforcement in the cross-section where the FRP is needed, i.e. where yielding of the steel reinforcement starts to occur, should be limited to the maximum ultimate anchorage strain ε_{max}:

$$\varepsilon_{max} = \begin{cases} 10 \cdot b_{notchperim}\sqrt{\frac{f_{ctk}}{E_f A_f}} & \text{for } l \geqslant l_{db} \\ 10 \cdot b_{notchperim}\sqrt{\frac{f_{ctk}}{E_f A_f}} \cdot \frac{l}{l_{db}}\left(2 - \frac{l}{l_{db}}\right) & \text{for } l < l_{db} \end{cases} \tag{6}$$

where $b_{notchperim}$ is the effective perimeter of notch, which is generally assumed as the minimum depth plus the minimum width if no special methods were employed to prepare the notch sides, l is the