Strengthening of RC Buildings with Composites

Giorgio Monti and Floriana Petrone

1 Introduction

Still today the number of existing structures designed without considering seismic action or poorly designed/constructed is significantly larger than the number of structures conceived and built to resist seismic loads. For this reason, the research in the field of earthquake engineering of the last two decades has directed its focus towards the development of effective and efficient strengthening techniques of existing buildings.

As a first consideration, there is a substantial difference between methods for designing new buildings and the development of approaches for retrofitting existing structures. If on one hand well-established procedures are available for designing new structures according to the capacity design principles, on the other hand no unified or official methods for providing the existing structures with a sough ductility level are available. In addition, any retrofitting process is based on the assessment of the current capability of the structural system to dissipate energy. This requires a detailed analysis of the structure, aimed at identifying the actual material properties and geometry as well as deficiencies/mistakes and then at determining the optimal way to fix them. Therefore, the retrofitting process should

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be interpreted as an original process, specific to each structure: as such, it does not (and cannot) consist in checking the compliance of the structure with Code provisions, but in a more comprehensive performance assessment, before and after the strengthening.

In this framework, FRP composites represent one of the technologies employed to locally strengthen structural elements (beams, columns, walls and joints). The first studies in this field date back to the beginning of the 1990s and still researchers strive at finding new solutions for enhancing the safety of existing constructions, seen as valid alternatives to more usual techniques, such as, mortar injections, concrete jacketing, steel tying and plating, base isolation and integrative (dissipative or not) bracings.

2 Materials

Continuous fibre-reinforced materials with polymeric matrix (commonly known as FRP) are composite, heterogeneous, and anisotropic materials with a (prevalent linear) elastic-brittle behaviour, widely used for strengthening civil structures. Their main advantages can be summarized in: light weight, high strength and corrosion-resistance. Composites for structural strengthening are available in several geometries, from laminates used for structural members with regular surface to bi-directional fabrics easily adaptable to non-regular shapes.

This chapter gives an overview on composite materials, with an in-depth analysis of the main constituents (fibre, matrix, and adhesive), and their main physical and mechanical properties along with a documented reference to the principal design equations for flexural, shear, torsion and axial strengthening of RC members. A full understanding of pros and cons is necessary to optimize the use of FRP and mitigate their disadvantages; this is of particular relevance to ensure durability of FRP strengthening applications where traditional materials, such as concrete and masonry, are paired with high technology materials.

2.1 Characteristics of Composites

In general composite materials are made of two or more basic-components (phases) of different nature and "macroscopically" distinguishable. At least two of the phases have different physical and mechanical properties, so to provide FRP composites with features different from those of the single component.

FRPs are made of (i) organic polymeric matrix and (ii) reinforcing fibres, whose main characteristics are summarized in Table 1. The matrix can be considered an isotropic material, whereas the reinforcing phase, with the exception of glass fibres, an anisotropic material. As shown in the table, Young's modulus and tensile strength of carbon fibers can be significantly higher than those of typical construction materials, making FRPs more effective from a structural point of view, especially when the weight of the structure becomes a critical issue.

Table 1 Comparison between properties of fibres, resin, and steel (typical values)

	Young's modulus	Tensile strength	Strain at failure	Coef. of thermal exp.	Density
	E	σ_r	ε_r	α	ρ
	[GPa]	[MPa]	[%]	[10^{-6} °C^{-1}]	[g/cm^3]
E-glass	70–80	2000–3500	3.5–4.5	5–5.4	2.5–2.6
S-glass	85–90	3500–4800	4.5–5.5	1.6–2.9	2.46–2.49
Carbon (high modulus)	390–760	2400–3400	0.5–0.8	−1.45	1.85–1.9
Carbon (high strength)	240–280	4100–5100	1.6–1.73	−0.6 to −0.9	1.75
Aramid	62–180	3600–3800	1.9–5.5	−2	1.44–1.47
Polymeric matrix	2.7–3.6	40–82	1.4–5.2	30–54	1.10–1.25
Steel	206	250–400 (yield)	20–30	10.4	7.8
		350–600 (failure)			

In general, FRP composites can be synthetically described by the following properties:

- Geometry: shape and thickness.
- Fibre orientation with respect to the symmetry axes of the material.
- Fibre concentration (volume fraction).

In most cases, composites are non-homogeneous and anisotropic materials.

Fibre-reinforced composites can be conveniently divided into two categories:

- Fabrics
- Laminates

Fabrics are usually single- or multi-layer strips/sheets, very flexible in bending and few tenths of a millimetre thick. They usually come in rolls as dry materials to be later glued to the elements. Laminates are stiff in bending and few millimetres thick, made of several layers already glued. They usually come in long and narrow plates.

Structural failure of FRP composites is often due to lack of bond between matrix and fibres. Therefore, the FRP material manufacturer or suppliers should take special care in choosing the most appropriate component to ensure bond.

2.2 Fibres and Matrices Used in Composites

The most common fibres used in composites are glass, carbon, and aramid. Their unique mono-dimensional geometry provides FRP laminates with stiffness and strength higher than those of three-dimensional FRP. This is due to the density of defects, which is lower in mono-dimensional configurations than in three-dimensional ones.

As for the matrices, thermoset resins are the most commonly used in the production of FRP materials. They are usually available in a partially polymerized state with fluid or pasty consistency at room temperature. When mixed with a proper reagent, they polymerize to become a solid vitreous material. The reaction can be accelerated by adjusting the temperature. Thermoset resins have several advantages, including low viscosity that allows for a relative easy fibre impregnation, good adhesive properties, room temperature polymerization characteristics, good resistance to chemical agents, and absence of melting temperature. The main disadvantages are: limited range of operating temperatures, with the upper bound limit given by the glass transition temperature, brittle behaviour, and sensitivity to moisture during field applications. The most common thermosetting resins for civil engineering are the epoxy, polyester and vinylester resins. Considering that the material is mixed directly at the construction site and achieves its final structural characteristics through a chemical reaction, it should always be handled by specialized personnel.

2.3 FRP Strengthening Systems

FRP systems suitable for external strengthening of structures may be classified as follows:

- Pre-cured systems: manufactured in various shapes by pultrusion or lamination. Pre-cured systems are directly bonded to the structural member.
- Wet lay-up systems: manufactured with fibres lying in one or more directions, e.g. FRP sheets or fabrics, and impregnated with resin at the construction site.
- Prepreg (pre-impregnated) systems: manufactured with unidirectional or multidirectional fibre sheets or fabrics pre-impregnated at the manufacturing plant with partially polymerized resin. They may be bonded to the member to be strengthened with (or without) the use of additional resins.

2.4 Mechanical Properties of FRP Strengthening Systems

In FRP composites, fibres provide both capacity and stiffness, whereas the matrix ensures the distribution of the load among the fibres and protects the same fibres from corrosion/deterioration. Most FRPs are made of fibres with high strength and stiffness, and fail at strains lower than those of the matrix.

Figure 1 shows the stress-strain relationship of fibre, matrix and FRP. The resulting FRP composite has a stiffness lower than that of fibres and fails at the same strain of the fibres, $\varepsilon_{fib,\max}$.

Table 2 summarizes the mechanical properties of a pre-cured laminate compared to the average values of the corresponding fibres.

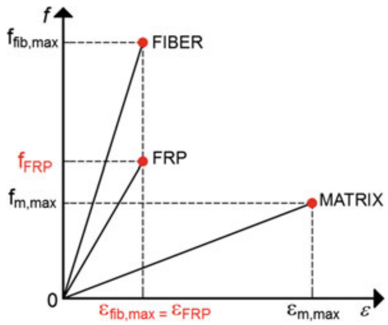


Fig. 1 Stress-strain relationship of fibres, matrix and FRP

Table 2 Comparison between mechanical properties of a pre-cured laminate and fibres

Pre-cured systems	Modulus of elasticity [GPa]		Ultimate strength [MPa]		Ultimate strain [%]	
	FRP	Fibre	FRP	Fibre	FRP	Fibre
	E_f	E_{fib}	f_f	f_{fib}	ε_{fu}	$\varepsilon_{fib,u}$
CFRP (low modulus)	160	210–30	2800	3500–4800	1.6	1.4–2.0
CFRP (high modulus)	300	350–500	1500	2500–3100	0.5	0.4–0.9

The values of the Young's modulus, E_f, and the strength, f_f, of FRP at failure are lower than those of the fibre itself, whereas the ultimate tensile strain is essentially the same, since the failure of the fibre determines FRP's failure.

3 Basis of Design for FRP Strengthening

The design of any structural strengthening through FRPs must meet the requirements of serviceability, durability and resistance to ordinary loads and exceptional actions. For example, in case of fire, the strengthening has to be designed to resist for the prescribed exposure time.

The design working life of the strengthened structure is the same as that of new structures, meaning that the design actions are those of the current Codes for new constructions.

Safety verifications are performed for Serviceability Limit State (SLS) and Ultimate Limit States (ULS), following the format of the partial safety factor method, established in EN 1990 [3], where the design properties of materials and products are derived from the characteristic values, divided by the appropriate partial safety factor.

A fundamental aspect in assessing the safety of existing structures is the treatment of all uncertainties, mainly related to (1) materials mechanical properties, (2) geometry of the structure, and (3) evaluation of possible materials deterioration. As per EN 1990 [3], the design properties X_d of the materials used in the structure are calculated as function of the number of tests performed to acquire information on them:

$$X_d = \frac{\eta}{\gamma_m} m_X (1 - k_n V_X) \tag{1}$$

where η (<1) is a conversion factor, accounting for special design conditions, γ_m is the material partial safety factor, m_X is the mean value of the property X resulting from n experimental tests, k_n is a factor that accounts for the epistemic uncertainty of each X property depending on n, and V_X is the coefficient of variation (CoV), usually available for most common materials (e.g. 0.10 for steel, to 0.20 for concrete and to 0.30 for masonry and timber). For Ultimate Limit States verifications, the partial factor γ_m of FRP takes on different values depending on the failure mechanism: in case of FRP rupture, $\gamma_m = 1.0$; whereas in case of FRP debonding γ_m ranges between 1.2 and 1.5 in consideration of the possibility that debonding can actually occur, based on tests performed by the FRP supplier and as-built conditions.

Equation (1) deserves some additional comments regarding the determination of the parameters. Concrete mechanical properties, usually affected by the highest uncertainties, may be estimated using non-destructive tests (for example by measuring the ultrasonic pulse speed in conjunction with rebound tests). The reliability of these measurements largely depends on the correlation between the indirect quantity actually measured (speed, rebound, etc.) and the mechanical value sought (strength, modulus, etc.). Additional information gained by comparison to destructive tests carried out on the same structure can be used to better calibrate such correlation, thus reducing the risk of systematic errors; however, the number of destructive tests should be kept low, both for economic reasons and to limit any damage to the structure.

A similar situation arises when determining quantity and arrangement of reinforcement. In existing structures, built in the absence of rules imposing detailed working drawings, it is very unlikely that any direct information be available on the geometry as well as on the distribution of the reinforcement. In such cases, the amount of reinforcement can be estimated on the basis of a simulated project developed according to the Code in force at the time. It is acknowledged that such estimate is highly uncertain and needs to be validated by means of in-situ investigations, which can be either direct (clear exposure of the steel reinforcements by elimination of cover concrete and any other material covering them) or indirect (for example by magnetic inductance measurements using pacometer). Since direct measurements are partially destructive and imply damage to the structure, the considerations reported above about the limited number of tests for assessing the material properties hold also for the number of test needed to characterize the reinforcement.

Once the material properties of the existing structure are assessed and the materials adopted for strengthening are selected, the design capacity of the strengthened structure is given by:

$$R_d = \frac{1}{\gamma_{Rd}} R\{X_{d,i}; a_{d,i}\} \tag{2}$$

where $R\{\cdot\}$ is the function describing the relevant mechanical model considered (e.g., flexure, shear, confinement, etc.) and γ_{Rd} is the partial factor acounting for uncertainties in the above capacity model, set equal to 1 for flexure, to 1.2 for shear and to 1.1 for confinement. The arguments of the function are sets of mechanical and geometrical properties, $X_{d,i}$ and $a_{d,i}$, respectively, representing the design value of the i-th quantity (for geometrical properties, nominal values are usually adopted).

Another aspect is related to the safety assessment in case of exceptional actions, as fire. If the strengthening is designed for a predefined fire exposure time (i.e. $E_d \neq 0$, where E_d is the design value of the indirect thermal action due to fire), the service actions of the *frequent combination*, instead of *quasi-permanent combination,* have to be considered. However, the capacity of the structural elements, appropriately reduced to account for the fire exposure time, should be computed with the partial factors relevant to the exceptional situation.

4 Reinforced Concrete Structures

4.1 Anchorage

When strengthening RC members with FRP composites, the role of bond between concrete and FRP is of great relevance due to the brittleness of the loss of adhesion, the so-called "debonding" failure mechanism. According to the capacity design criterion, such failure should not precede flexural or shear failure of the strengthened member.

In general, debonding may involve different components of the strengthened structure and may take place: (1) within laminates and sheets applied to concrete for flexural/shear strengthening, (2) within the adhesive, (3) between concrete and adhesive (4) in the concrete itself, or (5) within the FRP reinforcement with different fibre inclination angles (e.g., at the interface between two adjacent layers bonded each other). When proper installation is performed, since the adhesive strength is typically much higher than the concrete tensile strength, debonding most likely takes place within the concrete itself.

Debonding failure modes for laminates or sheets used for flexural strengthening can be classified in the following four categories, schematically represented in Fig. 2:

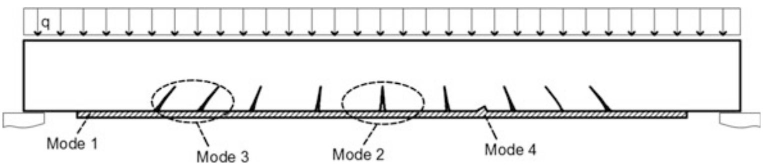


Fig. 2 FRP flexural strengthening: debonding failure modes

Mode 1: Laminate/sheet end debonding
Mode 2: Intermediate debonding, caused by flexural cracks
Mode 3: Debonding caused by diagonal shear cracks
Mode 4: Debonding caused by irregularities and roughness of concrete surface

In the following, reference is made to Modes 1 and 2 only, as they are the most frequent in ordinary design situations.

Before any flexural or shear strengthening design takes place, the evaluation of the maximum force that can be transferred from concrete to FRP and the calculation of shear and normal stresses at the concrete-FRP interface are required. The former is necessary when designing for ULS, and the latter when designing for SLS.

With reference to a typical bond test, as represented in Fig. 3, the ultimate value of the force transferred to the FRP system prior to debonding depends on the length, l_b, of the bonded area. The optimal anchorage length, l_{ed}, is defined as the length corresponding to the maximum force F that can be transferred, meaning that even if this length was increased, there would be no increase in the transferred force.

The optimal anchorage length (in mm) is given as:

$$l_{ed} = 0.10\sqrt{\frac{\pi^2 E_f t_f}{2\Gamma_{Fd}}} \geq 200 \text{ mm} \tag{3}$$

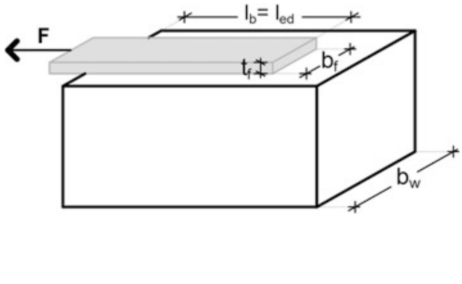
where E_f and t_f are Young's modulus and thickness of the FRP, respectively, and:

$$\Gamma_{Fd} = \frac{k_b k_G}{CF}\sqrt{f_{cm} f_{ctm}} \tag{4}$$

is the design value of the specific fracture energy, expressed as function of f_{cm} and f_{ctm}, which are the mean values of the concrete compressive and tensile strength, respectively, CF, which is an appropriate confidence factor that depends on the attained knowledge level of the existing structure, k_G, which is equal to 0.023 mm for preformed composites and to 0.037 for on-site impregnated composites, and k_b given as:

$$k_b = \sqrt{\frac{2 - \left(b_f/b_w\right)}{1 + \left(b_f/b_w\right)}} \geq 1 \tag{5}$$

Fig. 3 Maximum force transferred between FRP and concrete



However, if the ratio between the FRP and concrete width, $b_f/b_w < 0.25$, see Fig. 2, then $k_b = 1.18$.

The design debonding strength for mode 1 is:

$$f_{fdd,1} = \frac{1}{\gamma_{fd}} \sqrt{\frac{2E_f \Gamma_{Fd}}{t_f}} \tag{6}$$

where γ_{fd} is the partial factor for debonding, ranging between 1.2 and 1.5.

The design debonding strength for mode 2 is:

$$f_{fdd,2} = 1.25 f_{fdd} \tag{7}$$

where in the computation of f_{fdd}, $k_G = 0.10$ mm should be assumed.

4.2 *Flexural Strengthening*

Flexural strengthening is necessary for structural members subjected to a bending moment that exceeds the flexural capacity. Only the case of uniaxial bending (i.e. when the moment axis coincides with a principal axis of inertia of the cross-section) is addressed here.

Flexural strengthening with FRP materials may be carried out by applying one or more laminates/ sheets to the tension side of the element.

The flexural capacity is attained when either the concrete compressive strain or the FRP tensile strain reaches its ultimate value, that is $\varepsilon_{fd} = \min\,(\eta_a \varepsilon_{fu}/\gamma_f,\, f_{ffd}/E_f)$, where the first value corresponds to concrete crushing and the second to FRP debonding, as previously defined. The flexural capacity is then expressed as:

$$M_u = \psi\, b\, x f_{cd}(d - \lambda x) + A_{s2}\sigma_{s2}(d - d_2) + A_f \sigma_f d_1 \tag{8}$$

where the neutral axis position x is found by solving:

$$0 = \psi\, b\, x f_{cd} + A_{s2}\sigma_{s2} - A_{s1} f_{yd} - A_f \sigma_f \tag{9}$$

where ψ and λ are non-dimensional coefficients representing the magnitude and the position of the compressive concrete resultant, respectively.

However, the capacity after strengthening cannot be greater than twice the initial capacity. Moreover, according to the capacity design approach, flexural strenghtening should be designed to avoid the activation of shear failure mechanisms.

Because a member strengthened with FRP is generally loaded at the time of FRP application, the existing strain state in the structure should be taken into account.

4.3 Shear and Torsion Strengthening

Shear strengthening is necessary when the shear demand is greater than the member shear capacity, evaluated considering the contributions of both concrete and steel transverse reinforcement. It may also be necessary after designing a flexural strengthening, in order to re-establish the strength hierarchy between bending and shear failure mechanisms.

Shear strengthening shall be verified at ULS only. Shear strengthening is usually realized by applying one or more layers of FRP, externally bonded to the surface of the structural member to strengthen. External FRP reinforcement can be applied in a discontinuous fashion, with gaps between following strips, or continuously, with strips next to each other.

Figure 4 shows two allowed FRP strengthening configurations: U-wrapped, and completely wrapped beams.

For U-wrap strengthening of rectangular or T-sections, delamination of the end portions of FRP reinforcement can be avoided by using laminates/sheets and/or bars installed in the direction of the member's longitudinal axis. In such case, the behaviour of U-wrap strengthening can be considered equivalent to that of a completely wrapped member, provided that the effectiveness offered by these technological solutions is demonstrated by the applicator.

The design shear strength of the strengthened element is based on the variable angle truss model and is expressed as:

$$V_{Rd} = \min\left\{V_{Rd,s} + V_{Rd,f}, V_{Rd,c}\right\} \tag{10}$$

where $V_{Rd,s}$ and $V_{Rd,f}$ are the contributions of transverse steel and FRP to shear-tension capacity, respectively, and $V_{Rd,c}$ is the contribution of concrete to shear-compression capacity. A method for evaluating the actual contribution of each component to the shear strength can be found in [4].

The FRP contribution to the overall strength is based on the selected strengthening configuration. For U-jacketing and wrapping:

$$V_{Rd,f} = \frac{1}{\gamma_{Rd}} \cdot 0.9\,d \cdot f_{fed} \cdot 2\,t_f \cdot (\cot\theta + \cot\beta) \cdot \frac{w_f}{p_f} \tag{11}$$

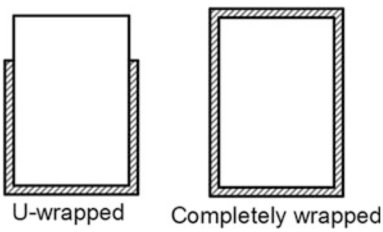


Fig. 4 Two configurations for shear strengthening

with d = cross-section effective depth, t_{f} = thickness of the FRP strip/sheet with angle β, θ = crack angle, p_{f}, w_{f} = FRP strip spacing and width, respectively, measured orthogonally to the fibre direction β and f_{fed} = the so-called "effective debonding strength". For the case of U-jacketing and wrapping, respectively, f_{fed} is given by:

$$f_{fed} = f_{fdd} \cdot \left[1 - \frac{1}{3}\frac{l_e \sin\beta}{\min\{0.9\,d, h_w\}}\right] \tag{12}$$

$$f_{fed} = f_{fdd} \cdot \left[1 - \frac{1}{6}\frac{l_e \sin\beta}{\min\{0.9\,d, h_w\}}\right] + \frac{1}{2}(\phi_R \cdot f_{fd} - f_{fdd}) \cdot \left[1 - \frac{l_e \sin\beta}{\min\{0.9\,d, h_w\}}\right] \tag{13}$$

where f_{fd} is the FRP design strength, h_{w} is the beam web depth and:

$$\phi_{\mathrm{R}} = 0.2 + 1.6\,\frac{r_{\mathrm{c}}}{b_{\mathrm{w}}} \quad \text{with} \quad 0 \leq \frac{r_{\mathrm{c}}}{b_{\mathrm{w}}} \leq 0.5 \tag{14}$$

is a coefficient that depends on the ratio of the rounding radius r_c to the beam web width b_w.

With regard to the strengthening in torsion, it is achieved through the application of wrapping strips/sheets at an angle of 90° to the element axis. The design torsional strength of the strengthened element is given as:

$$T_{Rd} = \min\{T_{Rd,s} + T_{Rd,f}, T_{Rd,\max}\} \tag{15}$$

where $T_{Rd,s}$ and $T_{Rd,f}$ are the transverse steel and FRP contribution, respectively, and $T_{Rd,max}$ is the torque producing collapse in the compressed diagonal concrete strut. The FRP contribution to the torsional strength is given as:

$$T_{Rd,f} = \frac{1}{\gamma_{Rd}} \cdot 2 f_{fed} \cdot t_f \cdot b \cdot h \cdot \frac{w_f}{p_f} \cdot \cot\theta \tag{16}$$

where f_{fed} is given by Eq. (12) or (13).

4.4 Confinement

Appropriate confinement of RC members may improve their structural performance, by increasing the ultimate capacity and strain of structural members subjected to axial -or slightly eccentric—loads.

Ductility and capacity under combined bending and axial force, when FRP reinforcements are placed with the fibres lying along the longitudinal axis of the member, should be verified.

Confinement of RC members can be realized with FRP sheets arranged along the member perimeter as either continuous or discontinuous external wrapping.

The increase of axial capacity and ultimate strain of FRP-confined concrete depends on the applied confinement pressure, which is function of the member cross-section and FRP stiffness.

FRP-confined members (FRP is linear-elastic up to failure), unlike steel confined members (steel has an elastic-plastic behaviour), exert a lateral pressure that increases with the transversal expansion of the confined members.

In case of elements with circular cross-section of diameter D, the confined/unconfined concrete strength ratio is:

$$\frac{f_{ccd}}{f_{cd}} = 1 + 2.6\left(\frac{f_{l,eff}}{f_{cd}}\right)^{2/3} \tag{17}$$

while the ultimate concrete strain is:

$$\varepsilon_{ccu} = 0.0035 + 0.015\sqrt{\frac{f_{l,eff}}{f_{cd}}} \tag{18}$$

where both depend on the confinement pressure exerted by the FRP sheet, given as:

$$f_{l,eff} = k_{eff} \cdot f_l \quad \text{with} \;\; f_l \; = \; \frac{1}{2}\,\rho_f\,E_f\,\varepsilon_{fd,rid} \tag{19}$$

where k_{eff} is an efficiency factor (≤ 1), E_f is, again, the FRP modulus of elasticity, $\varepsilon_{fd,rid}$ is the FRP reduced design strain, defined in the following, and ρ_f is the geometric strengthening ratio, which is function of the cross-section shape (circular or rectangular), that is:

$$\begin{aligned} \rho_f &= \frac{4 t_f b_f}{D \cdot p_f} && \text{circular sections} \\ \rho_f &= \frac{2 \cdot t_f \cdot (b+d) \cdot b_f}{b \cdot d \cdot p_f} && \text{rectangular sections} \end{aligned} \tag{20}$$

being t_f and b_f the thickness and the width of the generic FRP strip, p_f the centre-to-centre distance between strips, D the diameter of the circular cross-section, and b and d the dimensions of the rectangular cross-section.

The efficiency factor is given as:

$$k_{eff} = k_H \cdot k_V \cdot k_\alpha \tag{21}$$

where k_H is the horizontal efficiency factor, equal to 1.0 for circular sections and to:

$$k_H = 1 - \frac{b'^2 + d'^2}{3 \cdot A_g} \tag{22}$$

for rectangular sections, with $b' = b - 2r_c$, $d' = d - 2r_c$ and A_g = area of the cross-section; and k_V is the vertical efficiency factor, calculated as:

$$k_V = \left(1 - \frac{p'_f}{2\,d_{\min}}\right)^2 \tag{23}$$

where p'_f is the edge-to-edge distance between adjacent strips and $d_{\min}$ is the minimum transverse dimension of the element; when the fibres are wrapped at an angle α_f with respect to the element axis, the angle efficiency factor, k_α, is:

$$k_\alpha = \frac{1}{1 + (\tan\alpha_f)^2} \tag{24}$$

Finally, the reduced design strain is:

$$\varepsilon_{fd,rid} = \min\{\eta_a \varepsilon_{fk}/\gamma_f;\ 0.004\} \tag{25}$$

where $\varepsilon_{\mathrm{fk}}$ is the FRP characteristic strain, and η_a and γ_f are the environment conversion factor and the partial factor of the FRP strengthening, respectively.

5 FRP Strengthening in Seismic Zones

Composite materials can be used effectively to seismically retrofit reinforced concrete structures. The objective is that of strengthening buildings that do not meet the safety requirements defined by the current seismic Codes under the design seismic action, with respect to one or more limit states.

Once a preliminary seismic assessment is performed on the existing structure, the strengthening intervention is designed based on its outcomes. The entire process goes through the following steps: (a) identification of safety requirements, (b) definition of protection levels (which yield the intensity of the seismic action), (c) choice of analysis methods, (d) choice of verification criteria, (e) assessment of the seismic safety, (f) definition of the material properties to use in the safety verifications.

Regarding the criteria for selecting the FRP strengthening method, it is widely recognized that stiffness irregularities cannot be solved by applying FRPs. In fact an intervention performed with FRP is classified as a selective technique, since strength irregularities can be adjusted by strengthening a selected number of elements. However, attention should be paid to ensure that the global ductility is not reduced.

The design of a strengthening intervention with FRP should include the following activities: (a) justification of the intervention type, (b) selection of techniques and/or materials, (c) preliminary design of the strengthening intervention, (d) structural analysis of the upgraded structure.

As mentioned above, from the seismic standpoint, FRP strengthening is regarded as a selective intervention technique, aiming at: (a) increasing the flexural capacity of deficient members through the application of composites with the fibres placed parallel to the element axis, (b) increasing the shear strength through the application of composites with the fibres placed transversely to the element axis, (c) increasing the ductility (or the chord rotation capacity) of critical zones of beams and columns through FRP wrapping (confinement), (d) improving the efficiency of lap splice zones, through FRP wrapping, (e) preventing buckling of longitudinal rebars under compression through FRP wrapping, (f) increasing the tensile strength of the panels of partially confined beam-column joints through the application of composites with the fibres placed along the principal tensile stress direction.

In general, the inspiring principles of the intervention strategies should be the followings: (a) all potential brittle failure mechanisms should be avoided, (b) all potential “soft story” collapse mechanisms should be eliminated, and (c) the global deformation capacity of the structure should be enhanced, either by (c1) increasing the ductility of the potential plastic hinge zones without changing their position, or, (c2) relocating the potential plastic hinge zones by flexure-strengthening the columns, with the aim of transforming the frame structure into a high dissipation mechanism with strong columns and weak beams.

For principle (a), as well-known, brittle failure mechanisms such as shear in beams and structural joints, lap splicing, and bar buckling should be avoided. For shear, the same criteria apply as for the non-seismic case, with the exception that side bonding is not allowed and FRP strips/sheets should only be applied orthogonally to the element axis. When avoiding potential brittle failure mechanisms, the relative strengthening modalities are quite straightforward. The most common case is that of potential shear failure either in beams or structural joints: in this case a strengthening of the regions of the structural member where shear mechanisms take place should be designed. More peculiar cases are those of longitudinal bars lap splices and buckling: in the former case, due to either bond degradation in splices or insufficient splice length, the relevant regions of potential plastic hinge formation should be adequately confined through an FRP wrapping; in the latter case, the strengthening intervention should consist in confining the potential plastic hinge zones where the existing transverse reinforcement cannot prevent the bars post-elastic buckling.

For principle (b), specific consideration should be given to potential “soft story” collapse mechanism, which can occur in the absence of walls, due to the simultaneous formation of plastic hinges at top and bottom of all columns at a certain story. In such cases, the strengthening intervention should aim at increasing the flexural capacity of these zones, with the objective of inhibiting the hinges formation.

For principle (c), when all possible brittle and soft story mechanisms are prevented, one could ascertain the extent to which the structure could exploit its ductility. This can be done, for example, through a nonlinear pushover analysis, included in the most modern seismic Codes. Usually, one is requested to check if the structure can actually exhibit a given ductility: this is expressed either by a pre-selected behaviour factor or by an attained target displacement obtained from the displacement spectrum. Such analysis allows to identify all those elements whose local collapse prevents the structure from exploiting its global ductility and from reaching the target displacement.

At this stage, one could face two different situations: (c1) the number of local collapses is not significant, or (c2) the number of local collapses is significant.

In the former case (c1), it is necessary to increase the deformation capacity of only those elements that collapse before the global target displacement is attained. The deformation capacity of beams and columns can be measured by the chord rotation q, that is, the rotation of the chord connecting the element end section with the contra-flexure section (shear span). Generally, the plastic deformation capacity is controlled by the compressive behaviour of concrete. An intervention of FRP-confinement on such elements (usually columns) increases the ultimate compressive strain of concrete, thus determining a ductility increase of the element.

In the latter case (c2), local collapses are so numerous that a different strategy should be pursued: the request of ductility should be spread over a larger number of elements. This can be achieved by relocating all potential plastic hinges in the columns to the framing beams, according to the capacity design criterion, which implies the elimination of all potential plastic hinges in columns. In “weak column-strong beam” situations, typical of frame structures designed for gravity loads only, the columns sections are under-designed both in size and reinforcement. In such cases, it is necessary to increase their flexural strength with the objective of changing the structure into a “strong column-weak beam” situation. It should be noted that, pursuing this strategy implies an increase of shear demand on columns due to the flexural capacity increase. It is therefore necessary to perform the required shear verifications and to increase the shear strength in order to avoid brittle failure modes.

As a matter of fact, the evaluation of the deformation capacity of FRP-strengthened existing RC elements under cyclic loads, has been a primary research for the past two decades; as a result, a relatively large number of analytical models describing the “axial load - bending - shear” cyclic response of RC structural members with FRP have been proposed, together with empirical formulas derived from experimental observations. However, such large number of available models and related research work denote also the difficulties that exist in finding a unified and undisputed assessing/design approach, which should include both a mechanics-based view of all FRP-strengthening techniques and a reliability-based framework. This stems from the relatively limited accuracy shown by some of the proposed models, as well as from the difficulty in extrapolating results from a limited number of experimental tests that, by their nature, cannot cover the full range of peculiarities of the response of FRP-strengthened RC elements under cyclic actions.

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Flexure and Shear Strengthening

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