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# Strengthening of Existing Concrete Structures with Fibre Reinforced Polymers (FRP) in the New Version of Eurocode 2

## *Refuerzo de estructuras existentes de hormigón con polímeros reforzados con fibras (PRF) en la nueva versión del Eurocódigo 2*

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**ABSTRACT**

This paper aims to introduce the content of Annex J "Strengthening of Existing Concrete Structures with CFRP" of Eurocode 2 [1]. This is first time that the design of adhesively bonded reinforcement with CFRP has been introduced in the European regulations through an informative annex. Annex J considers two different bonded strengthening techniques: externally bonded reinforcement (EBR) that consists of bonding CFRP strips or sheets to the surface of concrete elements, and near surface mounted reinforcement (NSM) that consists of embedded CFRP strips or rods to the slot cut in the concrete cover. Since, the content of Annex J is new, a summary and background related to all aspects required for designing CFRP strengthened systems for concrete structures, are given in this paper..

KEYWORDS: Strengthening, fibre reinforced polymer, externally bonded reinforcement, near surface bonded reinforcement, bond, reinforced concrete, prestressed concrete.



**RESUMEN**

Este artículo tiene por objeto introducir el contenido del Anejo J del Eurocódigo 2 [1]: "Refuerzo de estructuras de hormigón existentes con CFRP". Esta es la primera vez que el dimensionamiento de refuerzos adheridos con CFRP se introduce en la normativa europea a través de un anejo informativo. El Anejo J contempla dos técnicas de refuerzo adherido diferentes: el refuerzo adherido externamente (EBR) que consiste en pegar laminados o tejidos de CFRP a la superficie de los elementos de hormigón a reforzar, y el refuerzo insertado en el recubrimiento (NSM) que consiste en instalar el laminado o barra de CFRP en una ranura realizada en el recubrimiento del hormigón. Como el Anejo J en sí es una novedad, este artículo presenta un resumen de su contenido y algunos antecedentes relacionados con todos los aspectos necesarios para diseñar un sistema de refuerzo con CFRP para estructuras de hormigón.

PALABRAS CLAVE: Refuerzo, polímeros reforzados con fibras, refuerzo adherido externamente, refuerzo embebido en ranuras, adherencia, hormigón armado, hormigón pretensado.



## 1. INTRODUCTION

Strengthening of existing reinforced and prestressed concrete structures might be necessary to restore or increase their load-bearing capacity due to different reasons: an increase of load demand caused by a change of use, a loss of carrying capacity due to deterioration or structural damage, or to eliminate structural design or construction deficiencies.

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a) b) c)

Figure 1. a) Flexural strengthening, b) shear strengthening, c) column confinement (courtesy of Mapei).

a) b)

Figure 2. a) Externally bonded reinforcement (courtesy of Mapei), b) Near surface mounted reinforcement [16].

During the 1990's, fibre reinforced polymer (FRP) laminates, which were related to other industries such aeronautics or sports, were introduced in the construction field to overcome the drawbacks of steel plates bonded to the tensile concrete surface, which were corrosion and weight among others.

FRP is the denomination of a composite material formed by a polymeric matrix reinforced with continuous glass (GFRP), basalt (BFRP), carbon (CFRP) or aramid (AFRP) fibres. Recently, other composites formed by natural fibres or PBO (polyphenylene bezobisoxazole) with a cementitious matrix [2,3] have been introduced in the strengthening field but with limited research applications. The first application of FRP strengthening in Europe was in 1991, in the Ibach bridge (Switzerland), a historic wooden bridge that was strengthened by externally bonded CFRP laminates [4]. Since then, the strengthening technique consisting on adhesively bonding a reinforcement (ABR) of FRP to an existing structure constitutes a well established technology. FRPs have been effectively applied as flexural strengthening, shear strengthening and confinement of columns (see Figure 1). There are many reasons of their increasing use, especially in aggressive environments, since they show a high strength-to-weight and stiffness-to-weight ratio, a potential high durability given by their resistance to corrosion, no need of scaffolding, reduction in labour costs, and versatility with practically unlimited availability of dimensions. However, FRP strengthened systems present also some drawbacks such as their reduced ductility due to their linear elastic behaviour up to failure, possible degradation under high temperatures depending on the glass transition temperature of the resin, and the cost of the material itself.

The future version of Eurocode 2 [1] will include an informative annex (Annex J) with the rules for strengthening existing plain, reinforced and prestressed normal weight concrete structures only with CFRP materials. This annex covers only CFRP materials because it is the most common fibre type in research and real applications. In addition, there is not enough experience in strengthening of special concrete structures, such as lightweight concrete or concrete with recycled aggregates.

Adhesively bonded reinforcement (ABR) gives wider applications for different methods and products. Annex J of Eurocode 2 [1] considers two possible applications: externally bonded reinforcement (EBR) [5–8] and near surface mounted reinforcement (NSM) [9–13]. EBR consists of strips or sheets bonded on the surface of a concrete support (see Figure 2a) and NSM consists of strips or bars applied in slot cuts in the concrete cover (see Figure 2b). Compared to EBR, the NSM technique provides better bond characteristics, the reinforcement can be anchored more easily to prevent debonding and it is more protected against mechanical damage or vandalism. Other strengthening techniques such as Textile Reinforced Mortar (TRM) [14], embedded through the section reinforcement [15] or other bonded configurations are not included in Annex J because there is not a consolidate experience with all of them. In addition, prestressed ABR is not considered for the same reason.

This paper aims to introduce the content of Annex J "Strengthening of Existing Concrete Structures with CFRP" of Eurocode 2 [1]. This is the first time that the design of CFRP strengthening systems has been introduced in European regulations. Model Code 2010 [17] included FRP reinforcement in two sections 5.5 "Non-metallic reinforcement" and 6.2 "Bond of non-metallic reinforcement", explaining the main principles of this technique. There are also some European guidelines such as DAfStb Heft 595 [18], TR-55 [19], CNR-DT-200 R1 2013 [20], AFGC [21], SIA [22] and GRECO [23]. The fib Bulletin 14 [24] was published in 2001 and gave detailed guidelines on the use of FRP externally bonded reinforcement, practical execution and quality control, based on the expertise of the members of fib TG9.3 "FRP Reinforcement for concrete structures". The advance of the state-of-the-art of the last two decades was updated in fib Bulletin 90 [16] by fib TG5.1 (former TG9.3) in 2019. This document aimed to cover both externally bonded and near-surface mounted reinforcement for concrete structures. It was presented in a Eurocode-compatible format, with the objective of being also a background document for Annex J and to form the basis for the updating of the text on seismic retrofitting with composites in the next version of Eurocode 8. In addition, there is a background document of Annex J [25] with more details about the derivation of the formulations included in this paper.

## 2.
## BASIS OF DESIGN, MATERIALS AND DURABILITY

In general, the basis of design of concrete structures with conventional materials can be applied to reinforced (RC) and prestressed concrete (PC) strengthened with adhesively bonded CFRP reinforcement. However, there are some aspects such as the material safety factors that should be particularized for this case. Unless a National Annex gives different values, Table 1 complies the partial safety factors for ABR. As observed, safety factors are higher for in-situ wet lay-up sheets than for prefabricated strips and bars. These safety factors were obtained based on the regulations of prEN 1990:2020 [26] and the products considered in the determination of these factors were those used in the structural tests used in the calibration of approaches included in the subsections of Annex J related to Ultimate Limit States, Serviceability Limit States, Fatigue and Bond. The safety concept for bond is based on design assisted by testing. The safety factor for bond, $\gamma_{BA}$, was taken from [16], assuming failure in the concrete substrate or failure of the adhesive. This factor is the one specified in the main text of Eurocode 2 [1] for the design value of the ultimate bond stress.

**TABLE 1.**
**Partial safety factors for ABR strengthening [1].**

| Design situation | Tensile strength | | Bond strength |
|---|---|---|---|
| | CFRP strips and bars | In-situ lay-up CF sheets | Failure in concrete or adhesive |
| Designation | $\gamma_f$ | | $\gamma_{BA}$ |
| Persistent and transient | 1.30 | 1.40 | 1.50 |
| Accidental | 1.10 | 1.15 | 1.15 |
| Serviceability | 1.00 | 1.00 | 1.00 |
| Fatigue | 1.30 | 1.40 | 1.50 |

In relation to the materials employed for strengthening, there is a different particularity in comparison with other construction materials. The system is made by the combination of fibres and a matrix, designed to work together, and with a specific binder applied to the surface of the support [16]. Therefore, only systems that have been tested and applied to real scale structures can be applied as ABR. In addition, the selection of the system type depends on the configuration and on the structure to be strengthened. A general description on FRP materials, systems and techniques can be found in fib Bulletin 90 [16]. The material properties should be given by the suppliers. The FRP strengthening systems shall comply with national or international product standards, such as ISO 10406 [27] that specify their geometrical, mechanical and technological properties. Annex J gives recommended values of some parameters where test procedures are not standardized yet.

Test procedures for the essential characteristics of construction products at European level, may be included in two types of technical specifications: European harmonised product standards or European Assessment Documents (EAD). EADs are currently under preparation for CFRP strengthening systems.

The design rules included in Annex J are for CFRP systems that accomplish the following conditions:

- interlaminar shear strength of CFRP strips according to EN ISO 14130 [28] shall be equal or larger than the adhesive bond strength for any system,
- mean modulus of elasticity of CFRP strips: 150 000 MPa $\leq E_f \leq$ 250 000 MPa,
- elastic stiffness per unit width of carbon fibre (CF) sheets: 20 kN/mm $\leq E_f \ A_f/b_f \leq$ 400 kN/mm,
- total CF cross section per unit width of CF sheets in the total of all layers determined in the direction of the tension action effect applied to the system: 100 mm²/m $\leq A_f/b_f \leq$ 1800 mm²/m.
- characteristic tensile strength of the adhesive $f_{Atk}$, determined in accordance with EN 1504-4 [29] shall be $f_{Atk} \geq$ 14 N/mm².

Annex J requires the definition of the following properties for CFRP strips and sheets that are going to be used as ABR strengthening systems: $f_{fuk}$, characteristic short-term tensile strength of the ABR according to ISO 10406 [27]; $\eta_f$, reduction factor applied to the tensile strength; $E_f$, average mean modulus of elasticity of the ABR in the longitudinal direction; $\varepsilon_{fuk}$, characteristic ultimate strain; and $A_f$, cross sectional area. For strips $A_f$ is taken as $b_f \cdot t_f$ (where $b_f$ is the width and $t_f$ is the thickness of the cross section). For sheets, $A_f$ is obtained from relevant production data, considering $t_f = n_f^{\,k_f} A_f/b_f$ being $n_f$ the number of layers, $A_f/b_f$ the cross sectional area of the fibres per meter of a single layer of CF sheet, and $k_f = 0.85$ if the number of layers is higher than 3, or 1.00 otherwise. For the adhesive, Annex J requires the characteristic compressive strength, $f_{Ack}$, and the characteristic tensile strength, $f_{Atk}$, determined in accordance with EN 1504-4 [29].

FRP materials are linear elastic up to failure, as shown in the design stress-strain relationship in Figure 3.

The design tensile strength of the ABR system shall be obtained as:

$$f_{fud} = \frac{\eta_f \ f_{fuk}}{\gamma_f} \tag{1}$$

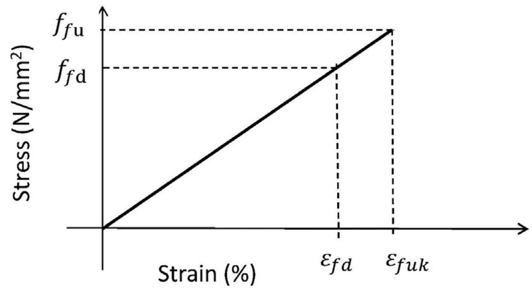



Figure 3. Design stress-strain relationship for the CFRP strengthening system [16].

where: $f_{fuk}$ is the characteristic tensile strength, $\gamma_f$ is the partial safety factor, and $\eta_f$ is a reduction factor applied to the tensile strength of the ABR for the relevant exposure conditions in accordance with ISO 10406 [27], and may be taken as 0.7 unless more accurate information is available.

In a similar manner to conventional RC and PC structures, durability of the strengthened structure, and in particular of the CFRP system and adhesive should be ensured during lifetime according to the exposure classes. The FRP-concrete interface is the critical component of the system since the transfer of stresses occurs through it, and bond quality is affected by the environmental conditions. Therefore, additional protective measures should be included to ensure durability if necessary.

Special attention should be paid to the exposure of the strengthened element to direct UV radiation, penetration of moisture and temperature.

## 3. STRUCTURAL ANALYSIS

According to Annex J, members strengthened with ABR should not be analysed using linear elastic analysis with limited redistribution or plastic analysis, since the CFRP systems are linear elastic up to failure.

The glass transition temperature usually ranges from 50 to 80°C for epoxy and for processed FRP elements ranges from 130 to 140°C. This means that in the event of fire, protection systems may be required in such a way that service temperature is limited with respect to the glass transition temperature. During fire, the CFRP strengthening system will be lost due to the weakening of the adhesive. If this is the case, the existing structure should bear this accidental design situation without collapse, complying with the robustness requirement. This is similar to other accidental situations such as vandalism, blast or impact, where the design engineer should verify the structure against accidental loss of FRP.

## 4. ULTIMATE LIMIT STATES

### *4.1. Bending with and without axial forces*

When strengthening in flexure RC or PC sections, the design of the required ABR area can be obtained by applying sectional equilibrium and compatibility conditions, in a similar manner than conventional concrete elements but with an additional reinforcement, assuming that the slip between the CFRP reinforcement and the concrete substrate is neglected, that is, full composite action between the CFRP and the substrate. The strain state of the unstrengthened element before strengthening should be considered since the strains from additional bending effects after strengthened should be superimposed to the existing ones when verifying the capacity of the strengthened element. Fib bulletin 90 [16] recommends the process given by the flow chart of Figure 4.

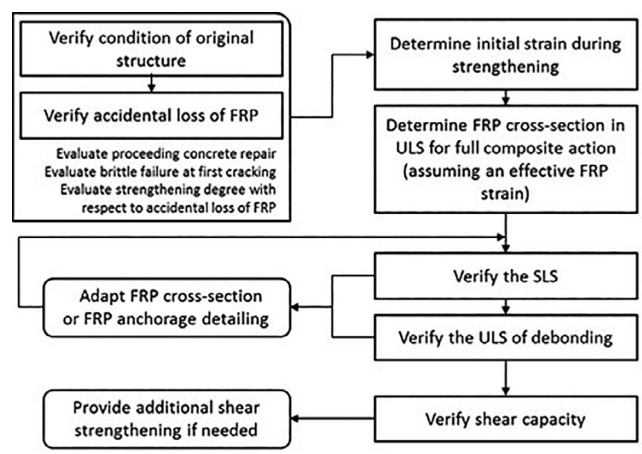



Figure 4. Design process when strengthening a section in flexure recommended by fib Bulletin 90 [16].

When designing the CFRP strengthening system, it is desirable that the strengthened element fails in a ductile manner after steel yielding. So, the governing modes of failure of a flexural element will be steel yielding followed by concrete crushing or steel yielding followed by FRP rupture. As observed in many experimental programs, debonding of the CFRP strengthening system might occur before reaching a classical failure mode (see Figure 5). Debonding is more common in externally bonded reinforcement and might initiate at any location different from the critical section considered to design the flexural strengthening system. To avoid this type of premature failure, debonding should be checked following the procedure described in Section 7 of this paper.

Usually, the reason for strengthening is motivated by strength increase to comply with ULS requirements. However, sometimes the serviceability limit state governs the design, and larger amounts of CFRP than those required for ULS should be applied.

Provisions of Annex J for flexural strengthening with and without axial forces for both EBR and NSM are in accordance with fib Bulletin 90 [16].

Figure 5. Debonding of the CFRP strengthening system [30].

### *4.2. Confinement*

It is well known that confinement can enhance the load bearing capacity of axial loaded members.

Concrete confined by FRP behaves differently from concrete confined by steel. Due to the linear elastic behaviour up to failure, FRP apply an ever-increasing confinement pressure to the concrete core. The stress–strain behaviour of FRP-confined concrete typically displays an approximately bilinear ascending response, and ultimate capacity is governed by tensile failure of the FRP (Figure 6). The ultimate strength of the confined concrete is closely related with the rupture strain of the FRP reinforcement. Many experimental studies have shown that the rupture strain values of FRP jackets are consistently lower than the ultimate tensile strain obtained by standard tensile testing of FRP coupons [31,32]. The ratio between the two values is called strain efficiency factor. There are a number of possible reasons for this premature failure of the FRP jacket, such as the multiaxial stress state, stress concentrations due to concrete failure, or the jacket curvature, especially at corners with low radius.

Figure 6. Failure mode of FRP confined concrete.

Current international design guidelines provide predictive design equations to calculate the ultimate strength and strain of FRP confined concrete columns subjected to pure axial load, as a function of the confining pressure applied by the FRP jacket. It is known that the confinement of non-circular columns is less efficient than the confinement of circular columns [33–35] (see Figure 7a). In a circular cross section, the jacket exerts a uniform confining pressure over the entire perimeter. In the case of a rectangular cross section, the confining action is mostly concentrated at the corners. The predictive equations found in the design guides are mostly based on approaches deduced for circular columns and then modified by a shape factor, usually defined as the ratio of the effectively confined area to the gross area.

Annex J gives provisions to consider the effect of concrete confinement achieved by bonding hoop CFRP around existing columns (see Figure 7b and 7c). Since the formulations have an empirical basis, Annex J limits them to columns with a diameter greater than 150 mm and with characteristic concrete strength less than 50 MPa. Experimental studies outside this range are scarce and show that the effect of confinement is very limited in high strength concrete.

In addition, the first-order eccentricity of the axial load must meet the condition $e_0 / D_{eq} \leq 0.20$ and the slenderness satisfy the condition $l_0 / D_{eq} \leq 40$.

For the application of the equations given in Annex J to rectangular sections the rounding radius of the corners must be $r_c \geq 20$ mm and the aspect ratio $h / b \leq 2$.

According to Annex J, the increase in compressive strength of concrete due to CFRP confinement can be calculated as follows:

**For circular columns:**

$$\Delta f_{cd} = 0 \qquad \text{for} \quad \frac{t_f f_{fud}}{D\, f_{cd}} < 0.07 \tag{2}$$

$$\Delta f_{cd} = k_{cc} \frac{t_f}{D} f_{fud} \qquad \text{for} \quad \frac{t_f f_{fud}}{D\, f_{cd}} \geq 0.07 \tag{3}$$

with $k_{cc} = 2.5$ unless more accurate information is available.

In the case of discontinuous and/or helical wrapping the value of $f_{fud}$ in equations (2) and (3) should be multiplied by the efficiency factor $k_h$ (see Figure 7b and 7c):

$$k_h = \left(1 - \frac{(s_f - b_f)}{2\,D}\right)^2 \left(\frac{1}{1+(tan\beta_f)^2}\right) \tag{4}$$

**For rectangular columns:**

$$\Delta f_{cd} = 0 \qquad \text{for} \left(\frac{b}{h}\right)^2 k_e \frac{t_f\, k_r\, f_{fud}}{D_{eq}\, f_{cd}} < 0.07 \tag{5}$$

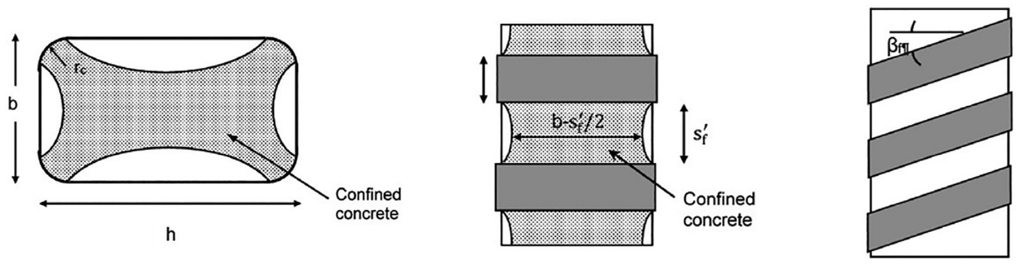
$$\Delta f_{cd} = k_{cc} \left(\frac{b}{h}\right)^2 k_e \frac{t_f}{D_{eq}} k_r\, f_{fud} \quad \text{for} \left(\frac{b}{h}\right)^2 k_e \frac{t_f\, k_r\, f_{fud}}{D\, f_{cd}} \geq 0.07 \tag{6}$$

where:

$k_{cc} = 1.5$ unless more accurate information is available.

$$D_{eq} = \frac{2\,b\,h}{b+h} \tag{7}$$

$$k_e = 1 - \frac{(b-2\,r_c)^2+(h-2\,r_c)^2}{3\,b\,h} \tag{8}$$




a) b) c)

Figure 7. a) Effectively confined section in a rectangular section; b) confinement with discrete strips and c) with helically bonded strips, adapted from [16].

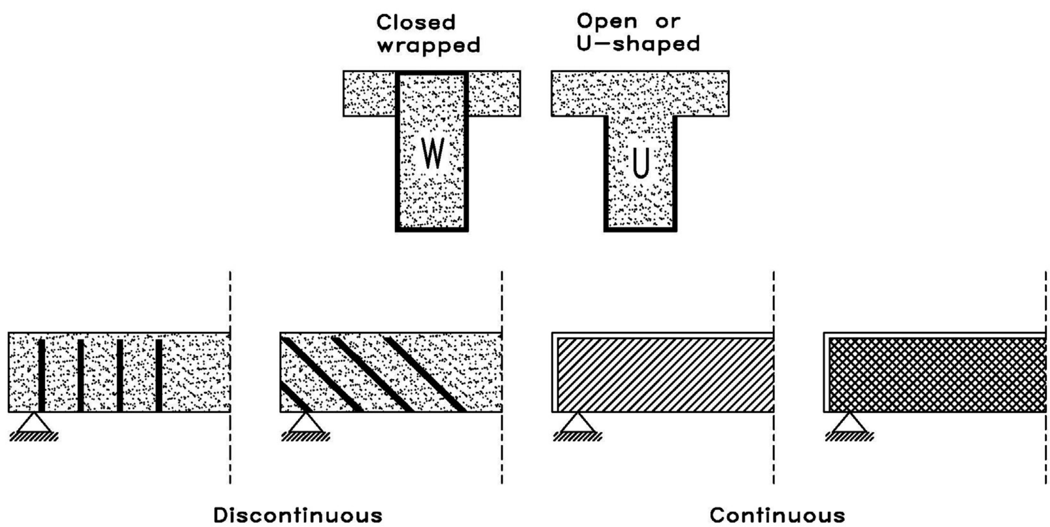



Figure 8. Shear strengthening configurations.

$$k_r = \begin{cases} 1.0\left(\frac{r_c}{50}\right)\left(2-\frac{r_c}{50}\right) & \textit{for } r_c \leq 50\ mm \\ 1.0 & \textit{for } r_c \geq 50\ mm \end{cases} \tag{9}$$

$k_e$ is the ratio between effectively confined area and gross area (see Figure 7a).

$k_r$ is a reduction factor that takes into account that, for rectangular sections, the smaller the corner radius the lower the rupture strain of the FRP jacket.

For discontinuous and/or helical wrapping on rectangular columns the value of $f_{fud}$ in equations (5) and (6) should be multiplied by $k_h$ according to (10):

$$k_h = \left(1-\frac{(s_f-b_f)}{2\,b}\right)\left(1-\frac{(s_f-b_f)}{2\,h}\right)\left(\frac{1}{1+(tan\beta_f)^2}\right) \tag{10}$$

The above equations largely align with the provisions of *fib* bulletin 90 [16]. Equations (3) and (6) implicitly include a strain efficiency factor equal to 0.5, in accordance with [16], for circular and square or rectangular sections with a corner radius $r_c \geq 50$ mm.

However, while in [16] the factor $k_{cc}$ is 3.3, according to the original model of Lam and Teng [32], Annex J indicates that the value of $k_{cc}$ can be taken as 2.5 for circular columns and 1.5 for square and rectangular columns unless more accurate information is available.

### *4.3. Shear*

In a RC or PC element, the shear strength should be checked by following the general provisions of §8.2 of new Eurocode 2 [1], without considering the flexural CFRP ABR (if this is the case) in the contribution of the longitudinal reinforcement. If the design shear stress is higher than the shear strength, then shear strengthening is required, and the section can be strengthened by EBR or NSM techniques.

Externally bonded CFRP shear strengthening can be performed in two different configurations (see Figure 8): a) sheets fully wrapping the section (closed wrapped); b) sheets or L-shaped strips bonded on the lateral sides and the bottom surface of the beam (open or U-shaped systems). The side-bonded configuration, which consists of bonding sheets or strips in the lateral faces of the section is not allowed since they are prompt to debond at both sides of the critical shear crack once it opens and widens. The sheets and strips can be bonded in a continuous or discontinuous configuration.

Closed wrapped CFRP configurations fail due to fibre rupture, sometimes initiated near the corner of the sections that have been rounded to avoid sharp zones that may lead to fibre rupture. Open or U-shaped configurations are susceptible of debonding once a critical shear crack opens and propagates. Then, if the bonded length of each strip at the upper side of the crack (for the U-shaped) is not long enough to anchor the tensile force of the FRP, the laminate debonds suddenly

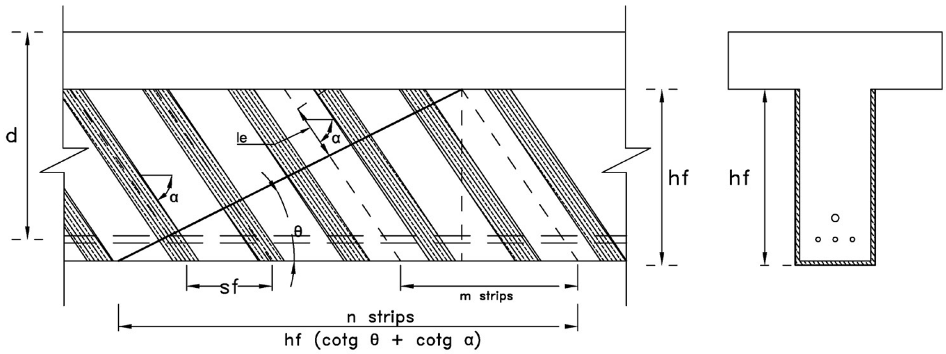



Figure 9. n and m parameters in a CFRP shear-strengthened beam [1].

before reaching its ultimate tensile strength. This debonding failure mode should be considered in the calculation of the shear capacity of the strengthened element and can be delayed or can be avoided by using appropriate anchorage devices. Annex J recommends the application of anchorage devices when strengthening T-shaped cross sections.

The total shear strength of a section strengthened with CFRP may be taken as:

$$\tau_{Rd,CFRP} = \tau_{Rd} + \tau_{Rd,f} \leq 0.5\ v\ f_{cd} \tag{11}$$

where:

$\tau_{Rd}$ is the design shear strength according to Section 8.2 of Eurocode 2 [1].

$$\tau_{Rd,f} = \frac{A_f}{s_f}\frac{f_{fwd}}{b_w}(cot\theta + cot\alpha_f)\ sin\alpha_f \tag{12}$$

$$\frac{A_f}{s_f} = \begin{cases} \frac{2t_f\, b_f}{s_f}\ \textit{for discrete CFRP strips or CF sheets} \\ 2t_f\ \ sin\alpha_f\ \textit{for continuous CF sheets} \end{cases} \tag{13}$$

$\alpha_f$ is the angle formed between the CFRP system and the longitudinal member axis;
$f_{fwd}$ is the design shear strength of the CFRP system.
$\theta$ should be taken as 45 degrees for the calculation of $\tau_{Rd}$ and $\tau_{Rd,f}$, unless more rigorous analysis is undertaken.

Formulations to assess the contribution of the CFRP strip or sheet to the total shear strength consider the different type of configurations. For closed CFRP systems, the design shear strength is defined as Equation (14).

$$f_{fwd} = 0.8\ k_r\ f_{fud} \tag{14}$$

Where:

$f_{fud}$ should be determined using Equation (1) and $k_r$ should be determined using Equation (9).

For open discrete CFRP systems, debonding should be taken in consideration and the shear strength can be obtained through Equation (15) and (16), depending on the length of the strip above the critical shear crack and the maximum bond length, $l_{bf,max,k}$. In any case, the shear strength $f_{fwd}$ is limited by Equation (14).

$f_{fwd}$ is determined by Equation (15) if the anchorage length into the compression zone of the member of all CFRP strips, $l_{bf}$ is less than $l_{bf,max,k}$.

$$f_{fwd} = \frac{2}{3}\ \frac{n\ s_f}{l_{bf,max,k}\,[(cot\theta + cot\alpha_f)\ sin\alpha_f]}\ f_{bfRd} \tag{15}$$

$f_{fwd}$ is determined by Equation (16) if the anchorage length into the compression zone of the member of some CFRP strips, $l_{bf}$ is less than $l_{bf,max,k}$.

$$f_{fwd} = \left[1 - \left(1 - \frac{2}{3}\ \frac{n\ s_f}{l_{bf,max,k}\,[(cot\theta + cot\alpha_f)\ sin\alpha_f]}\right)\frac{m}{n}\right] f_{bfRd} \tag{16}$$

where the parameters $m$ and $n$ are defined in Equation (17) and (18), and in Figure 9. The maximum bond length $l_{bf,max,k}$ and the anchorage resistance $f_{bfRd}$ shall be determined according to §8 of this paper.

$$n = \ integer\left(\frac{h_f(cot\theta + cot\alpha_f)}{s_f}\right) \tag{17}$$

$$m = integer\left(\frac{l_{bf,max,k}\,(cot\theta + cot\alpha_f)\ sin\alpha_f}{s_f}\right) \tag{18}$$

The open continuous sheet system can be treated as a particular case of the discontinuous case with $s_f = b_f / sin\alpha$. Then, $n\ s_f = h_f\ (\cot\theta + \cot\alpha_f)$, $m\ s\alpha_f = l_{bf,max,k}\ (cot\theta + cot\alpha_f)\ sin\alpha_f$ and $m / n = l_{bf,max,k}\ sin\alpha_f / h_f$ [16].

D'Antino and Triantafillou [36] performed an assessment of five design guidelines (EN 1998-3 [37], ACI 440.2 R-08 [38], DAfStb Heft 595 [18], TR-55 [19], CNR-DT/200-R1. 2013 [20]) and a new proposed model, based on the German guideline [18], which is very similar to the proposal included in Annex J. The assessment was performed with a database of 229 RC shear - strengthened beams that failed in shear. They concluded that all models tend to underestimate the FRP shear strength for the completely wrapped configuration. However, models were more accurate for the U-shaped configuration. The proposal gave conservative results (mean value of the experimental to theoretical ratio MV=1.77 and coefficient of variation COV=2.21 for U-shaped and MV=3.51 and COV=4.32 for wrapped).

In the framework of TG1, Oller and Kotynia presented in [39] an analysis of the performance of different existing

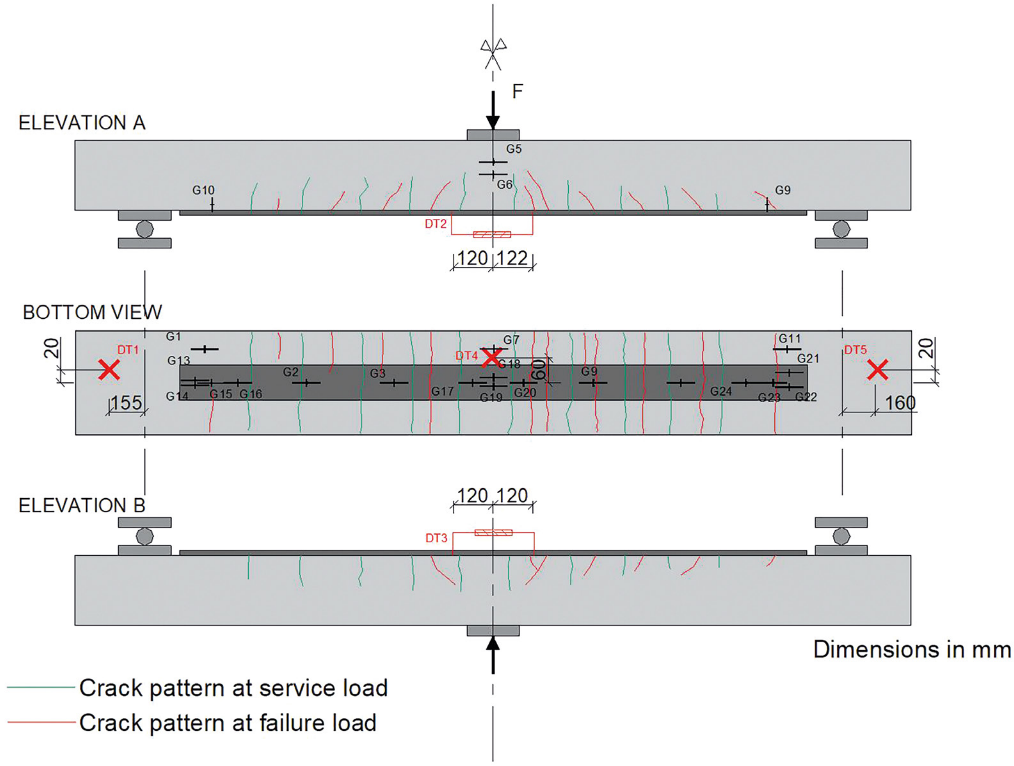



Figure 10. Example of cracking pattern after strengthening [30].

formulations to quantify the FRP contribution to the shear strength of RC elements strengthened in shear by externally bonded FRP sheets [16,18–20,24,40–48]. A large database of 555 tests (355 with rectangular section and 200 with T-section) has been assembled distinguishing between the shape of the section, the existence of internal transverse reinforcement and the FRP configurations. Selected beams with *a/d* higher than 2.5, that were well-documented, and which had a rectangular (276) or a T (180) cross-section, were externally strengthened in a closed (71 R + 68 T), open (114 R + 98 T) or side bonded (91 R + 14 T) configuration with FRP wet lay-up or pultruded strips in a continuous or discontinuous manner, and with or without internal transverse steel reinforcement. In general, predictions for all models were more conservative for beams without transverse reinforcement. In some cases, predictions were unsafe for beams with transverse reinforcement, showing a possible interaction with the internal transverse reinforcement which is not considered in the experimental FRP contribution to the shear strength.

For closed FRP configurations, models generally assumed failure at the bottom corner of the section and predictions were very conservative where failure was experimentally observed along the web. This is the case of the formulation included in Annex J, with a mean value of the experimental to the theoretical ultimate shear force ranging between 1.21 and 2.91 for beams without transverse reinforcement and coefficient of variation (COV) ranging from 38 to 49%. For beams with rectangular section with internal stirrups and continuous CFRP configuration, the mean value is less conservative than for the remaining cases, 0.83 with a similar COV. For open configurations, results depended mainly on the assumed bond model and are more accurate than in the previous case, showing for some models unsafe predictions for the continuous FRP system applied in beams with transverse reinforcement. This is the case of the formulation of Annex J, where MV = 0.85 and 0.53 for rectangular beams with internal stirrups and with a discontinuous and continuous CFRP configuration, respectively.

According to fib Bulletin 90 [16], the contribution of anchored NSM reinforcement to the shear capacity of the element can be approximately computed with the same model of EBR.

### *4.4. Torsion, Punching and Design with strut-and-tie models*

Annex J doesn't give provisions for CFRP strengthening in torsion or in punching-shear. There is not enough data in the literature to include provisions related to both torsion and punching-shear.

## 5.
## SERVICEABILITY LIMIT STATES

The verification of serviceability limit states (SLS) considers the limitation of stresses to avoid steel yielding, damage or excessive creep of concrete, adhesives or FRP, or creep rupture of FRP, limitation of cracking and deflections. In some cases, SLS governs the design of the strengthening system, even the main purpose was the strength increase. The previous state of stresses and deflections should be considered in the verification of the SLS.

Under service load conditions, stresses in the concrete and in the longitudinal reinforcement of the strengthened structure are limited according to the main text of Eurocode. As a result of the limitations of the longitudinal tensile steel reinforcement stresses, the stress in the FRP should be limited due to compatibility reasons. Therefore, under the characteristic combination of loading, the stress in the EBR or NSM CFRP reinforcement should be limited to:

$$\sigma_f \leq 0.8\, f_{yk} \frac{E_f}{E_s} \tag{19}$$

In relation to cracking, it has been observed experimentally that the presence of the CFRP strengthened systems induces the appearance of new cracks in between the existing ones due to the additional tensile stress transfer from the CFRP system to the concrete (see Figure 10). These new cracks usually show smaller crack widths and might be less conditioning than for the unstrengthened element.

The strengthened element should fulfil the deflection limitations given by the main text of Eurocode 2 [1]. Deflections of beams or slabs strengthened with ABR may be estimated by ignoring the slip between the CFRP and concrete and transforming the area of CFRP to steel by taking account of the modular ratio, as considered in Annex J [1]. Deflections can be obtained for instance, by the double integration of the curvature, determined by a cross-section analysis along the RC element. In relation to long-term effects, they can be considered by considering the quasi-permanent load combinations and the modular ratios that consider the creep coefficient. However, there are limited existing studies about the long-term behaviour of concrete elements strengthened with FRP [49–51].

## 6. FATIGUE

Fatigue damage is not significant if the strengthened structure is exposed to typical service load ranges, but damage can occur if the load range exceeds 60% of load at first yield [52]. For this reason, special care should be taken in consideration, if the increase of service loading in the strengthened structures is significantly high. According to fib Bulletin 90 [16], in such cases under fatigue loading, failure occurs due to fracture of the longitudinal tensile reinforcement. Despite that FRP has an excellent fatigue strength, fatigue of bond should be considered since the loss of bond may lead to higher stresses in the longitudinal reinforcement with increased number of cycles.

### *6.1. Fatigue of externally bonded reinforcement (EBR) systems*

For EBR, Annex J presents a basic and a refined analysis for fatigue analysis for EBR CFRP systems, which is based on [18]. The basic analysis verifies that no damage due to cycling loading occurs, by limiting the increment of CFRP tensile forces in between cracks or at the laminate end to the elastic zone of the bond stress-slip relationship. Fatigue checking for EBR may be omitted if Equation (20) is accomplished. If this is the case, the upper load is limited by the load associated to the maximum bond stress. Then, strains are in the elastic range and no damage occurs.

$$\Delta F_{fE,equ} \leq \Delta F_{fRd,fat1} = 0.35\, f_{ctm,surf}^{1/4}\ f_{bfRd}\ b_f\ t_f \tag{20}$$

where:

$$\Delta F_{fE,equ} = max\,\{\, b_f\ t_f\ \Delta f_{fEd,max}\ ;\ F_{fEd,cr}\,\} \tag{21}$$

$f_{bfRd}$ is the limiting design strength of the bond in the area being considered, calculated according Equation (33).

$f_{ctm,surf}$ is the mean value of the tensile strength at the surface that can be determined by testing or estimated by Equation (31).

$\Delta f_{fEd,max}$ is the maximum difference in CFRP stress under the relevant load combination between cracks given by Equation (39).

$F_{fEd,cr}$ is the force in the CFRP at the first crack of the strengthened area.

If Equation (20) is not accomplished, the refined analysis should be performed. In the refined analysis, the fatigue range $S$ is used to obtain the tolerable number of cycles based on the $S$-$N$ curve. As explained in [25], the refined analysis uses the $S$-$N$ curve determined by some experimental data [53–58]. The number of load cycles needed for reaching a debonded length of 30 mm are calculated from the experimental programs of [53,54,57,58] by linear interpolation using the bond length and the number of cycles until completely debonding. For fitting the curve $S$-$N$, the unified related load ranges $S_{o,i}$, at a lower load level of 0, and the corresponding number of cycles $N_{30}$ is needed. $S_{o,i}$ is determined in a projection analysis using the Goodman relation. $S_o$ is the difference between the maximum and minimum load related to the monotonic quasi-static load carrying capacity of the interface. The curve $S$-$N$ has been obtained fitted to the experimental data.

Under the frequent combination, the following condition, Equation (22), should be checked:

$$\Delta F_{fEd,fat} \leq \Delta F_{fRd,fat2} = \alpha_{fat2} \frac{\Delta F_{fk,B}}{\gamma_{BA}} \tag{22}$$

where:

$\Delta F_{fEd,fat}$ is the design force range due to forces at the crack edge, $\Delta f_{f,max} - \Delta f_{f,min}$

$\Delta f_{f,min}$ is the minimum value of $b_f \cdot t_f \cdot \Delta f_{fEd}$ under the relevant fatigue load combination specified in Clause 10.2 of the main text of Eurocode 2 [1].

$\Delta f_{f,max}$ is the maximum value of $b_f \cdot t_f \cdot \Delta f_{fEd}$ under the relevant fatigue load combination specified in Clause 10.2 of the main text of Eurocode 2 [1].

$\Delta f_{fk,B} = b_f\ t_f\ \Delta f_{fEd}$ where $\Delta f_{fk,B}$ is calculated according Equation (44).

$$\alpha_{fat2} = -c_{fat} \frac{\Delta F_{f,max}}{\Delta F_{fRd}} + c_{fat} \tag{23}$$

$$\alpha_{fat} = 0.35 \left( \frac{N^*}{2\ 10^6} \right)^{-\frac{1}{k_{f3}}} \tag{24}$$

$N^*$ is the number of stress cycles.

$$k_{f3} = \begin{cases} 23.2 \quad for\ N^* > 2\ 10^6 \\ 23.2 \quad for\ N^* \leq 2\ 10^6 \end{cases} \tag{25}$$

### *6.2. Fatigue of near surface mounted reinforcement (NSM) CFRP strips*

In relation to fatigue of near surface mounted reinforcement, there is a low number of available tests (see [59]). Therefore, it is not possible to specify an *S-N* curve for this case, and then it is not possible to extrapolate the number of load cycles higher than that given in the tests results, which is $2{\cdot}10^6$ cycles. Annex J states that near surface mounted strips are adequate for fatigue under the frequent load combination given in Clause 10 of the main text, if the following conditions are accomplished:

1) The number of stress cycles is less than $2{\cdot}10^6$.
2) The maximum force in the NSM CFRP system, considering also the shift of the tension envelope into account does not exceed $F_{f,NSM,max}$ given by Equation (26).

$$F_{f,NSM,max} = 0.6\ f_{bfRd}\ b_f\ t_f \tag{26}$$

3) The strip stress range $\Delta\sigma_f$ accomplishes the condition given by Equation (27).

$$\Delta\sigma_f = \frac{\Delta F_{f,max} - \Delta F_{f,min}}{b_f\ t_f} \leq \frac{500}{t_f} \tag{27}$$

## 7. BOND AND ANCHORAGE OF ADHESIVELY BONDED CFRP SYSTEMS

### *7.1. Anchorage of externally bonded reinforcement (EBR)*

The existing experimental research in beams externally strengthened by plate bonding has commonly shown the appearance of premature failures due to loss of bond that involve the laminate debonding before reaching the design failure load (Figure 5). As mentioned in Oller *et al.* [60, 61], this debonding failure mode was observed in 375 well-documented FRP flexural strengthened beams without external anchorages assembled in a database. Bond failure implies the complete loss of bond between reinforcement and concrete substrate. Debonding can occur through the FRP, the adhesive, or the concrete cover, or in the FRP-adhesive or adhesive-concrete interfaces. The most common case is debonding along the concrete surface which is the weakest material in tension.

In flexural strengthening, laminate debonding can initiate at an intermediate section along the span at flexural or flexural-shear cracks along the span (intermediate crack IC debonding) or at the laminate end (end debonding) (see Figure 11). Even though both initiation points are critical for design or verification purposes, tests results compiled in the database of Oller [30] have shown that IC debonding is more common (70% of specimens) than end debonding (30% of specimens). For IC debonding, the laminate detachment involves a thin layer of the concrete due to the predominance of bond stresses and propagates towards the laminate end. A shear induced crack separation can also be observed due to the movement of the crack edges produced by the sear force. This latter case, will be treated separately even it initiates along the span length. In relation to end debonding, two types of failures can be observed according to *fib* Bulletin 90 [16]. The first one, named interfacial debonding at the anchorage zone, is related to the combination of bond and normal stresses at the laminate end, and usually involves the concrete layer adjacent to the adhesive interface and propagates from the laminate end to midspan. The second one is related to the shear deficiency of the RC element and is named end cover separation. This failure involves the ripping-off of the concrete cover along the longitudinal tensile reinforcement. Both failure modes can be avoided by providing end anchoring devices or shear strengthening in the second case.

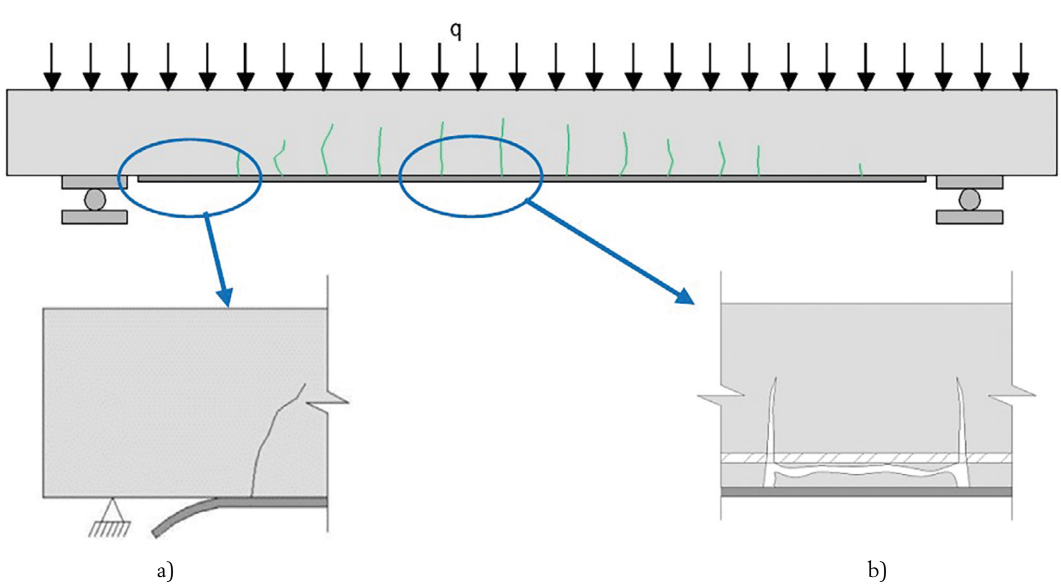



Figure 11. Debonding failure modes: a) end debonding, b) intermediate crack debonding.

For bond verifications, two areas will be distinguished: the end anchorage zone and the remaining length of the element. At the anchorage zone, the force of the CFRP at the outermost bending crack should be anchored along the length from this point to the CFRP laminate end. This element can be assumed as a pure shear specimen. In addition, end cover separation should also be checked. In the remaining beam length, intermediate crack debonding should be checked in between each pair of subsequent cracks. For each intermediate crack element, the increment of tensile forces in the CFRP laminate between both crack tips should be transferred by bond to the structural element.

The behaviour of a bonded joint can be described through the governing equations obtained from equilibrium and compatibility assuming a bond-slip relationship as the constitutive behaviour of the interface, which in this case can be assumed as a bilinear law (see Figure 12) defined by Equation (28).

$$\tau_f\,(s_f) = \begin{cases} \left(1-\dfrac{s_f}{s_{f0}}\right)\tau_{f1} & \text{where } s_f < s_{f0} \\ 0 & \text{where } s_f \geq s_{f0} \end{cases} \qquad (28)$$

where:

$\tau_{f1}$ is the maximum bond stress. The characteristic value, $\tau_{f1k}$, is given by Equation (29).

$$\tau_{f1k} = 0.37\; k_{sys,b1}\; (f_{cm}\;\; f_{ctm,surf})^{0.5} \qquad (29)$$

$s_{f1}$ is the slip associated to the maximum bond stress.

$s_{f0}$ is the ultimate slip. The characteristic value, $s_{f0k}$, is given by Equation (30).

$$s_{f0k} = 0.2\; k_{sys,b2} \qquad (30)$$

$k_{sys,b1}$ is a constant that can be taken as 1.0 unless more accurate information is available based on production data of the EBR system.

$k_{sys,b2}$ is a constant that can be taken as 1.0 unless more accurate information is available based on production data of the EBR system.

$f_{cm}$ is the mean concrete compressive strength.

$f_{ctm,surf}$ is the surface tensile strength of the prepared concrete surface to be bonded. If it cannot be determined, it can be estimated as a function of the position during concreting (top, side or bottom) according Equation (31).

$$f_{ctm,surf} = f_{ctm} \begin{cases} 0.3+0.6\left(\frac{f_{ck}}{60}-0.2\right) & \text{top} \\ 0.4+0.5\left(\frac{f_{ck}}{60}-0.2\right) & \text{side} \\ 0.6+0.3\left(\frac{f_{ck}}{60}-0.2\right) & \text{bottom} \end{cases} \qquad (31)$$

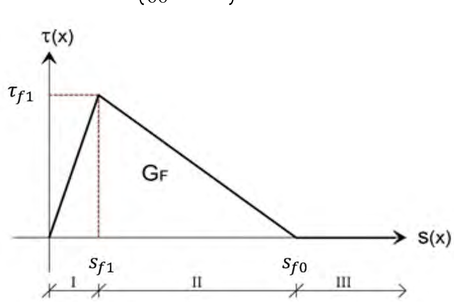



Figure 12. Bilinear bond-slip relationship for the interface.

#### 7.1.1. End anchorage

Annex J presents two methods to check end debonding: a refined method and a simplified method. Both methodologies are based on the bilinear bond-slip law given in Figure 12.

*a) Refined method*

For the simplified bond-slip relationship, the tensile stress $\sigma_f$ that can be anchored at the single crack may be determined as a function of the bond length $x = l_{bf}$ (see Equation 32).

$$f_{bf}\,(l_{bf}) = \sqrt{\frac{E_f\;\tau_{f1} - s_{f0}}{\tau_f}}\; sin\left(\sqrt{\frac{\tau_{f1}}{E_f\;\tau_f\;s_{f0}}}\;\; l_{bf}\right) \qquad (32)$$

Equation (32) can be replaced by a quadratic parabola as shown by Equation (33) where for design purposes, the characteristic values affected by the partial safety coefficient of bond, $\Upsilon_{BA}$, are used.

$$f_{bfRd}\,(l_{bf}) = \sqrt{\frac{\eta_{cc}\,k_{tc}\,k_{tt}}{\Upsilon_{BA}}}\, f_{bfk,max} \begin{cases} \dfrac{l_{bf}}{l_{bf,max,k}}\left(2-\dfrac{l_{bf}}{l_{bf,max,k}}\right) & \text{where } l_{bf} < l_{bf,max,k} \\ 1 & \text{where } l_{bf} \geq l_{bf,max,k} \end{cases} \qquad (33)$$

The maximum characteristic tensile stress that can be anchored is $f_{bfk,max}$ given by Equation (34).

$$f_{bfk,max} = \sqrt{\frac{E_f\;\tau_{f1k} - s_{f0k}}{t_f}} \qquad (34)$$

Values of $\eta_{cc}$, $k_{tc}$, and $k_{tt}$ are defined in accordance with Section 5.1.6 of [1].

The effective bonded length $l_{bfk,max}$, given by Equation (35), is the bond length beyond which the transfer force remains almost constant and is the minimum length that ensures the transfer of the maximum force or stress between the CFRP laminate and the concrete substrate.

$$l_{bfk,max,k} = \frac{\pi}{2}\sqrt{\frac{E_f\;\tau_f\;s_{f0k}}{t_{f1k}}} = \frac{2}{k_{sys,b3}}\sqrt{\frac{E_f\;\tau_f\;s_{f0k}}{t_{f1k}}} \qquad (35)$$

where:

$k_{sys,b3}$ is a constant that can be taken as 1.0 unless more accurate information is available based on production data of the EBR system.

*b) Simplified method*

Equation (33) can be simplified with the following definitions of the maximum characteristic tensile stress to be anchored $f_{bfk,max}$ and the effective bonded length $l_{bf,max}$.

$$f_{bfk,max,k} = \frac{0.2}{\Upsilon_{BA}}\sqrt{\frac{E_f\,(f_{cm}\,f_{ctm,surf})^{0.5}}{t_f}} \qquad (36)$$

$$l_{bfk,max,k} = \;1.5\;\sqrt{\frac{E_f\;t_f}{E_f\,(f_{cm}\,f_{ctm,surf})^{0.5}}} \qquad (37)$$

The simplified approach was assessed on the basis of a wide experimental database with more than 280 bond tests [62] on concrete elements strengthened with FRP strips or sheets with the following parameters: mean concrete strength, $f_{cm}$ = 15-62 N/mm²; modulus of elasticity of the FRP, $E_f$ = 82-400 GPa; FRP laminate thickness, $t_f$ = 0.083-1.6 mm ; 1-3 layers of sheets, also used in the *fib* Bulletin 90 [16].

In any case, according to [26], the EBR shall be anchored from the section where the existing structure is able to carry the design load forces without any additional strengthening system.

### *7.1.2. Intermediate crack debonding*

To avoid intermediate crack debonding, there are two approaches according to the state-of-the-art and the existing guidelines: a) to limit the maximum CFRP strain or stress [16,20,48,63–65] or b) to limit the increment of the tensile forces for each pair of adjacent cracks [60,61,66–69], which is a more accurate approach based on bond transfer. Annex J is based in the second approach. The formulation presented in this Annex to avoid intermediate crack debonding is based on a refined model included in the DAfStb [18] and in *fib* Bulletin 90 [16] with several simplifications. The mechanical model was developed by Finckh and Zilch [68] and its simplification was developed in [70].

The anchorage capacity between flexural cracks shall be enough to transfer the increment of tensile forces along the crack spacing. Therefore, the design value of the increment of CFRP laminate tensile stresses in between two adjacent cracks, $\Delta f_{fEd}$, should be limited to the bond strength in between these two sections, $\Delta f_{fEd}$, according to Equation (38), which should not be applied if the strain in the CFRP laminate exceeds 10 mm/m or if it exceeds the ultimate strain.

$$\Delta f_{fEd} \leq \Delta f_{fRd} \tag{38}$$

where:

$$\Delta f_{fEd} = \frac{F_{fEd,b} - F_{fEd,a}}{b_f\ t_f} \tag{39}$$

being $F_{fEd,a}$,$F_{fEd,b}$ the tensile forces at two adjacents cracks a and b, respectively.

According to Annex J, $\Delta f_{fEd}$ and $\Delta f_{fRd}$ should be calculated using the minimum crack spacing, $s_{cr,min}$, given by Equation (40) unless a more accurate analysis has been performed.

$$s_{cr,min} = 1.5\ \frac{M_{cr}}{0.85\ h\ F_{bsm}} \tag{40}$$

where:

$M_{cr}$ is the cracking bending moment of the unstrengthened section according to [1].

$F_{bsm}$ is the bond strength per length of the longitudinal reinforcing steel according to Equation (41).

$$F_{bsm} = \textstyle\sum_{i=1}^{n} n_{s,i}\ \phi_i\ \pi\ f_{bsm} \tag{41}$$

$$F_{bsm} = \begin{cases} k_{vb1}\, 0.43\ f_{cm}^{\ 2/3} & \textit{for ribbed bars} \\ k_{vb2}\, 0.28\ f_{cm}^{\ 1/2} & \textit{for plain bars} \end{cases} \tag{42}$$

$k_{vb1}$ and $k_{vb2}$ are parameters that depend on the bond conditions (for good conditions $k_{vb1}$=$k_{vb2}$=1.0 and for medium bond conditions, $k_{vb1}$=0.7 and $k_{vb2}$=0.5.

The bond strength between two adjacent cracked sections, $\Delta f_{fRd}$, can be obtained from Equation (43). It is constant for each pair of cracks, and considers the effects of bond friction, $\Delta f_{fk,F}$, clamping curvature of the beam, $\Delta f_{fk,C}$, and adhesive bond resistance between cracks, $\Delta f_{fk,B}$.

$$\Delta f_{fRd} = \frac{1}{\gamma_{BA}} \left( (\eta_{cc}\ k_{tc}\ k_{tt})^{0.5}\ \Delta f_{fk,B} + \Delta f_{fk,F} + \Delta f_{fk,C} \right) \tag{43}$$

$$\Delta f_{fk,B} = 0.84\ k_{sys,b1} \sqrt{f_{cm}\ f_{ctm,surf}}\ \frac{s_{cr,min}^{\ 0.5}}{t_f} \tag{44}$$

$$\Delta f_{fk,F} = f_{cm}^{\ -0.9}\ \frac{s_{cr,min}^{\ 4/3}}{t_f} \tag{45}$$

$$\Delta f_{fk,C} = \frac{k_h}{h_f}\ \frac{s_{cr,min}^{\ 0.5}}{t_f} \tag{46}$$

where:

$k_h$ is a parameter equal to 2000 for reinforced concrete (RC) elements and 0 for prestressed concrete (PC) elements

$h_f = min\{100\ mm, h\}$

$\eta_{cc}$, $k_{tc}$, $k_{tt}$ are defined in accordance with Section 5.1.6 of the main text of [1].

D'Antino and Triantafillou [36] performed an assessment of 11 analytical models for evaluating the effective strain in FRP strengthening systems or the increment of the FRP force along the crack spacing to prevent intermediate crack debonding and the model. The assessment was performed throudh the results of 154 RC beams collected from the literature. According to [36], the simplified and detailed approaches given by the German DAfStb [18], which is a similar approach to that given by Eurocode 2 [1] provide highly underestimated effective strain values. However, the German models were calibrated for applications of CFRP strips and do not cover other cases with different geometrical o mechanical characteristics of the strengthening system, that were inlcuded in the database.

Finck and Zilch [68] also applied their model, which is the original bases of the model included in DAfStb [18], to a database of 151 bending tests on single-span beams with CFRP strips that belong to a larger database of 473 tests with beams strengthened with CFRP sheets, steel or GFRP plates. Their comparison shows that the model fits very well with average tests values and that in most cases the mean values are not below the characteristic values.

### *7.1.3. End cover separation*

To avoid end cover separation, that is the detachment of the concrete layer beneath the reinforcement near the supports, the maximum design force at the end of the CFRP reinforcement should be lower than the value that generates this premature failure mode (see Equation (47)). The proposal included in Annex J for this limitation is in accordance to DafStb [18] and has been included in *fib* Bulletin 90 [16].

$$V_{Ed} \leq V_{Rd,cfE} = \left( 0.11 + 2.2\ \frac{(100\ \rho_l)^{0.15}}{\alpha_{fE}^{\ 0.36}} \right) (100\ \rho_l\ f_{ck})^{1/3}\ b_w\ d \tag{47}$$

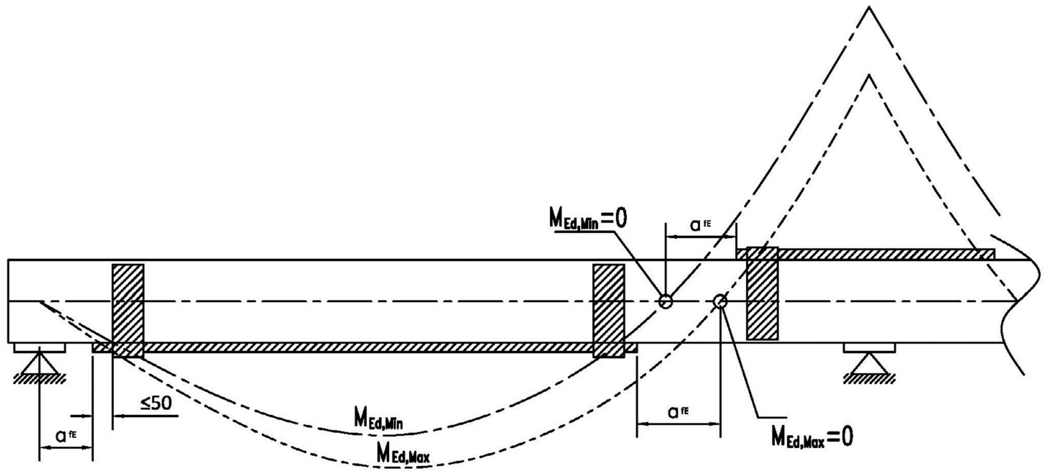



Figure 13. End concrete cover separation. Location of the shear strengthening system if required (adapted from Annex J).

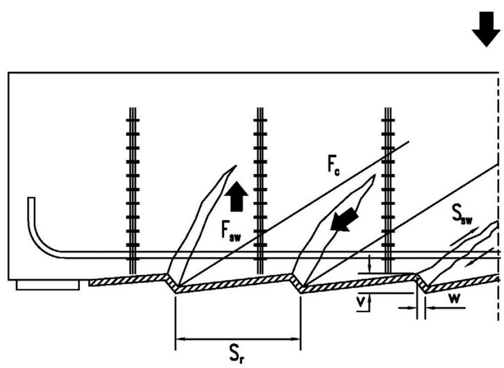



Figure 14. Shear induced separation, adapted from [18].

where:
$a_{fe}$ is the distance between the laminate end and the point of zero bending moment, and the remaining parameters are the longitudinal reinforcement ratio ($\rho_l$), the caracteristic compressive concrete strength ($_{fck}$), the web width ($b_w$) and the effective depth ($d$).

If Equation (47) is not met, it is required to provide shear stregthening at the end of the longitudinal CFRP laminate with enough anchorage provisions (see Figure 13).

### *7.1.4. Shear induced separation*

Shear induced separation refers to the delamination caused by the relative displacements in the crack tips between the crack edges due to the shear forces (Figure 14). This occurs if the strut and tie system is subjected to a high level of stresses (Figure 15). The CFRP laminate does not debond if Equation (48) and (49) are accomplished.

$$\frac{\tau_{Ed}\ \sigma_{swd}}{\tau_{Rd}} \leq \begin{cases} 75\ MPa & \text{for ribbed steel bars} \\ 25\ MPa & \text{for plain round steel bars} \end{cases} \tag{48}$$

$$\tau_{Ed} \leq 0.33\ f_{ck}^{\ 2/3} \tag{49}$$

Where:
$\tau_{Ed}$ and $\sigma_{swd}$ can be calculated according to the provisions of the general document of Eurocode 2 [1].

Adhesively bonded reinforcement may be required if the previous equations are not met to ensure that the tensile forces are transferred to the compression zone of the member. The design force for the shear EBR system can be obtained as the maximum of two components. The first component corresponds to the distribution of the design value of the total applied shear force over the elastic stiffness and the second one is the difference between the design value of the total applied shear force and the design value of the shear strength of the transverse reinforcement (see Equation (50)).

$$\tau_{Ed,f} = \max \begin{cases} \dfrac{E_f\ A_f}{E_f\ A_f + E_s\ A_s}\ \tau_{Ed} \\ \tau_{Ed} - \tau_{Rd,sy} \end{cases} \tag{50}$$

### *7.2. Anchorage of near surface mounted reinforcement (NSM)*

In relation to the anchorage of near surface mounted reinforcement, the formulation of Annex J is based again on the German DAfStb guideline [18] an on the fib Bulletin 90 [16]. As stated by Blaschko [71], it is assumed an effective composite action between the strengthening system and the concrete support that leads to a short anchorage length. Therefore, it is only required to check the anchorage length from the section where the near surface mounted reinforcement is not needed for the load carrying capacity.

According to Annex J, the design bond capacity per strip should be defined by Equation (51).

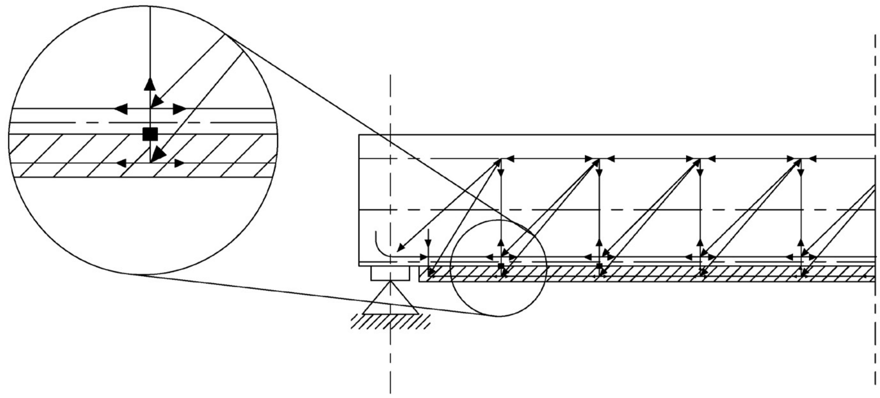


Figure 15. Strut-and-tie model with the mechanism for transferring tensile forces (adapted from *fib* Bulletin 90 [16]).

$$F_{bfRd} = \begin{cases} 0.95\, b_f\, \tau_{Ed} \sqrt[4]{a_r}\, lb_f (0.4 - 0.0015\, l_{bf}) & \text{for } l_{bf} \leq 115\ mm \\ 0.95\, b_f\, \tau_{bAd} \sqrt[4]{a_r}\, l_{bf} (26.2 + 0.065\, tanh\, (\frac{a_r}{70})\, (l_{bf} - 115) & \text{for } l_{bf} > 115\ mm \end{cases} \quad (51)$$

being $a_r$ the distance from the longitudinal axis of the strip to the free edge, which may not be larger than 150 mm.

The maximum design strength of the adhesive for the NSM systems, $\tau_{bAd}$ can be obtained by Equation (52).

$$\tau_{bAd} = \frac{1}{\Upsilon_{BA}} \min \begin{cases} \alpha_{bA}\, 0.6 \sqrt{\left(2\, f_{Atk} - 2\sqrt{f_{Atk}^2 + f_{Ack}\, f_{Atk}} + f_{Ack}\right) f_{Atk}} \\ \alpha_{bC}\, 4{,}5\, f_{cm}^{0.5} \end{cases} \quad (52)$$

where: $f_{Ack}$ and $f_{Atk}$ are the characteristic compressive and tensile strength of the adhesively, respectively, defined in Section 2 as a requirement of Annex for the design of ABR; $\alpha_{bA}$ may be taken as 0.5 unless the more accurate information is available based on production data of NSM CFRP strips; and $\alpha_{bC}$ may be taken as $(\eta_{cc} \cdot k_{tc} \cdot k_{tt})^{0.5}$ unless the more accurate information is available based on production data of NSM CFRP strips.

Factors 0.6 and 4.5 of Equation (52) have been calibrated by bond tests with CFRP strips with 10-30 mm width and 1-3 mm thickness. For found or square bars both factors should be recalibrated.

The NSM anchorage should accomplish the provisions for end cover separation given in 7.1.3 and shear induced separation given in 7.1.4.

## 8.
## DETAILING AND OTHER RULES

### *8.1. Detailing for flexural strengthening with adhesively bonded reinforcement (ABR)*

In the case of flexural strengthening with externally bonded reinforcement (EBR), Annex J recommends that the maximum spacing between strips, from centre to centre, should be lower than 0.2 times distance between points of zero moments; 3 times the thickness of the slab; 0.4 times the cantilever length and 400 mm.

The distance of the longitudinal edge of the strip from the member edge should be at least equivalent to the nominal concrete cover of the internal reinforcement.

Fib bulletin 90 [16] gives additional detailing rules on the location, arrangement and limitations for the FRP reinforcement required. Some of these rules are important to avoid premature debonding of the strengthened system.

For NSM CFRP systems, slots cut into the cover concrete should be located such that the cover is not adversely compromised when considering the accuracy of installation equipment along with adequate tolerance for installation.

Annex J provides Table 2 with some geometrical recommendations for NSM reinforcement. Slot dimensions, distance from the slot to the edge of the element and spacing between adjacent slots are important details to avoid premature debonding failure of the strengthened element.

### *8.2. Permissible parameters*

Annex J give additional permissible parameters such as the radius of bending, the number of sheets and strips, and lapping of the closed wrapped systems for shear strengthening or confinement.

In the case of straight prefabricated ABR CFRP bars, bending radius should be larger than 1000 times their thickness, unless stresses that arise from the bending process are considered in determining the tensile strength $f_{fuk}$.

In relation to the number of allowed layers, no more than five layers of CF sheets should be bonded for flexural or shear strengthening and no more than ten layers for column confinement. In the case of CFRP strips, no more than two layers should be bonded and the maximum thickness of the CFRP strip cross section should not exceed 3 mm (excluding adhesive). For the NSM systems, no more than one strip or bar should be bonded per slot.

When strengthening beams in shear or in the case of column confinement with a closed wrapped system, overlapping of the sheets or slips should be considered.

### *Conclusions*

This paper summarizes the content of the informative Annex J, developed by CEN/TC250/SC2/WG1/TG1 and Pro-

**TABLE 2.**
**Geometrical limits for NSM CFRP reinforcement in the form of bars or strips**

| Geometrical limits | Square NSM CFRP bars | Round NSM CFRP bars | NSM CFRP strips |
|---|---|---|---|
| $b_{slot}$ (slot width) | $t_f + 2 \leq b_{slot} \leq t_f + 6$ | $t_f + 2 \leq b_{slot} \leq t_f + 6$ | $t_f + 2 \leq b_{slot} \leq t_f + 4$ |
| $t_{slot}$ (slot thickness) | $b_f + 1 \leq t_{slot} \leq b_f + 3$ $t_f \leq b_f$ | $\phi_f + 1 \leq t_{slot} \leq \phi_f + 3$ | $b_f \leq t_{slot} \leq b_f + 2$ |
| $a_r$ (distance from the slot to the edge of the NSM) | | $a_r \geq 4\ b_f$ | |
| $s_f$ (centre-to-centre spacing of CFRP reinforcement) | | $max\,\{3\,b_{slot}\,;\,d_g\} \leq s_f \leq \min\{0.2\ l_{0b}\,;\,3\,h\}$ | |

ject Team 3, that for the first time incorporates in Eurocode 2 provisions the design of strengthening existing concrete structures with CFRP adhesively bonded systems. More detailed information about the formulations included in this Annex can be found in the *fib* Bulletin 90 [16], published in 2019, which was also intended to serve as a background document.

CFRP laminates or bars are linear elastic up to failure. Therefore, linear elastic analysis with limited redistribution or plastic analysis are not allowed.

Annex J includes design provisions for strengthening existing reinforced or prestressed concrete structures in flexure, shear or confinement with passive EB or NSM CFRP reinforcements. When designing CFRP strengthening systems in flexure or shear, the laminate might debond before reaching the ultimate bending moment or shear force. This premature failure mode should be correctly predicted during design, especially in the externally bonded case, more prompt to debond than the NSM since it is only bonded by one side of the strip.

Despite the FRP system might be design to increase the strength of the existing concrete cases, SLS might governs the design and should also be checked.

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