PROPOSED DESIGN PROCEDURE OF NSM FRP REINFORCEMENT FOR FLEXURAL AND SHEAR STRENGTHENING OF RC BEAMS

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Near-surface mounted (NSM) fiber-reinforced polymer (FRP) reinforcement has been found to perform efficiently as a flexural/shear strengthening technology for reinforced concrete (RC) members, while offering some additional practical advantages with respect to the most widespread technique of externally bonded laminates. This paper presents a design procedure for flexural and shear strengthening or RC beams with NSM FRP reinforcement, as well as for detailing aspects such as anchorage length. The design equations account for debonding mechanisms critical for NSM reinforcement. Experimental results and analytical studies currently available on this topic are used as basis and support to the proposed equations, and areas of lacking knowledge are pointed out.

INTRODUCTION

Fiber-reinforced polymer (FRP)-based techniques for repair/rehabilitation and strengthening of structures have been rapidly gaining popularity worldwide. Beside the most widespread technology of externally bonded wet-lay up or pre-cured FRP laminates, near-surface mounted (NSM) FRP reinforcement has been found to perform efficiently as a flexural/shear strengthening technology for reinforced concrete (RC) and prestressed concrete (PC) members, while offering some additional practical advantages as pointed out elsewhere[1].

The experimental data available thus far on flexural/shear strengthening of RC and PC beams with NSM FRP rods is relatively limited: flexural strengthening has been investigated by different authors[2-6]; shear strengthening has been addressed by De Lorenzis and Nanni[1]. As indicated by the experimental evidence, failure of the strengthened beams may occur by the mechanisms accounted for by the conventional RC theory, as well as by debonding of the NSM bar(s). In the context of beams strengthened with steel plates or FRP laminates, debonding phenomena leading to collapse are often indicated as “premature” failure mechanisms, to underline the fact that

they prevent the beam from reaching its ultimate capacity as conventionally predicted. Reliable models to estimate the ultimate load associated to such mechanisms are needed for a safe design of FRP strengthening systems.
This paper presents a design procedure for flexural and shear strengthening of RC beams with NSM FRP reinforcement, as well as for detailing aspects such as anchorage length. The design equations account for "traditional" flexural and shear failure mechanisms, as well as for debonding mechanisms critical for NSM reinforcement. Experimental results and analytical studies currently available on this topic are used as basis and support to the proposed equations, and areas of lacking knowledge are pointed out.

FLEXURAL STRENGTHENING

Background

Two debonding failure modes have been experimentally recorded on beams strengthened in bending with NSM FRP rods. Beams strengthened with sandblasted NSM bars may fail by debonding at the bar-epoxy interface[3], whereas those strengthened with deformed or spirally wound NSM bars are susceptible to failure by concrete cover delamination[3,4,6]. The latter mechanism is common to beams strengthened with externally bonded steel plates or FRP laminates.
A large number of models have been proposed to predict the debonding strength of beams strengthened in bending with steel plates or FRP laminates bonded to their tension face. A comprehensive survey has recently assessed the accuracy and safety of many different models by comparing their predictions with a wide experimental database[7].
A class of models are the so-called "concrete tooth models", based on the concept of a concrete tooth between two adjacent cracks behaving like a cantilever under the horizontal shear stresses acting at the interface of the beam with the reinforcement bonded to the tension face. In particular, the model by Raoof and Hassanen was found to provide reasonably accurate estimates of the debonding strength[7]. Although the model, by its nature, is suited to predict failure by cover delamination, prediction of the ultimate load of beams failed by other types of debonding was also found rather accurate.
As follows, the model is modified for the case of NSM FRP reinforcement and, on the basis on bond test results obtained elsewhere[8], is applied to the beams tested in previous studies[3,6].

Preliminary model for the debonding strength

The minimum stabilized crack spacing, l_{min}, can be computed by:

$$l_{\min} = \frac{A_e f_{ct}}{u_s \Sigma O_s + u_f \Sigma O_f} \tag{1}$$

where A_e is the area of concrete in tension, f_{ct} the concrete tensile strength, u_s the average bond strength between concrete and steel reinforcing bars, ΣO_s the total perimeter of the steel bars, u_f the average bond strength between NSM FRP bars and surrounding material, ΣO_f the total perimeter of the FRP bars. Eq. (1) is an extension of the classical expression of the minimum crack spacing in reinforced concrete, based on the assumption of a uniform distribution of bond stresses at both steel-to-concrete and NSM bar-to-concrete interfaces. For the typical values of minimum crack spacing, the average bond strength u_f can be reasonably approximated with the local bond strength, and the latter should be known from literature data[8] or experimental tests. The maximum stabilized crack spacing is twice the minimum, i.e. $l_{max}=2\ l_{min}$. For the case of an RC beam with a single layer of steel tension reinforcement, A_e is twice the distance from the centroid of the tension reinforcement to the base of the RC beam multiplied by the beam width. Moreover, it is assumed, as in the original model[7], that:

$$u_s = 0.28\sqrt{f_{cu}} \text{ (in MPa)}, \tag{2}$$

and that:

$$f_{ct} = 0.36\sqrt{f_{cu}} \text{ (in MPa)}, \tag{3}$$

f_{cu} being the concrete cube compressive strength.

The model assumes that failure of the concrete tooth between two adjacent cracks occurs when the stress at point A (Figure 1) exceeds the concrete tensile strength. Such stress can be determined as follows:

$$\sigma_A = \frac{M_A}{I_A}\left(\frac{l}{2}\right) \tag{4}$$

where $M_A = \tau n \pi d_b l h'$ and $I_A = bl^3/12$. I_A is the moment of inertia of the tooth, M_A is the moment at the base of the tooth, l is the crack spacing (minimum or maximum), h' is the distance from the base of the steel tension reinforcement to the centroid of the NSM reinforcement, τ is the shear stress at the interface between NSM FRP bars and surrounding material, n is the number of NSM bars, d_b is their diameter, b is the width of the beam. Substituting M_A and I_A in equation (4) and assuming that at the instant of debonding is $\sigma_A = f_{ct}$, the value of τ at which delamination of the concrete cover occurs is as follows:

$$\tau_{del} = \frac{f_{ct} l}{6h'} \frac{b}{n \pi d_b} \tag{5}$$

Within the shear span of the beam (for a beam under four-point bending), the shear stress τ is balanced by the axial stress σ_f in the NSM bars. At the location under the point load at delamination is:

$$\sigma_{fdel} = \frac{4}{\phi} L_p \tau_{del} = \frac{2bL_p l}{3n\pi d_b^2 h'} f_{ct} \tag{6}$$

where L_p is the effective length of the NSM bars in the shear span over which equivalent shear stresses at the interface of the bars with the surrounding material may be assumed to remain constant. The expression of L_p is the means through which empirical calibration has been conducted in the original model for externally bonded steel plates, and in its subsequent modifications[7]. Of the different expressions proposed for L_p, the most recent ones give L_p as the smaller of the plate length in the shear span, L_{p1}, and an equivalent length L_{p2} function of l_{min}. The latter function, calibrated with test data, had a parabolic form for low values of l_{min}, followed by a linear expression. In analogy with the original model and until more experimental data becomes available, it is herein suggested to take L_p for NSM bars as the smaller of the bar length in the shear span, L_{p1}, and an equivalent length L_{p2} given by:

$$L_{p2} = 1.86 l_{\min}^2 - 127 l_{\min} + 2436 \qquad \text{if} \qquad l_{\min} \le 50\,\text{mm} \tag{7a}$$

$$L_{p2} = 736\,\text{mm} \qquad \text{if} \qquad l_{\min} \ge 50\,\text{mm} \tag{7b}$$

Hence, the minimum and the maximum stress in the FRP required to cause flexural cracking and failure of a tooth can be determined from equation (6) with l taken as l_{min} or l_{max}, respectively.

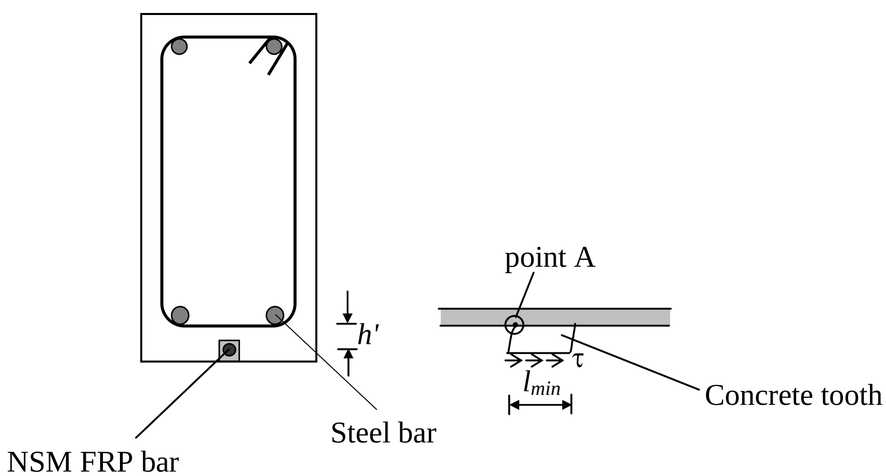

Figure 1. Concrete tooth model

Comparison with experimental results and discussion

Equation (7) was calibrated based on the upper bound of σ_{fdel} using test results of beams BFG4, BR1-a and BR1-b tested in previous studies[3,6] and then verified against results of the remaining beams BFC3, BFC4, BR2-a and BR2-b. The bond strength of the NSM FRP bars introduced in the equations, u_f, was taken as the local bond strength obtained from bond tests and modeling reported elsewhere[8]. Comparison between experimental results and theoretical predictions obtained by the conventional RC theory and the proposed model (upper bound) is summarized in Table 1. It appears that the model is able to accurately predict failure mode and ultimate load of the beams. Further studies are needed to assess the validity of the model on a wider experimental database.

Table 1. Comparison between experimental and theoretical results

Beam	Experimental		Theor. (conventional theory)		% Error	Theor. (proposed model, upper bound)		% Error
	Ultimate load (kN)	Failure mode	Ultimate load (kN)	Failure mode		Ultimate load (kN)	Failure mode	
BFC3	203.6	DBY	223.9	FRY	-9.1	210.0	DBY	-3.0
BFC4	226.0	DBY	290.7	FRY	-22.3	194.0	DBY	16.5
BFG4	196.9	DBY	216.0	CCY	-8.9	198.3	DBY	-0.7
BR1-a	84.7	DBY	95.4	CCY	12.6	84.5	DBY	0.2
BR1-b	125.1	CCY	121.7	CCY	-2.7	121.7	CCY	-2.7
BR2-a	97.3	DBY	113.5	CCY	16.6	97.0	DBY	0.3
BR2-b	135.4	CCY - DB	134.2	CCY	-0.9	134.2	CCY	-0.9

CC = Concrete crushing after steel yielding; DBY = Debonding after steel yielding; FRY: fiber rupture after steel yielding.

The model, in this form, can be applied to beams under four-point bending whereas other loading schemes would require recalibration of the equivalent length L_p. The presence of the epoxy (or other encapsulating material for the NSM bars) was neglected. Predictions of the model are sensitive to the values of l_{min} and L_p, whose expressions are then most critical. While the use of equation (1) is a reasonable simplification for design purposes, it is questionable whether the local bond strength of NSM bars is appropriate or a reduced value (closer to the average bond strength) should be used. However, using a larger bond strength results in a smaller value of l_{min}, hence, in a lower strength of the concrete tooth and in more conservative predictions.

Note that the model applies rather well also to beams BFC3 and BFC4, although they failed by debonding at the bar-epoxy interface rather than by cover delamination. As calibration has been conducted on the upper bound, the comparison with the experimental results is good with the upper bound prediction, while the lower bound would result in a very conservative estimate.

Proposed design procedure

The proposed design equations for computing the moment capacity of an RC cross-section strengthened in bending with NSM FRP rods are summarized as follows.

1. Obtain u_f (local bond strength of NSM bars) from literature data or by running bond tests with short bonded lengths with the same type of bar, concrete strength, groove-filling material and groove depth to bar diameter ratio to be used in the beams;

2. Compute l_{min} from equation (1) and l_{max} as twice l_{min};

3. Compute $\sigma_{fdelmax}$ from equation (6) taking $l=l_{max}$;

4. Compute the nominal ultimate moment with the conventional RC theory using for the NSM bars an effective tensile strength equal to:

$$\sigma_{fueff} = \min\left(\sigma_{fu}, \sigma_{fdel\max}\right) \quad (8)$$

where σ_{fu} is the tensile strength of the FRP bar.

5. Compute the design ultimate moment of the cross-section by multiplying the nominal ultimate moment by a reduction factor ϕ_f. As suggested in ACI440[9] for beams strengthened with externally bonded laminates, an additional reduction factor ψ_f should be used to further reduce the contribution of the FRP to the moment capacity, to account for the novelty of this strengthening technique. Hence, the design moment should be as follows:

$$\phi M_n = \phi_f \left\lfloor M_s + \psi_f M_f \right\rfloor \quad (9)$$

where M_s and M_f are the contributions to the moment capacity given by steel and FRP, respectively. For ψ_f, a value of 0.85 as suggested by ACI440[9] is recommended.

6. Check serviceability. This phase is not further detailed, as it presents no difference with respect to the case of externally bonded laminates.

SHEAR STRENGTHENING

Proposed design procedure

A design approach for computing the shear capacity of RC beams strengthened in shear with NSM FRP rods was proposed by the authors in a previous study[1]. Such approach includes two equations that may be used to obtain the FRP contribution to the shear capacity and suggests taking the lowest of the two results.

The proposed design equations are briefly summarized below. For more details, refer to the original paper.

1. Compute d_{net};

A reduced value is used for the height of the cross-section containing shear reinforcement in the form of NSM rods:

$$d_{net} = d_r - 2 \cdot c \tag{10}$$

where d_r is the height of the shear-strengthened part of the cross-section and c is the concrete cover of the internal longitudinal reinforcement. In the case of vertical NSM rods, d_r coincides with the length of the FRP rods. This reduction approximately accounts for the height of the Mörsch truss being lower than the total height of the beam. It can be reasonably assumed that the axis of the upper strut is situated on the resultant of the compressive stresses and the axis of the lower tie coincides with that of the longitudinal steel.

2. Compute V_{1F};

V_{1F} is the FRP shear strength contribution related to bond-controlled shear failure in the most unfavorable crack position It is computed using the following assumptions:

- inclination angle of the shear cracks constant and equal to 45 degrees;
- even distribution of bond stresses along the FRP rods at ultimate;
- the ultimate bond strength is reached in all the rods intersected by the crack at ultimate.

$$V_{1F} = 2 \cdot \pi \cdot d_b \cdot u_f \cdot L_{tot\,\min} \tag{11}$$

The value of $L_{tot\,\min}$ depends on d_{net}, on the spacing s of the rods and on their inclination. For vertical rods:

$$L_{tot\,\min} = d_{net} - s \qquad \text{if} \qquad \frac{d_{net}}{3} < s < d_{net} \tag{12a}$$

$$L_{tot\,\min} = 2 \cdot d_{net} - 4 \cdot s \qquad \text{if} \qquad \frac{d_{net}}{4} < s < \frac{d_{net}}{3} \tag{13a}$$

For 45-degree inclined rods:

$$L_{tot\,\min} = (2d_{net} - s)\frac{\sqrt{2}}{2} \qquad \text{if} \qquad \frac{2d_{net}}{3} < s < 2d_{net} \tag{12b}$$

$$L_{tot\,\min} = 2\sqrt{2}(d_{net} - s) \qquad \text{if} \qquad \frac{d_{net}}{2} < s < \frac{2d_{net}}{3} \tag{13b}$$

3. Check if calculation of V_{2F} is necessary;

If:

$$d_{net} < 0.002\frac{d_b \cdot E_b}{u_f} \qquad \text{or} \qquad d_{net} < \sqrt{2}\cdot 0.001\frac{d_b \cdot E_b}{u_f} \qquad (14a,b)$$

for vertical and 45-degree rods, respectively, calculation of V_{2F} is not necessary. If (14) is not satisfied, V_{2F} has to be computed. In equation (14), E_b is the elastic modulus of the NSM FRP bars.

4. Compute V_{2F} (if necessary);

V_{2F} is the FRP shear strength contribution corresponding to a maximum FRP strain of 4000 με. This limit is suggested to maintain the shear integrity of the concrete. V_{2F} has to be computed in the most unfavorable crack position, that it the position in which it is minimum. It can be shown that, for vertical rods, the minimum value is:

$$V_{2F} = 2\cdot\pi\cdot d_b \cdot u_f \cdot \overline{L_i} \qquad \text{if} \qquad \frac{d_{net}}{2} < s < d_{net} \qquad (15a)$$

$$V_{2F} = 2\cdot\pi\cdot d_b \cdot u_f \cdot \overline{L_i}\cdot\frac{3\cdot d_{net} - 4\cdot s}{d_{net}} \qquad \text{if} \qquad \frac{d_{net}}{4} < s < \frac{d_{net}}{2} \qquad (15b)$$

In the case of 45-degree rods, it is:

$$V_{2F} = 2\cdot\pi\cdot d_b \cdot u_f \cdot \overline{L_i} \qquad \text{if} \qquad d_{net} < s < 2d_{net} \qquad (16a)$$

$$V_{2F} = 2\cdot\pi\cdot d_b \cdot u_f \cdot \overline{L_i}\cdot\frac{3\cdot d_{net} - 2\cdot s}{d_{net}} \qquad \text{if} \qquad \frac{d_{net}}{2} < s < d_{net} \qquad (16b)$$

In the previous equations, $\overline{L_i}$ is the effective length of an FRP rod crossed by the crack corresponding to a strain of 4000 με, and is given by:

$$\overline{L_i} = 0.001\frac{d_b \cdot E_b}{u_f} \qquad (17)$$

5. Compute $V_{FRP} = \min(V_{1F}, V_{2F})$;

6. Check that limits on V_{FRP} are satisfied;

Limits on the value of V_{FRP} and of the sum ($V_s + V_{FRP}$) indicated by ACI440[9] should be extended to the case of NSM strengthening, as their rationale is of general validity.

7. Compute the shear capacity of the beam;

The nominal shear strength of an RC beam strengthened with an FRP system can be computed as the sum of the shear strength of the concrete, the shear strength provided by the steel shear reinforcement, and the contribution of the FRP reinforcement[9]:

$$V_n = V_c + V_s + V_{FRP} \qquad (18)$$

The design shear strength is obtained by applying a strength reduction factor, ϕ_s, to the nominal shear strength. As indicated by ACI440[9], the reduction factor ϕ_s = 0.85 given in ACI 318[10] should be maintained for the concrete and steel terms, and an additional reduction factor ψ_f should be applied to the FRP contribution, to account for the novelty of this strengthening technique:

$$\phi V_n = \phi_s \left[V_c + V_s + \psi_s V_{FRP} \right] \qquad (19)$$

Factor ψ_f should not exceed 0.85.

ANCHORAGE LENGTH

Background

Extensive experimental and analytical investigations on bond of NSM FRP reinforcement in concrete has been conducted in previous studies[8]. Results made available the calibrated local bond-slip relationship and, consequently, the entire curve of the bond failure load as a function of the embedment length for different types of FRP bars. Each curve is valid for a given concrete to groove-filling material tensile strength ratio, and for a given groove depth to bar diameter ratio. However, the trend of change of the local bond strength with these variables has been enucleated elsewhere, and the experimental data available, being referred to low concrete strength, should be safely applicable to practical cases.

Proposed design procedure

Based on the limit state philosophy, a design approach for the anchorage length of NSM FRP bars in concrete is suggested as follows:

1. Check that $P_s \leq P_1$. At service load level, it should be required that the free-end slip is zero and that the bar is anchored using only the ascending portion of the bond-slip relationship. This poses a limit to the service load that can be applied to the bar, P_s, which must be less than or equal to P_1. The value of P_1, being function of the calibrated local bond-slip relationship, is also available from previous studies.

2. Find the anchorage length at the ULS, $L_{an,u}$, *i*.e. the embedment length needed to anchor the bar under the design load at the ULS, P_u. In order to account for uncertainties in the bond behavior, the curves of the bond failure load as a function of the embedment length should be scaled homotetically by an appropriate reduction factor. Entering the reduced curve with the factored load, P_u, the corresponding anchorage length at the ULS can be found.

3. Find the anchorage length at service, $L_{an,s}$.

For $P_s \leq P_1$, the embedment length needed is[11]:

$$L_{an,s} = l_m \left(\frac{P_s}{P_1} \right)^{\frac{1-\alpha}{1+\alpha}} \qquad (20)$$

An additional check to be performed at service load level is that the loaded-end slip is not larger than a limiting value compatible with aesthetic and/or durability requirements. This limit has been proposed as 0.4 mm by previous researchers[11]. As the value of s_m found for NSM reinforcement is always less than 0.4 mm, this condition is authomatically satisfied as soon as the service load is less than or equal to P_1.
4. Compute the anchorage length. Finally, the anchorage length of the bar should be the maximum of $L_{an,u}$ and $L_{an,s}$.

CONCLUSIONS

This paper proposed a design procedure for flexural and shear strengthening of RC beams with NSM FRP reinforcement, as well as for the anchorage length. The design equations account for "traditional" failure mechanisms, as well as for debonding mechanisms critical for NSM reinforcement. Predictions of the models which are the basis of the proposed design procedures have been compared with experimental results either in this (flexural strengthening) or in previous studies (shear strengthening[1] and anchorage length[8]), showing a reasonably good correlation. Further research is needed to assess the validity of the proposed procedures on a wider experimental database and to clarify other behavioural aspects of beams strengthened with NSM FRP reinforcement, such as those related to serviceability and to long-term performance.

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