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Embedded Fibre Reinforced Polymer (FRP) Reinforcement in Concrete Structures According to the New Version of Eurocode 2

Armaduras de polímeros reforzados con fibras (PRF) para estructuras de hormigón en la nueva versión del Eurocódigo 2

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ABSTRACT

The new version of Eurocode 2 will include for the first time an informative annex, Annex R “Embedded FRP reinforcement”, to design reinforced concrete structures with fibre reinforced polymer (FRP) reinforcement. FRP embedded reinforcement has some advantages such as their low susceptibility to corrosion, high-strength, and low life-cycle cost. FRP rebars can be used as longitudinal or transverse reinforcement in a similar way than conventional steel rebars. However, in the design of FRP reinforced concrete structures, some particular aspects related to the reinforcement properties must be taken into account, among which it is worth highlighting their linear elastic behaviour until failure, their relatively low modulus of elasticity or their behaviour under sustained stresses. Since, the content of Annex R is new, a summary and background related to all aspects required for designing with FRP reinforcement are given in this paper.

KEYWORDS: Embedded FRP reinforcement, fibre reinforced polymer, reinforced concrete, Eurocode 2.

RESUMEN

La nueva versión del Eurocódigo 2 incluirá por primera vez un anejo informativo, el Anejo R “Armadura embebida de FRP”, para diseñar estructuras de hormigón armado con armaduras de polímeros reforzados con fibras. Estas armaduras tienen ventajas como su baja susceptibilidad a la corrosión, elevada resistencia y bajo coste de ciclo de vida. Las barras de FRP se pueden utilizar como armadura longitudinal o transversal de manera similar a las barras de acero convencionales. No obstante, en el cálculo de las estructuras de hormigón armadas con barras de FRP, hay algunos aspectos específicos que deben ser tenidos en cuenta, entre los que cabría destacar su comportamiento elástico lineal hasta rotura, su relativamente bajo módulo de elasticidad o su comportamiento con carga mantenida a largo plazo. Dado que el contenido del Anejo R es nuevo, en este artículo se proporciona un resumen del mismo y los antecedentes relacionados con todos los aspectos necesarios para dimensionar con armadura de FRP.

PALABRAS CLAVE: Armadura interna de FRP, polímeros reforzados con fibras, hormigón armado, Eurocódigo 2.

1. INTRODUCTION

Fibre reinforced polymer (FRP) embedded reinforcement can be an alternative to conventional steel reinforcement in concrete structures exposed to aggressive environments, where magnetic neutrality is required, or in some applications where good cuttability may be an advantage (for instance, a “soft eye” area of a diaphragm wall that will be cut by a Tunnel

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Figure 1. Real applications of FRP embedded reinforcement. a) Metro of Paris (courtesy of Schöck), b) Tramway in Liège (Belgium) (courtesy of Sireg), c) Highway sea wall in Maui (courtesy of Owens Corning) [8].

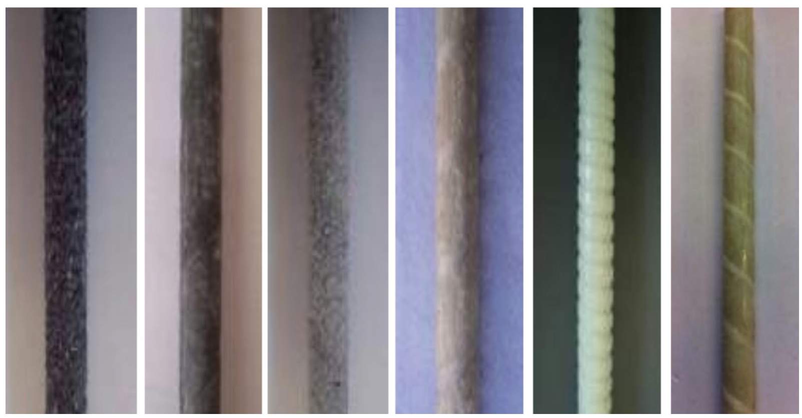


Figure 2. FRP reinforcement [9].

Boring Machine). Its low susceptibility to corrosion leads to longer service life, less maintenance and low life-cycle cost. Additionally, they present low density (ease of handling) and high fatigue endurance [1–5]. FRP rebars appeared in the market in the early 1990s. The first design guidelines for FRP reinforced concrete (RC) were introduced in Japan in 1997 [6]. Most initial applications of FRP reinforcement in concrete were made in Japan but nowadays there are applications worldwide (see Figure 1).

FRP reinforcement consists of continuous fibres of glass (in the case of GFRP), carbon (CFRP), basalt (BFRP) or aramid (AFRP) embedded in a polymeric resin. The fibres contribute with a high-strength and high-stiffness, and the matrix bind the fibres together and transfer the forces between fibres. FRP embedded reinforcement bars are usually manufactured through a pultrusion process where fibres are pulled and impregnated in a resin bath before curing by heat. To increase bond between bar and concrete, there are different surface treatments, such as sand coating, performing surface indentations, over-moulding a new surface on the bar or a combination of these techniques [4,7] (see Figure 2).

The basic principles of design for steel RC can be applied to FRP RC elements, however, the changes in properties of FRP reinforcement may have a different influence on the design [10,11]. Unlike steel, FRP reinforcement behaves linear elastic up to failure and does not yield. Additionally, FRPs subjected to constant stresses may present creep rupture, i.e. failure at a lower strength than the short-term strength, which can be influenced by adverse environments [4,12]. The linear behaviour of the FRP reinforcement up to high failure stresses leads to a different response of FRP RC members.

The modulus of elasticity of the FRP embedded reinforcement, and in particular for GFRP or BFRP, is much lower than that of steel. This affects bending and shear design, as well as serviceability conditions. Despite the absence of yielding, a proper design leads the FRP RC members to exhibit large deformability at failure. However, the low stiffness of FRP reinforcement may result in large crack widths and deflections, making the design often governed by serviceability requirements [13].

In recent years, concrete structures with embedded FRP reinforcement have successfully been applied in many projects all over the world, nevertheless, the lack of codes and standards equivalent to those for steel has been recognized as a limitation for its normalized use. Annex R of Eurocode 2 (FprEN 1992-1-1:2021) [14] is an informative annex that includes guidance for the design of new RC structures with FRP embedded reinforcement in the form of bars or mesh. Despite the several types of fibres, only glass (GFRP) and carbon (CFRP) reinforcement is covered by this annex. Although there are some recommendations for the use of prestressed FRP reinforcement [15,16] in the final version of Annex R it has been considered that there is not enough experience to cover it. Annex R applies only to normal weight concrete elements and not to lightweight concrete or concrete with recycled aggregates, as well as elements subjected predominantly to static loads, that is, with a maximum stress range of 10 % of $f_{ftk,100a}$ (long-term tensile strength, see Section 3) with a maximum stress $0.5f_{ftk,100a}$ for a maximum of $2x10^6$ cycles.

This paper aims to introduce the content of Annex R [14] since this is the first time that the design of FRP embedded reinforcement has been introduced in Eurocodes. Model Code 2010 [17] already introduced FRP reinforcement in the chapter of Materials (section on Non-metallic reinforcement) and Interface characteristics (section on Bond of non-metallic reinforcement). There are also some existing guidelines or codes such as the ACI 440.1R-15 [3] (which is currently developed as a Code), CSA S806-12 [18], CNR-DT 203/2006 [19] and JSCE [6]. The fib Bulletin 40 [4] was published in 2007 and gave the background of the main physical and mechanical properties of FRP reinforcing bars, as long as the design models to verify the ultimate and serviceability limit states. This fib bulletin was based on the expertise of the members of fib TG9.3 "FRP Reinforcement for concrete structures". In addition, there is a background document of Annex R [20] with more details about its content.

2. BASIS OF DESIGN

In general, the basis of design of concrete structures with conventional materials can be applied to concrete structures reinforced with FRP longitudinal rebars or transverse stirrups. 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 gives the partial safety factors for FRP reinforcement that consider also model uncertainty.

These partial safety factors have been obtained assuming a reliability index $\beta = 3.8$ and are based on the characteristic long-term strength of the FRP reinforcement, $f_{ftk,100a}$, which will be defined in §3, and on the short-term strength, f_{ftk}. According to the mentioned background document of Eurocode 2 [20], a reduction can be applied if the supplier can demonstrate the required reliability.

For beams with FRP transverse reinforcement, it has been observed experimentally that the strength in the bent area reduces in comparison to that in the straight part of the stirrup [21–24]. This reduction is a function of the geometry, the material properties and the manufacturing process. Long term reductions between straight and bent shapes are expected as well.

TABLE 1.
Partial safety factors for FRP embedded reinforcement [14].

Design situation	γ_{FRP}
ULS (Persistent and transient)	1.50
Accidental	1.10
Serviceability	1.00

3. MATERIALS

The design rules in Annex R are for members reinforced with embedded FRP reinforcement that meet the following conditions:

- Minimum modulus of elasticity of $E_{fR} \geq 40000$ *N/mm²*
- Ratio of $f_{ftk,100a}/E_{fR} \geq 0.005$
- Minimum long term bond strength of $f_{bd,100a} \geq 1.5$ *MPa*
- Characteristic compressive strength of concrete $f_{ck} \geq 20$ *MPa*
- Members with longitudinal reinforcement ratio $\rho_{lf} \leq 0.05$

The previous limits have been selected to reflect the values of testing specimens used for the calibration of the formulations. These values correlate with products widely available on the market and cover all usual types of reinforcement (ARFP, GFRP, CFRP, BFRP). The limit for the ratio of the long-term strength and the elastic modulus is given to avoid brittle failure. The lower limit on compressive strength of concrete results from the limit on the parameter $f_{bd,100a}$. The maximum reinforcement ratio ρ_{lf} was introduced to avoid an excessive amount of reinforcement and facilitate constructability and placement of concrete.

Annex R requires the definition of the following properties of FRP systems for design according to Eurocode 2 [14]: $f_{ftk,0}$, characteristic short-term tensile strength of the FRP and E_{fR} tensile modulus of elasticity of the FRP, both determined according to ISO 10406-1 [25]; and nominal diameter.

When designing a concrete structure with FRP reinforcement, the designer should specify the following properties, that should be provided by the manufacturer to ensure a performance as assumed in design: section sizes and tolerances; minimum characteristic short-term and long-term tensile strength ($f_{ftk,0}$ and $f_{ftk,100a}$, respectively); tensile modulus of elasticity, E_{fR}; long-term bond strength, $f_{bd,100a}$; strain at design tensile strength of FRP shear reinforcement, ε_{fwRd}; installation temperature; maximum and minimum temperature of the FRP reinforcement for the design lifetime of the structure; exposure classification; and durability requirements.

The provision of the design properties that should be considered by the manufacturer is given in the Annex because there is not yet a European Standard or execution standard, or European Assessment Document (EAD), for FRP reinforcement.

As previously mentioned, the behaviour of FRP reinforcement under tension is linear elastic and should only be considered as tension reinforcement. In addition, due to the effect of creep rupture under sustained stresses, there might be a significant reduction in the strength over the time.

For this reason, in relation to the design assumptions for the mechanical properties of the embedded FRP reinforcement, the design tensile strength of embedded FRP reinforcement shall be taken as:

$$f_{ftd} = \frac{f_{ftk,100a}}{\gamma_{FRP}} \tag{1}$$

where:
$f_{ftk,100a}$ is the design long-term strength, that can be obtained through tests or either by eq. (2) when it is not directly deter-

mined by production data. The long-term tensile strength is evaluated as the characteristic value of the stress leading to a 5% probability of a failure under 100 years of sustained stress in 40°C wet concrete.

$$f_{ftk,100a} = C_t \; C_c \; C_e \; f_{ftk0} \qquad (2)$$

C_t is the factor that considers the temperature effects and can be defined as:

C_t = 1.0 for indoor and underground environments,

C_t = 0.8 for outdoor members if heating through solar radiation cannot be excluded;

C_c is the ratio between the strength under sustained load and the strength under short-term load, that may be determined according ISO 10406-1 [25]. This value shall be taken as 0.35 for GFRP reinforcement and 0.8 for CFRP reinforcement, unless more accurate values are determined.

C_e is the ratio between the strength before ageing and after ageing, and may be determined according to the test concept in ISO 10406-1 [25] with exposure to 60°C for a duration of 3000 h. The value shall be taken as 0.7, unless more accurate values are determined.

The design long-term tensile strength considers the decrease in the short-term tensile strength due to sustained stresses, time, temperature and environmental influence. The previous coefficients are conservative and more accurate values can be obtained by performing tests defined in the EAD to directly obtain them. The background document of Eurocode 2 [20] describes tests methods to obtain in a direct way the value of $f_{ftk,100a}$, following tests setups from ISO 10406-1 [25] or comparable international standards [20]. These methods are based on the principles of linear reduction of the residual tensile strength in a time logarithmic scale and the time-temperature shift (i.e. an increase of temperature in the test is equivalent to a certain increase of time in the original temperature). This way $f_{ftk,100a}$, can be extrapolated from tests results with shorter times (i.e. some months)

Without tests and by using eq. (2), a typical design value for the long-term strength of a GFRP rebar in an outdoor element and in a persistent ULS situation might be calculated as indicated in eq. (3). A conservative value for design is obtained.

$$f_{ftd} = \frac{f_{ftk,100a}}{\gamma_{FRP}} = \frac{C_t \; C_c \; C_e \; f_{ftk0}}{\gamma_{FRP}} = \frac{0.8 \;\; 0.35 \;\; 0.7}{1.5} = 0.13 \, f_{ftk0} \qquad (3)$$

The stress-strain relationship for FRP embedded reinforcement is linear elastic up to failure. Figure 3 shows the characteristic short-term tensile stress f_{ftk0}, the long-term tensile strength $f_{ftk,100a}$ and the design value of the tensile strength, f_{ftd}.

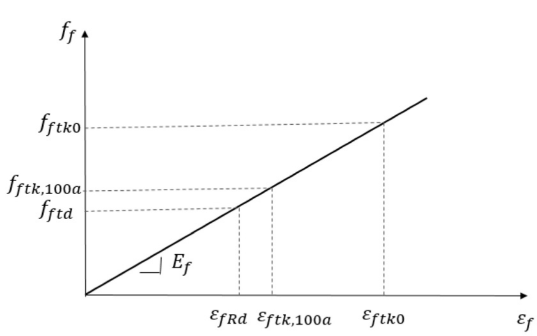


Figure 3. Stress-strain relationship for FRP embedded reinforcement.

Annex R gives also mean values of the FRP density for design purposes (2000 kg/m³ for GFRP and 1650 kg/m³ for CFRP reinforcement) and the coefficient of thermal expansion in the longitudinal direction ($5 \cdot 10^{-6}$ K^{-1} for GFRP and 0 for CFRP bars).

4.
DURABILITY

Durability conditions for design are defined by the main text of Eurocode 2 [14]. However, Annex R gives some provisions related to the concrete cover for reinforced concrete structures with FRP reinforcement. The nominal concrete cover, c_{nom}, is the sum of the minimum value, c_{min}, plus an allowance in design for deviation, as for steel RC members, Δc_{dev} The minimum concrete cover is defined as eq. (4).

$$c_{min} = \max \{c_{min,dur} + \Sigma\Delta\, c\, ; c_{min,b}\, ; 10 \text{ mm}\} \qquad (4)$$

In particular, for FRP reinforcement, $c_{min,dur}$, which is the minimum cover required for environmental conditions, is set to zero because corrosion induced by carbonation or chlorides does not occur for FRP reinforcement.

Unless more accurate information based on tests is available, the cover for transmission of forces by bond between reinforcement and concrete should be taken as $c_{min,b} \geq 2\phi$, being ϕ the bar diameter. At least the minimum cover for the FRP reinforcement shall be taken as $c_{min,b} \geq 1.5\phi$ and $c_{min,b} \geq 10$ mm. The concrete cover due to bond requirements may be higher than for steel reinforcement (where the minimum is 1ϕ), because of possibly higher splitting forces. According to the background document [20], for the same force to be anchored, higher slip values and higher splitting forces may occur for FRP reinforcement because of the lower modulus of elasticity and the bar surface.

One issue to keep in mind regarding corrosion is that CFRP reinforcement can form an electrical circuit which can cause corrosion in steel reinforcement in case of an electrical conductive contact. For this reason, direct contact of CFRP and steel reinforcement should be avoided.

Annex R does not address directly all the effects that might induce the deterioration of FRP in concrete (effect of water, chlorides, alkali, sustained stress, ultraviolet radiation, carbonation, acid attack, thermal actions). This might be justified by the limited design data available that can be used by design engineers related to this topic as mentioned in fib Bulletin 40 [4]. This is due to the lack of international agreement on FRP durability test methods, variability in production and variability in fibres, resin and FRP types.

5.
STRUCTURAL ANALYSIS

As explained in §3, FRPs are a linear elastic material up to failure. Therefore, linear elastic analysis with limited redistribution and plastic analysis shall not be undertaken for the case of RC elements with FRP embedded reinforcement. In addition, design with strut and tie models and stress fields for concrete elements with FRP reinforcement are not covered by this Eurocode [14].

6. ULTIMATE LIMIT STATES

6.1. Bending with and without axial forces

The design of longitudinal FRP embedded reinforcement for bending, follows equilibrium and compatibility as in a reinforced concrete elements with conventional reinforcement [3,4,10]. FRP RC sections may fail either by crushing of the concrete or FRP rupture and both modes of failure are accepted in Annex R. The main particularities are that FRP reinforcement does not yield as in the case of steel and additionally it is available in a large variety of properties (short-term strength, long-term strength, modulus of elasticity) all of them having incidence on the design [11]. The absence of yielding limits the tensile strain in FRP reinforcement to the design rupture strain, ε_{fRd} (see Figure 3).

Compression reinforcement is assumed to not contribute to the strength of the element. For columns or elements subjected to compression axial forces, unless more rigorous analysis is undertaken the benefit of the confining effect of FRP reinforcement should be reduced by the ratio E_{fR}/E_s in any direction that confinement is considered. This is because confinement is less effective for materials with lower modulus of elasticity [20,26].

6.2. Shear

Existing studies [27–31] have shown that the same shear resisting mechanisms can be assumed to develop in beams with FRP reinforcement and in beams with conventional steel reinforcement. However, the resisting mechanisms degrade at higher rates than in conventional RC beams because, FRP reinforced beams develop larger and deeper cracks [32], and less shear can be transferred by aggregate interlock. Therefore, provisions of §8.2 of the main text of Eurocode 2 [14] can be applied to elements with FRP longitudinal reinforcement by applying some modifications that are explained in this section. As a summary, the procedure to verify the shear strength of linear members and the out-of-plane shear strength of planar members consists of three different steps:

Step 1. If the design average shear stress over the cross-section, τ_{Ed}, is lower than the minimum shear resistance, $\tau_{Rdc,min}$, a detailed verification of the shear resistance may be omitted.

$$\tau_{Ed} \leq \tau_{Rdc,min} \tag{5}$$

where:

τ_{Ed} is the average shear stress defined as eq. (6):

$$\tau_{Ed} \begin{cases} \dfrac{V_{Ed}}{b_w z} & \textit{for linear members} \\ \dfrac{V_{Ed}}{z} & \textit{for planar members} \end{cases} \tag{6}$$

being:

- V_{Ed} the design shear force at the control section for linear elements
- V_{Ed} the design shear force per unit width at the control section for planar elements
- b_w is the width of the cross-section of linear members and is the smallest width of the cross-section between the tension chord and the neutral axis for sections with variable width.
- z is the lever arm defined as $z = 0.9\,d$, where d is the effective depth, that is the distance between the most compressed fibre to the centroid tensile reinforcement.
- $\tau_{Rdc,min}$ is the minimum shear resistance of elements with FRP longitudinal reinforcement without transverse stirrups, based on the Critical Shear Crack Theory (CSCT) and is given by eq. (7):

$$\tau_{Rdc,min} = \frac{11}{\gamma_v}\sqrt{\frac{f_{ck}}{f_{ftk0}}\,\frac{E_{fR}}{E_s}\,\frac{d_{dg}}{d}} \tag{7}$$

where:

- γ_v is the partial safety factor defined in Table 1 of §2.
- f_{ck} is the characteristic value of the concrete compressive strength.
- f_{ftk0} is the characteristic short-term strength of the FRP embedded reinforcement given by the manufacturer (see Figure 3).
- E_{fR} is the modulus of elasticity of the FRP reinforcement.
- d_{dg} is a size parameter that describes the failure zone roughness, which is a function of D_{lower}, the smallest value of the aggregate size.

$$d_{dg} = \begin{cases} 16mm + Dl_{ower} \leq 40mm \textit{ for concrete with } f_{ck} \leq 60\ MPa \\ 16mm + Dl_{ower}\,(60/f_{ck})^2 \leq 40mm \textit{ for concrete with } f_{ck} > 60\ MPa \end{cases} \tag{8}$$

Step 2. If the design average shear stress over the cross-section, τ_{Ed}, is lower than the design value of the shear resistance, $\tau_{Rd,c}$, no calculated shear reinforcement is required (see §8.2.2 of [14]).

$$\tau_{Ed} \leq \tau_{Rd,c} \tag{9}$$

In this case, for elements without shear reinforcement, the formulas provided in §8.2.2 of the main text of Eurocode 2 [14], to obtain the ultimate shear strength, can be adapted for the FRP embedded reinforcement by reducing the longitudinal reinforcement ratio ρ_{lf} by the ratio E_{fR}/E_s. This is to account for the lower stiffness of the FRP reinforcement in comparison with conventional steel. This modification is also applied in other codes or guidelines such as the ACI440.1R-15[3], CNR-DT203/2006 [19] and JSCE [33]. Therefore, $\tau_{Rd,c}$, can be obtained as eq. (10):

$$\tau_{Rd,c} = \frac{0.66}{\gamma_V}\left(100\ \rho_{lf}\,\frac{E_{fR}}{E_s}\,f_{ck}\,\frac{d_{dg}}{d}\right)^{1/3} \geq \tau_{Rdc,min} \tag{10}$$

where:

$$\rho_{lf} = \frac{A_{f,1}}{b_w\,d} \tag{11}$$

- $A_{f,1}$ is the effective area of the FRP longitudinal embedded reinforcement.

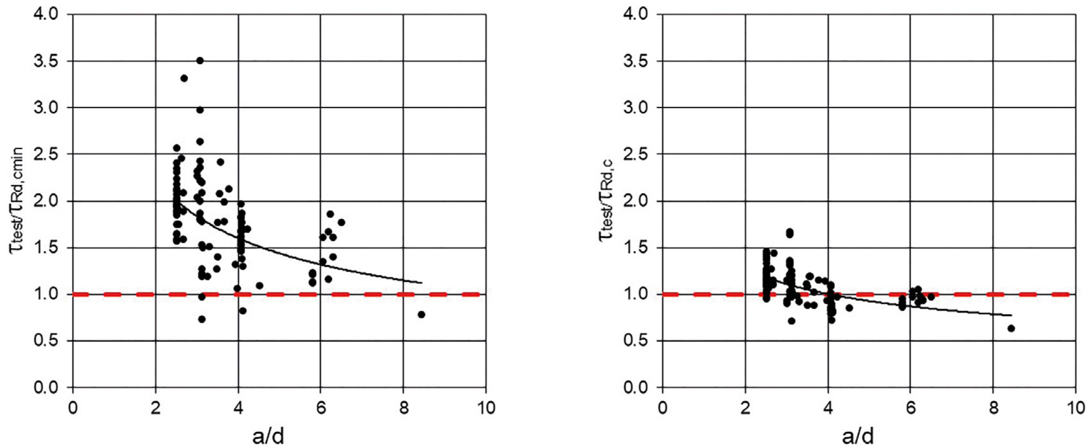


Figure 4. a) Experimental average shear stress vs. minimum shear strength, including trend, b) Experimental average shear stress vs. shear strength for elements without FRP shear reinforcement, including trend.

In the presence of tensile forces, equations given in §8.2.2 should not be applied if the height of the compression zone in the cracked state of the section is less than 0.1*d*.

Step 3. If eq. (9) is not satisfied, shear reinforcement is required. Then, provisions of §8.2.3 of the Eurocode [14] can be applied, but with the following modifications in order to adapt the formulation to the case with FRP longitudinal and transverse embedded reinforcement.

First of all, the shear stress resistance perpendicular to the longitudinal member axis shall be calculated according to eq. (12):

$$\tau_{Rd,f} = \tau_{Rd,c} + \rho_w\, f_{fwRd}\, cot\,\theta \leq 0.17\, f_{cd} \tag{12}$$

where:

$$f_{fwRd} = f_{fwk,100a} \;/\; \gamma_{FRP} \leq \varepsilon_{fwRd}\, E_{fwR} \tag{13}$$

$f_{fwk,100a}$ is the characteristic long-term shear strength of the FRP shear reinforcement
γ_{FRP} is the partial safety factor given in Table 1.
E_{fwR} is the modulus of elasticity of the FRP shear reinforcement.

$$\varepsilon_{fwRd} = 0.0023 + 1/15\; E_{fR}\, A_{fl}\, (0.8\; d)^2\, 10^{-15} \leq 0.007 \tag{14}$$

θ is the inclination of the compression field, and $cot\theta$ should be considered as eq. (15):

$$cot\theta = 0.8 \tag{15}$$

For the inclination of the struts, θ, Kurth et al. [34,35] developed a formula to determine this inclination based on the compression field theory. The calculated inclination ranged between 20° and 50°. The cotangent of the inclination is then used to compute the contribution of the shear stress resistance provided by the shear reinforcement. In a pragmatic sense, the cotangent of the inclination may be taken as 0.8, since this is a value on the safe side.

As explained in the background document [20], a database of shear tests without transverse reinforcement, compiled by Kurth [34], has been used to verify eq. (7). The experimental ultimate shear strength has been compared to the minimum shear resistance, with mean values for the material properties and without applying the partial safety factor, obtaining a safer estimated for longitudinal reinforcement with characteristic short term tensile strengths of 1400 N/mm². In addition, the experimental shear strength has been compared with the predicted value obtaining a good agreement. There are also other published database, such as that of [36] and [37]. In this paper, the same comparison has been done but with the database compiled in Marí et al. [36], observing the same conservative trend for the minimum shear resistance, as shown in Figure 4a. When predicting the shear strength with the database of beams with longitudinal FRP reinforcement and without FRP stirrups, a good agreement is observed (see Figure 4b). The mean value (MV) of the experimental average shear stress to the theoretical shear strength ratio is 1.07 and the coefficient of variation (CoV) is 17.59%.

The formulation for the shear resistance in conventional RC elements with transverse reinforcement is based on a strut-and-tie model, where the concrete contribution is included through the cotθ and the shear capacity is limited by stirrups yielding and by the carrying capacity of the struts. This approach cannot be directly applied to the FRP shear reinforcement because of it is linear elastic up to failure. In addition, it has been observed in some tests that a shear compression failure can occur before failure of the FRP transverse reinforcement. To take this into account, the shear strain in the reinforcement should be limited. Kurth et al. [35] proposed a strain limit that depends on the flexural stiffness. In addition, the capacity of the compression struts is modified because the larger deformations expected when using FRP reinforcement. Therefore, the efficiency factor for the capacity of the concrete strength is reduced to $v = 0.35$.

For elements with transverse FRP reinforcement, and based on the previous statements, an initial formulation which is a modification of the strut-and-tie model was firstly developed. However, when applying this formulation to the database compiled by Kurth [34], results are very conserv-

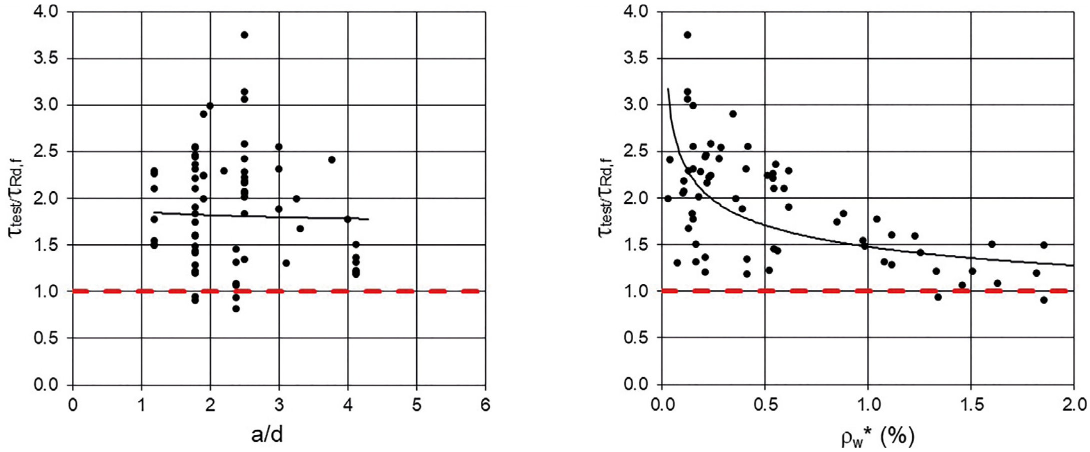


Figure 5. Experimental average shear stress to shear strength ratio for elements with FRP longitudinal and shear reinforcement vs a/d (a) and vs ρ^{*}_{w}, including trends.

ative, especially for small shear reinforcement ratios, where sometimes it was lower than the strength given by eq. (10). Then, to avoid an uneconomical design, an additive approach derived by Kurth et al. [34,35] and given by eq. (12) has been included in Annex R.

According to the experimental program performed by Sczech and Kotynia [38] of 22 beams, very small values of the shear strength are obtained when neglecting the concrete contribution to the shear strength of beams with FRP stirrups. The real θ achieved in their tests was much lower than the assumed $cot\theta$ = 0.8. So the calculated shear stresses obtained according eq. (12) are within a safe range.

Eq. (12) has been applied to the database compiled in Oller et al. [39] obtaining a mean value of the experimental shear stress to the theoretical shear strength ratio of 1.93 with a coefficient of variation of 36%, which is conservative. Figure 5 shows this ratio plotted as a function of the shear span to effective depth ratio and as a function of a modified transverse shear reinforcement ratio ($\rho^{*}_{w} = \rho w\ E_{fwR}/E_c$) observing a decreasing trend in this last case. In addition, almost all the specimens show a conservative ratio above 1.0.

For shear between web and flanges, provisions in §8.2.5 of [14] may be used by replacing f_{yd} by f_{ftd}, cotθ = 1.0, and v = 0.35.

In relation to shear at interfaces, §8.2.6 of [14] may also be used but after applying some changes. As mentioned in [14], the shear at the interfaces should be checked if the static equilibrium depends on the shear transfer across a given interface. Then, the shear transfer should accomplish eq. (16):

$$\tau_{Edi} \leq \tau_{Rdi} \tag{16}$$

where:

τ_{Edi} is the design value of the shear stress in an interface given by:

$$\tau_{Edi} = \frac{V_{Edi}}{A_i} \tag{17}$$

V_{Edi} is the shear force parallel to the interface.

A_i is the area of the interfaces according to §8.2.6.

τ_{Rdi} is the design shear resistance at the interface that can be calculated by eq. (18) if reinforcement is not required.

$$\tau_{Rdi} = c_{v1} \frac{\sqrt{f_{ck}}}{\gamma_c} + \mu_v\, \sigma_n \leq 0.17\, f_{cd} \tag{18}$$

where the definition of the parameters can be found in §8.2.6.

Finally, the provisions of the main text of Eurocode 2 related to not ensure yielding of the reinforcement crossing the interface due to insufficient anchorage do not apply for FRP reinforcement.

6.3. *Torsion*

The provisions for torsion of the main text of Eurocode 2 [14] are valid for elements with FRP reinforcement but after applying some changes, related to the definition of the longitudinal and transverse strength of the FRP rebars, because of their linear elastic performance. These changes mainly consists of replacing the longitudinal steel yielding f_{yd} by the design tensile strength of the FRP reinforcement, f_{ftd}, and the transverse steel yielding f_{ywd} by the design strength value of the FRP transverse reinforcement, f_{fwRd}.

Therefore, for a single cell, thin-walled section of a sub-section with a constant effective wall thickness, t_{eff}, the design torsional strength can be calculated as eq. (19):

$$\tau_{t,Rd} = min\ \{\tau_{t,Rd,sw}\ ;\ \tau_{t,Rd,sl}\ ;\ \tau_{t,Rd,max}\} \tag{19}$$

where:

$$\tau_{t,Rd,sw} = cot\theta\ \frac{A_{fw}}{t_{eff}\ s}\ f_{fwRd} \tag{20}$$

$$\tau_{t,Rd,sl} = \frac{\Sigma A_{fl}\ f_{ftd}}{t_{eff}\ u_k\ cot\theta} \tag{21}$$

$$\tau_{t,Rd,max} = \frac{v\ f_{cd}}{cot\theta + tan\theta} \tag{22}$$

being:

$$cot\theta = 1.0 \tag{23}$$

f_{fwRd} is given by eq. (13), but should be limited to $f_{fwRd} \leq 0.004\ E_{fwR}$.

f_{ftd} is given by eq. (1), but should be limited to $f_{ftd} \leq 0.004\ E_{fR}$.

u_k is the perimeter of the area A_k, which is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas.

t_{eff} is the effective wall thickness that may be taken as A/u, being A, the total area of the cross-section, including inner hollow areas and u, the outer perimeter of the cross-section.

$$\nu = 0.35 \tag{24}$$

For combined shear and torsion, the compatibility of strains has to be ensured because different approaches have been applied for shear and for torsion. In addition, the transverse reinforcement should be the sum of the reinforcement required for shear and for torsion.

6.4. Punching

The provisions for the punching-shear for slabs without shear reinforcement of §8.4.3 of [14] and with shear reinforcement of §8.4.3 of [14] shall not be applied to concrete slabs with longitudinal FRP reinforcement. This is because there is not enough database to validate any proposal. There are only few existing studies related to punching-shear with FRP embedded reinforcement.

TABLE 2.
Verifications, stress and crack width limits for appearance according to Annex R [14].

Verification	Calculation of minimum reinforcement according to §9.2.2	Verification of width according to §9.2.3	Verification of reinforcement stresses to avoid failure at SLS
Combination of for calculating σ_f	Cracking forces according to §9.2.2	Quasi-permanent combination of	Characteristic combination of
Limiting value of σ_f	$\sigma_f \leq f_{ftd}$	$w_{lim,cal} = 0.4$ mm $\sigma_f \leq f_{ftd}$	$\sigma_f \leq 0.8\ f_{ftd}$

TABLE 3.
Verifications, stress and crack width limits for durability according to Annex R [14].

Exposure class	Concrete members with FRP reinforcement	
	Combination of actions	
	Quasi-permanent	Characteristic
XC, XF, XD	$w_{lim,cal}$ =0.4 mm c)	$\sigma_c \leq 0.6\ f_{ck}$ a),b)

a) No limitation in serviceability conditions is necessary for stresses under bearings, partially loaded areas and plates of headed bars.

b) The compressive stress σ_c may be increased to 0.66 f_{ck} if the cover is increased by 10 mm or confinement by the transverse reinforcement is provided.

c) In absence of appearance conditions, fasteners, punctual wheel pressure, lap splice or freeze thaw, this limit may be relaxed to values up to 0.7 mm.

7. SERVICEABILITY LIMIT STATES (SLS)

Since the behaviour of FRP RC members is governed by the same principles of steel RC [2,11], the general equations in the main text of Eurocode 2 [14] apply. Again, the possible difference in the design solution will be due to the different properties of FRP reinforcement with respect to steel one. The design of FRP concrete is often controlled by SLS due to the lower modulus of elasticity in comparison with steel reinforcement.

7.1. Stress limitation and crack control

Stresses in concrete and reinforcement are limited in a similar way to steel RC members.

For elements with FRP embedded reinforcement, cracking shall be usually limited for appearance conditions to $w_{lim,cal}$ = 0.40 mm. In the absence of appearance conditions this limit may be relaxed.

Table 2 and Table 3 provide the verifications, stress and crack width limitations for elements reinforced with FRP according to Annex R. These tables are an adaptation from Tables 9.1 (NDP) and 9.2 (NDP) from the main text [14]. As observed, since FRP embedded reinforcement does not have corrosion problems, there is no need to limit the crack width for durability reasons, only for appearance, or in environments with freeze/thaw, and where wheel loads are present.

The provisions relevant to steel reinforcement for the calculation of minimum reinforcement areas (§9.2.2) and refined control of cracking (§9.2.3) may be applied to concrete with FRP reinforcement by replacing the parameters corresponding to steel reinforcement by those corresponding to FRP, under the assumption that the bond behaviour of both types of reinforcement are similar.

7.2. Deflection control

Existing equations for the calculation of deflections of FRP RC members are based on the same principles as for steel RC structures. Being the deformability an issue of major importance for FRP RC structures, a significant number of studies about their short-term deflections have been carried out [13,40–43]. With different levels of approximation, either double integration of curvatures or constant average stiffness along the member, the proposals lead to acceptable predictions [3,44–46].

Although less work has been done on long-term deflections, some proposals have also been presented [47–51]. Since long term curvatures (and deflections) of RC members are highly dependent on the reinforcement stiffness, $E_f\ A_f$, differences in behaviour with respect to steel RC arise from possible changes in that value. Similarly to short-term deflections, application of general analytical procedures provides reasonable predictions [51]. A practical alternative consists on the use of multiplicative coefficients to obtain long-term deflections from the short-term ones. Some proposals of multiplicative coefficients have been presented, either empirically modifying the values for steel RC [3,52] or analytically deducing factors from the general equations [47,51]. Some larger deviations may appear in the case of empirical methods [51].

Annex R proposes the application of the general method for deflection calculations in §9.3.4 of Eurocode 2 [14]. The simplified approach for deflections of steel RC building structures given in §9.3.3 of Eurocode 2 does not apply to FRP embedded reinforcement.

Likewise, the limits of span to effective depth ratios calibrated for steel RC flexural members given in section §9.3.2 do not apply to FRP RC structures. Some proposals with different levels of approximation and different framework (i.e. ACI, Eurocode) can be found in the literature as in [3,43,53,54].

8.
BOND AND ANCHORAGE OF FRP REINFORCEMENT

According to Annex R, the provisions related to detailing with FRP reinforcement apply only to straight rebars. The main text of Eurocode 2 [14] is valid for spacing between FRP embedded rebars.

In relation to the permissible mandrel diameters for bent rebars, the minimum diameter shall avoid damage of the FRP reinforcement and failure in the concrete inside the bend of the bar (crushing, splitting or spalling). To accomplish the first condition, the mandrel diameter may be found in the Technical Product Specification and should be at least:

$$\phi_{mand,min} = \begin{cases} 4\phi & for\ \phi \leq 16\ mm \\ 7\phi & for\ \phi > 16\ mm \end{cases} \qquad (25)$$

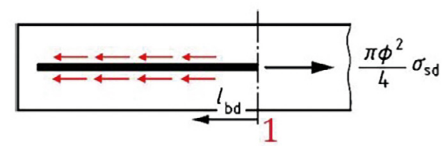
The verification of the concrete inside the bend may be omitted (provided that $f_{ftd} \leq 25 f_{cd}$ and $\gamma_c \leq 1.5$): for stirrups that accomplish conditions described in §12.3.3 of [14]; for standard hook and bend anchorage; if $c_x \geq 1.5\phi$ from an edge parallel to the bent and a clear distance between bars $c_s \geq 3\phi$ according to Figure 6c of [14]; or for all bends with an angle $\alpha_{bend} \leq 45^0$ at a clear distance $c_x \geq 2.5\phi$ from an edge parallel to the bent, a clear distance between bars $c_s \geq 5\phi$ and a length $\geq 2\phi$ of the straight segments between multiple bends.

For the remaining cases, the design value of the stress in the FRP rebar should accomplish eq. (26) to avoid concrete failure inside the bend area.

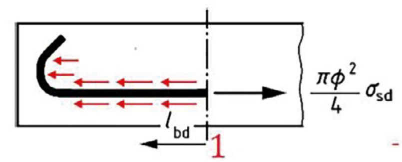
$$\sigma_{ftd} \leq 25\ f_{cd} \qquad (26)$$

One of the main limitations of the thermosetting rebars, which are the commercial FRP rebars, is bending at the construction site or re-bending which is not possible. The thermosetting based bars cannot bend once the matrix has solidified because they are fully cross-linked. Then, these thermosetting rebars should be manufactured with the required length and their bent configurations, and bending should only be done under controlled factory and controlled temperature conditions. There are also some additional limitations related to the number of bends per rebar, to the bent radius and to the spacing between two successive bends. Lap splices are required to overcome these problems which consume more material [55].

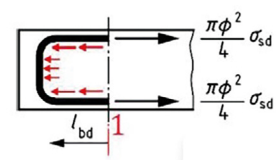
When anchoring FRP reinforcement in tension and compression, provisions of the main text of Eurocode 2 [14] may be applied except for the modifications included in Annex R. It is only possible to anchor the rebars by following only 3 of the 6 methods described in [14]. These methods are anchorage of straight bars, bend and hooks and loops (see Figure 6).



Anchorage of straight bars



Anchorage of bends and hooks



U-bar loops

Figure 6. Methods for anchoring FRP reinforcement [14].

Eq. (27) can be applied to determine the anchorage length of FRP reinforcement.

$$l_{bd} = k_{lb}\ k_{cp}\ \phi \left(\frac{\sigma_{ftd}}{217}\right)^{\eta_\sigma} \left(\frac{25}{f_{ck}}\right)^{1/2} \left(\frac{\phi}{25}\right)^{1/3} \left(\frac{1.5\sigma}{25}\right)^{1/2} \geq \begin{cases} 10\ \phi \\ \frac{\phi}{4}\ \frac{\sigma_{ftd}}{f_{bd,100a}} \end{cases} \qquad (27)$$

where:

$$\eta_\sigma = \begin{cases} 1.0 & for\ \sigma_{ftd} \leq 217\ MPa \\ 1.5 & for\ \sigma_{ftd} > 217\ MPa \end{cases} \qquad (28)$$

$f_{bd,100\,a}$ may be taken as 1.5 MPa unless there is more accurate information based on production data. This value has been conservatively defined for $f_{ck} = 20 MPa$.

$$c_d = min\ \{0.5\ c_s\ ;\ c_x\ ;\ c_y\} \qquad (29)$$

Figure 7 gives the definition of the parameters cover and clear distance between rebars, to obtain c_d.

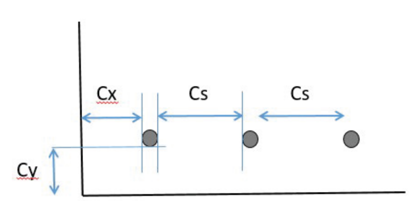


Figure 7. Concrete cover and clear distance between rebars to calculate c_d [14].

k_{cp} is a coefficient that accounts for casting effects on bond conditions. k_{cp} =1.0 for bars with good bond conditions, k_{cp} =1.2 for poor bond conditions, and k_{cp}=1.4 for all bars executed under bentonite or similar slurries.

k_{lb} is equal to 50 for persistent and transient design situations or 35 for accidental design situations unless a National Annex gives different values.

If the clear distance between FRP reinforcement bars c_s<7.5ϕ, concrete cover spalling shall be prevented by limiting the design strain to $\varepsilon_{fRd} \leq 0.0035$ in straight bars or with confining of the anchorage zone.

Laps splices for FRP reinforcement shall be placed in the zone where the stress in the reinforcement at ultimate limit state is less than 80% of the design strength.

The provisions of §11.4.4 of [14] for anchorage with bents and hooks, of §11.4.6 and of §11.5.4 for anchorage and lap splices with U-bar loops, respectively, may be applied with the assumption, that only the straight part is considered determining the anchorage length and that the design long-term tensile strength f_{fwRd} is considered.

The general provisions for bundles in anchorage or lap splices should not be applied for FRP reinforcement.

Provisions of this section have been checked in the background document [20] through a database of 126 available tests from 15 authors with GFRP-reinforced lap splices. Part of this database was previously analysed by [56]. According to this data, the influence of concrete strength and diameter is similar to that of reinforcing steel. Due to different surface preparations and to the different products tested, it is not clear enough the influence of the concrete cover and the bar spacing. The maximum strain limit for unconfined lap splices was given according to this database.

9.
DETAILING OF MEMBERS WITH FRP REINFORCEMENT

Detailing of the RC elements with FRP reinforcement should be consistent with the design models and rules included in Annex R. In general, detailing given in the main text of Eurocode 2 [14] can be applied except for the specific modifications given in Annex R, after applying the following changes: steel yielding, f_{yk}, is replaced by the design FRP tensile strength, f_{ftd}; the elasticity modulus of steel, E_s, is replaced by the modulus of FRP, E_{fR}; and the area of tensile steel reinforcement, A_s, is replaced by the area of FRP, A_f. Annex R only provides rules for straight FRP rebars.

The minimum area of FRP reinforcement for elements under pure tension is given by eq. (30).

$$A_{f,min} = A_c \ f_{ctm} / f_{ftd} \tag{30}$$

The minimum reinforcement shall be anchored and lapped following the previous section and considering a stress level of f_{ftd}.

9.1. Beams

In the case of beams with FRP longitudinal and transverse embedded reinforcement, reinforcement should be detailed following the requirements of Table 12.1 (NDP) of the main text of Eurocode 2 [14], but using $s_{max,l}$ < 250 mm.

Some rules are given for the minimum reinforcement: 1) it should be distributed over the width and proportionally over the height of the tension zone; 2) it should be fully provided between the supports; and 3) the area required for lever arm must be provided for the total length of the lever arm.

When distributing the longitudinal reinforcement, for members with constant depth, the bending moment law should be shifted at a distance a_l, that for members with and without shear reinforcement, it may be assumed $a_l = d$.

The shear reinforcement shall only consist of a combination of stirrups/links (enclosing the longitudinal tension reinforcement and the compression zone) or cages/ladders properly anchored in the compression and tension zones

Anchorages with headed bars or welded/connected transverse reinforcement are generally not considered for FRP reinforcement.

Laps on legs of stirrups in shear reinforcement may be used and designed according to the previous section of this paper and considering a stress level equal to the design tensile strength, f_{ftd}.

Annex R does not give provisions for the additional suspension reinforcement for indirect support of loads (i.e. intersection of primary and secondary beams or hanging loads).

9.2. Slabs

In the case of slabs with FRP longitudinal and transverse embedded reinforcement, reinforcement should be detailed following the requirements of Table 12.2 (NDP) of the main text of Eurocode 2 , but using $s_{max,slab}$, $s_{max,l}$,$s_{max,bu}$,$s_{max,tr}$ < 250 mm.

The minimum height of the concrete slab is 200 mm if shear reinforcement is provided.

In relation to the shear reinforcement, the maximum longitudinal spacing $s_{max,l}$ of shear stirrups is 0.3d instead of 0.75d.

Annex R does not give provisions for the minimum area of reinforcement for robustness in case of progressive collapse utilising FRP reinforcement in slabs. In addition, some of the rules for shear reinforcement do not apply in the case of FRP, in particular, rules are not provided for using FRP reinforcement for punching-shear, since there is not enough experience.

9.3. Columns and foundations

Although design criteria for columns and foundations can be found elsewhere (e.g. ACI 440.11-22 [57]), the new Eurocode 2 [14] does not provide rules for using FRP reinforcement under compression of for their use in foundations.

9.4. Other elements

For walls and deep beams, provisions of the main text can be used but using $s_{max,l}$< 250 mm. No changes are required with respect to the main text, when using FRP reinforcement for tying systems for robustness of buildings, supports, bearings and expansions joints.

For precast concrete elements and structures, the rules given in §13 of the main text, can be applied when using FRP reinforcement with some restrictions. Most of them are related with the fact that prestressing with FRP is not covered by Annex R.

10.
CONCLUSIONS

The purpose of this paper is to summarize the content of the informative Annex R, developed by CEN/TC250/SC2/WG1/TG1 and Project Team 3, in the new Eurocode 2 [14] provisions for the design with embedded FRP reinforcement. In addition, there is also a background document [20] that provides additional explanations and supporting information about Annex R.

The provisions of Annex R for verifying the ultimate limit states are an adaptation of the main text to the particular case of FRP reinforcement.

The main differences between FRP and conventional steel reinforcement are that FRPs are anisotropic linear elastic up to failure and have lower modulus of elasticity. The design tensile strength of the FRP rebar is defined as the long-term tensile strength affected by a safety factor. This long-term tensile strength considers the decrease in the short-term tensile strength due to time, temperature and environmental influence.

For the ULS of shear, for elements with transverse reinforcement, an initial formulation which was a modificacions of the strut-and-tie model was firstly developed. However, results were very conservative when applying the formulation to a database compiled by Kurth [34]. To avoid an uneconomical design, an additive approach, derived by Kurth et al. [34,35] was included, in Annex R.

Due to the lower modulus of elasticity, serviceability limit states may often govern the design of FRP RC members. Limitation of cracking is mainly due appearance conditions, since FRP rebars present low susceptibility to corrosion.

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