NATIONAL RESEARCH COUNCIL

ADVISORY COMMITTEE
ON TECHNICAL RECOMMENDATIONS FOR CONSTRUCTION

Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Existing Structures

Materials, RC and PC structures, masonry structures

CNR-DT 200/2004

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CONTENTS

1 FOREWORD

It is a common feeling, among those involved in research and design activities in the field of strengthening with fiber-reinforced composites, that Italy is getting a worldwide reputation, both for the value of its contribution in improving the knowledge in this field as well as for the presence of a peculiar and important building heritage. This includes those of historical and architectural relevance as well as more recent masonry, reinforced concrete, prestressed concrete, and steel structures. Most of the latter structures are now over 30 years old and in need of urgent structural remedial works.

The main international initiatives for the identification of design guidelines to address these needs are well known. It is worth mentioning the Japanese (JSCE – 1997), the American (ACI 440 – 2000), as well as the European guidelines (FIP-CEB – 2001). For the sake of completeness, the study report entitled “Non-metallic reinforcements in RC structures,” approved by the Italian National Research Council (CNR) in January 1999 will also be included. All the aforementioned documents deal with structures made out of reinforced concrete.

The purpose of this guideline is to provide, within the framework of the Italian regulations, a document for the design and construction of externally bonded FRP systems for strengthening existing structures. A guideline, by its nature, is not a binding regulation, but merely represents an aid for practitioners interested in the field of composites. Nevertheless, the responsibility of the operated choices remains with the designer.

The document deals with the following topics:

- Materials
- Basic concepts on FRP strengthening
- Strengthening of reinforced and prestressed concrete structures
- Strengthening of masonry structures

Specific guidelines for the strengthening of reinforced and prestressed concrete structures as well as masonry structures for construction subjected to earthquakes according to the most recent national and international design codes are provided.

The first topic includes a summary of the several advantages and some disadvantages of FRP materials. It also includes an Appendix (Appendix A) where notions on the mechanical characterization of composite materials are presented. The peculiar differences between FRPs as compared to traditional materials (such as their anisotropic behaviour) as well as emphasis to their constitutive laws are highlighted.

The remaining topics are approached according to the usual style of technical documents published by CNR. The approach of the Eurocodes is adopted; statements are divided between *Principles* and *Application Rules*. Each statement is marked by a progressive numbering, with the principles being marked by the label (P). *Principle* statements include the following:

- *General statements and definitions of mechanical-structural nature.*
- *Recognized needs and/or analytical models accepted by the scientific community, whose value is universally deemed to be pre-eminent with respect to possible alternatives, unless otherwise explicitly stated.*

Application Rules are *procedures of widely recognized value, following the Principles and satisfying their needs.*

The document contains four more Appendices:

- Appendix B includes a section on the failure modes due to debonding and a section on the constitutive law for bond between FRP and concrete substrate.
- Appendix C on the design of FRP-reinforced concrete columns under combined bending and axial forces.
- Appendix D on confined concrete.
- Appendix E containing numerical examples on FRP strengthening of reinforced concrete members.

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1.1 PUBLIC HEARING

After its publication, the document n.200/2004 was subject to public hearing between November 2004 and January 2005. Following the public hearing, some modifications and/or integrations have been made to the document including corrections of typos, additions of subjects that had not been dealt with in the original version, and elimination of others deemed to be not relevant.
The updated document has been discussed and approved by the authors during the meetings held on March 2005 at the CNR headquarters in Rome.

This Technical Document has been approved by the “Advisory Committee on Technical Recommendation for Construction” as a draft version on 13/07/2004, and as a final version on 26/04/2005; the latter document includes the modifications derived from the public hearing.
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1.2 SYMBOLS

General notations

$(.)_c$ value of quantity (.) for concrete
$(.)_{cc}$ value of quantity (.) for confined concrete
$(.)_d$ design value of quantity (.)
$(.)_f$ value of quantity (.) for the fiber-reinforced composite
$(.)_k$ characteristic value of quantity (.)
$(.)_{mc}$ value of quantity (.) for confined masonry
$(.)_R$ value of quantity (.) as resistance
$(.)_s$ value of quantity (.) for steel
$(.)_S$ value of quantity (.) as demand

Uppercase Roman letters

A_c area of concrete cross-section, net of steel reinforcement
A_f area of FRP reinforcement
A_{fw} area of FRP shear reinforcement
A_l overall area of longitudinal steel reinforcement
A_{sw} area of one stirrup leg
A_{s1} area of steel reinforcement subjected to tension
A_{s2} area of steel reinforcement subjected to compression
E_c Young's modulus of elasticity of concrete
E_f Young's modulus of elasticity of FRP reinforcement
E_{fib} Young's modulus of elasticity of fiber itself
E_m Young's modulus of elasticity of matrix
E_s Young's modulus of elasticity of steel reinforcement
$F_{max,d}$ design value of the maximum tensile force transferred by FRP reinforcement to the concrete support
F_{pd} design value of the maximum anchorage force transferred by FRP reinforcement bonded on a masonry structure in the presence of a force perpendicular to the bonded surface area
G_a shear modulus of adhesive
G_c shear modulus of concrete
I_o moment of inertia of cracked and un-strengthened reinforced concrete section
I_1 moment of inertia of cracked and FRP-strengthened reinforced concrete section
I_c moment of inertia of transformed section
I_f moment of inertia of FRP reinforcement about its centroidal axis, parallel to the beam neutral axis
M_{Rd} flexural capacity of FRP-strengthened member
M_{Sd} factored moment
M_o bending moment acting before FRP strengthening
M_1 bending moment applied to the RC section due to loads applied after FRP strengthening
$N_{Rcc,d}$ axial capacity of FRP-confined concrete member
$N_{Rmc,d}$ axial capacity of FRP-confined masonry
N_{Sd} factored axial force
P_{fib} weight fraction of fibers
P_m weight fraction of the matrix
T_g glass transition temperature of the resin
T_m melting temperature of the resin
T_{Rd} torsional capacity of FRP-confined concrete member
$T_{Rd,f}$ FRP contribution to the torsional capacity
$T_{Rd,max}$ torsional capacity of the compressed concrete strut

$T_{Rd,s}$ steel contribution to the torsional capacity
T_{Sd} factored torsion
T_x Yarn count in x direction
V_{fib} volumetric fraction of fibers
V_{Rd} shear capacity of FRP-strengthened member
$V_{Rd,ct}$ concrete contribution to the shear capacity
$V_{Rd,max}$ maximum concrete contribution to the shear capacity
$V_{Rd,s}$ steel contribution to the shear capacity
$V_{Rd,f}$ FRP contribution to the shear capacity
$V_{Rd,m}$ masonry contribution to the shear capacity
V_{Sd} factored shear force

Lowercase Roman letters

b_f width of FRP reinforcement
d distance from extreme compression fiber to centroid of tension reinforcement
f_{bd} design bond strength between FRP reinforcement and concrete (or masonry)
f_{bk} characteristic bond strength between FRP reinforcement and concrete (or masonry)
f_c concrete compressive strength (cylindrical)
f_{ccd} design strength of confined concrete
f_{cd} design concrete compressive strength
f_{ck} characteristic concrete compressive strength
f_{ctm} mean value of concrete tensile strength
f_{fd} design strength of FRP reinforcement
f_{fdd} design debonding strength of FRP reinforcement (mode 1)
$f_{fdd,2}$ design debonding strength of FRP reinforcement (mode 2)
f_{fed} effective design strength of FRP shear reinforcement
f_{fk} characteristic strength of FRP reinforcement
f_{fpd} design debonding strength of FRP reinforcement
f_{mk} characteristic compressive strength of masonry
f^h_{mk} characteristic compressive strength of masonry in the horizontal direction
f_{mcd} characteristic compressive strength of FRP-confined masonry
f_{md} design compressive strength of masonry
f^h_{md} design compressive strength of masonry in the horizontal direction
f_{mtd} design tensile strength of masonry
f_{mtk} characteristic tensile strength of masonry
f_{mtm} mean value of the tensile strength of masonry
f_{vd} design shear strength of masonry
f_{vk} characteristic shear strength of masonry
f_y yield strength of longitudinal steel reinforcement
f_{yd} design yield strength of longitudinal steel reinforcement
f_{ywd} design yield strength of transverse steel reinforcement
f_l confining lateral pressure
$f_{l,eff}$ effective confining pressure
h section depth
k_{eff} coefficient of efficiency for confinement
k_H coefficient of efficiency in the horizontal direction
k_V coefficient of efficiency in the vertical direction
k_α coefficient of efficiency related to the angle α of fibers respect to the longitudinal axis of confined member
l_b bond length
l_e optimal bond length

p_b distance between layers of bars in the confinement of masonry columns
p_f spacing of FRP strips or discontinuous FRP U-wraps
s interface slip
s_f interface slip at full debonding
t_f thickness of FRP laminate
w_f width of FRP laminate
x distance from extreme compression fiber to neutral axis

Uppercase Greek letters

Γ_{Fk} characteristic value of specific fracture energy
Γ_{Fd} design value of specific fracture energy

Lowercase Greek letters

α_{fE} safety coefficient for fabric stiffness
α_{ff} safety coefficient for fabric strength
γ_m partial factor for materials
γ_{Rd} partial factor for resistance models
ε_o concrete strain on the tension fiber prior to FRP strengthening
ε_c concrete strain on the compression fiber
ε_{ccu} design ultimate strain of confined concrete
ε_{co} concrete strain on the compression fiber prior to FRP strengthening
ε_{cu} ultimate strain of concrete in compression
ε_f strain of FRP reinforcement
ε_{fd} design strain of FRP reinforcement
$\varepsilon_{fd,rid}$ reduced design strain of FRP reinforcement for confined members
ε_{fk} characteristic rupture strain of FRP reinforcement
ε_{fdd} maximum strain of FRP reinforcement before debonding
ε_{mcu} ultimate compressive strain of confined masonry
ε_{mu} ultimate compressive strain of masonry
ε_{s1} strain of tension steel reinforcement
ε_{s2} strain of compression steel reinforcement
ε_{yd} design yield strain of steel reinforcement
η conversion factor
ν_{fib} Poisson’s ratio of fibers
ν_m Poisson’s ratio of matrix
ρ_{fib} fiber density
ρ_m matrix density
σ_c stress in the concrete
σ_f stress in FRP reinforcement
σ_s stress in tensile steel reinforcement
σ_{Sd} stress normal to masonry face acting on the bonded surface area between FRP reinforcement and masonry
$\tau_{b,e}$ equivalent shear stress at the adhesive-concrete interface
ϕ_u curvature at ultimate
ϕ_y curvature at yielding

2 MATERIALS

2.1 INTRODUCTION

Continuous fiber-reinforced materials with polymeric matrix (FRP) can be considered as composite, heterogeneous, and anisotropic materials with a prevalent linear elastic behavior up to failure. They are widely used for strengthening of civil structures. There are many advantages of using FRPs: lightweight, good mechanical properties, corrosion-resistant, etc. Composites for structural strengthening are available in several geometries from laminates used for strengthening of members with regular surface to bi-directional fabrics easily adaptable to the shape of the member to be strengthened. Composites are also suitable for applications where the aesthetic of the original structures needs to be preserved (buildings of historic or artistic interest) or where strengthening with traditional techniques can not be effectively employed.

There are also examples of applications of composite strengthening with discontinuous fibers and polymeric matrix as well as continuous fibers and inorganic matrix; the latter has been proven to be of particular interest. Such strengthening methodologies, however, will not be discussed in this document because available literature is not sufficient to ensure reliable structural applications.

This chapter reports the basic information on composite materials, their constituents (fiber, matrix, and adhesive), and their physical and mechanical properties. Such information is necessary to know the pros and cons of fiber-reinforced composite materials to make use of their advantages and mitigate, if possible, their disadvantages. This is of particular relevance to ensure durability of FRP strengthening applications where traditional materials such as concrete or masonry are coupled with high technology materials.

The readers familiar with the technological and mechanical properties of fiber-reinforced composite materials may postpone the reading of Sections 2.2 and 2.3 and proceed to Section 2.4.

2.2 CHARACTERISTICS OF COMPOSITES AND THEIR CONSTITUENTS

Composite materials exhibit the following characteristics:

- They are made of two or more materials (phases) of different nature and “macroscopically” distinguishable.
- At least two phases have physical and mechanical properties quite different from each other, such to provide FRP material with different properties than those of its constituents.

Fiber-reinforced composites with polymeric matrix satisfy both of the above characteristics. In fact, they are made out of both organic polymeric matrix and reinforcing fibers, whose main characteristics are summarized in Table 2-1. As it can be seen, carbon fibers may exhibit values of Young’s modulus of elasticity much larger than those of typical construction materials. Therefore, they are more effective from a structural point of view. Potential problems with other materials used as support need to be carefully evaluated by designers and practitioners.

The matrix may be considered as an isotropic material, while the reinforcing phase, with the exception of glass fiber, is an anisotropic material (different properties in different directions). The defining characteristics of FRP materials are as follows:

- Geometry: shape and dimensions.
- Fiber orientation: the orientation with respect to the symmetry axes of the material; when random, the composite characteristics are similar to an isotropic material (“quasi-

isotropic"). In all other cases the composite can be considered as an anisotropic material.
- Fiber concentration: volume fraction, distribution (dispersion).

Therefore, composites are in most cases a non-homogeneous and anisotropic material.

Table 2-1 – Comparison between properties of fibers, resin, and steel (typical values)

	Young's modulus E	Tensile strength σ_r	Strain at failure ε_r	Coefficient of thermal expansion α	Density ρ
	[GPa]	[MPa]	[%]	$[10^{-6}\ ^\circ C^{-1}]$	$[g/cm^3]$
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 - -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) 350 – 600 (failure)	20 – 30	10.4	7.8

To summarize FRP properties, it is convenient to recognize fiber-reinforced composites in two categories, regardless of their production technology:
- Single-layer (lamina)
- Multi-layer (laminates)

Laminates are materials composed of stacked layers (the lamina) whose thickness is usually of some tenths of a millimeter. In the simplest case, fibers are embedded only in the lamina's plane (there are no fibers arranged orthogonally to that plane). The size of laminates is intermediate between those of the fibers and those of engineering structures (Table 2-2). There is also a special class of multi-layer composites, so-called hybrid laminates, where each single lamina is made out of both different fibers (*e.g.*, epoxy matrix composites with carbon and aramid fibers to get a stiff and tough composite) or different materials (*e.g.*, composites with alternate layers of epoxy resin with aramid and aluminium fibers). The main advantage of laminates is represented by the greater freedom of fiber arrangement.

Table 2-2 – Size of fiber composites with polymer matrix.

	representative dimensions										
	pm		nm		µm		mm		m		km
Atom	*	*									
Polymer molecules			*	*							
Biological polymers				*	*						
Crystallites					*	*					
Spheroids						*	*				
Diameter of fibers							*				
Thickness of FRP sheets							*	*	*		
Thickness of FRP laminates									*	*	
Length of laminates										*	*	*
Structures											*	*	*

Due to the anisotropic characteristics of FRP material, their mechanical properties depend on the choice of the reference system. The main axes are usually chosen to be concurring with the symmetry axes of the material (natural axes). The case of a unidirectional FRP material is illustrated in Figure 2-1.

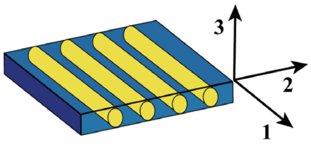


Figure 2-1 – Choice of axes for a unidirectional FRP material.

The ratio between values of the properties of composite materials in different directions is named anisotropic ratio. Some values of the anisotropic ratio related to the main characteristics of interest in unidirectional laminates (E_i: Young modulus of elasticity; G_{ij}: shear modulus; σ_{ri}: failure stress; α_i: coefficient of thermal expansion) are shown in Table 2-3.

Table 2-3 – Anisotropic ratios of fiber-reinforced unidirectional laminates (typical values).

	E_1/E_2	E_1/G_{12}	σ_{r1}/σ_{r2}	α_1/α_2
Silicon carbide/ceramic	1.09	2.35	17.8	0.93
Boron/aluminium	1.71	5.01	11.6	0.30
Silicon carbide/aluminium	1.73	5.02	17.0	0.52
S-Glass/epoxy	2.44	5.06	28.0	0.23
E-Glass/epoxy	4.42	8.76	17.7	0.13
Boron/epoxy	9.27	37.40	24.6	0.20
Carbon/epoxy	13.60	19.10	41.4	-0.07
Aramid/epoxy	15.30	27.80	26.0	-0.07

Composite materials can be stronger and stiffer (carbon FRP) than traditional construction materials. As a result, composites may become very attractive when the weight of the structure becomes an issue. FRP tensile strength and Young's modulus of elasticity can be up to four and two times that of traditional materials, respectively. This means that a composite material structure may weigh nearly half of a traditional construction material structure of equal stiffness.

The nature of the phases of the composite determines the final properties of FRP materials. To obtain a composite with high mechanical strength, using "strong" fibers is not enough. A good adhesion between matrix and fibers used as loading carrying component is also necessary. The adhesion is usually obtained through a third component applied in a very thin layer on the fiber surface that makes them compatible with the organic matrix. Such surface treatment requires the presence of an intermediate phase between the matrix and the fibers, named interface, or interphase (Figure 2-2). The interphase is typically made of a very thin layer (often a single-atom) placed directly on the fiber that is essential for determining the final properties of the material.

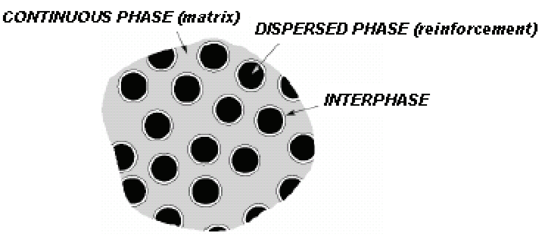


Figure 2-2 – Representation of phases in a FRP composite.

Structural failures of FRP composites are often due to lack of bond between matrix and fibers. Therefore, the FRP material manufacturer should take special care in choosing the most appropriate component to use to promote the bond.

2.2.1 Fibers used in composites

The most common fibers used in composites are glass, carbon, and aramid. Their unique monodimensional geometry, in addition to being particularly suitable for the realization of composites, provides FRP laminates with stiffness and strength higher than those of three-dimensional FRP shapes. This is due to the lower density of defects of mono-dimensional configurations as opposed to that of three-dimensional members.

2.2.1.1 Types of fibers available in the market and their classification

Fibers are made of very thin continuous filaments, and therefore, are quite difficult to be individually manipulated. For this reason, they are commercially available in different shapes (Figure 2-3). A brief description of the most used is summarized as follows:

- Monofilament: basic filament with a diameter of about 10 μm.
- Tow: untwisted bundle of continuous filaments.
- Yarn: assemblage of twisted filaments and fibers formed into a continuous length that is suitable for use in weaving textile materials.
- Roving: a number of yarn or tows collected into a parallel bundle with little or no twist.

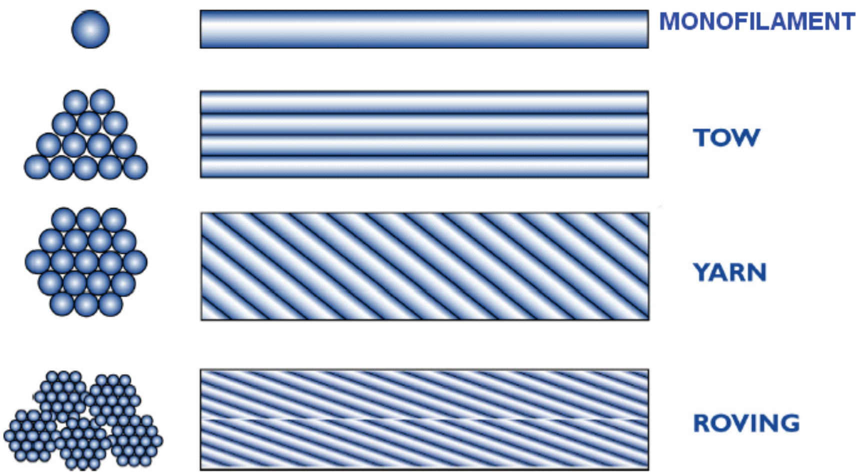


Figure 2-3 – Types of fibers.

By combining a number of tows or yarns together, a tape is obtained, where tows or yarns can be simply arranged side by side or sewed or fastened on a bearing. The classification of fibers is directly taken from that traditionally used for textile fibers. The filaments used to produce yarns are basically characterized by their chemical composition or by their mass per unit length. The unit of linear mass or count (mass per unit length) according to ISO 2974:2000(E) is the TEX, equivalent to 1 g per km of fiber. Another unit of linear mass, now obsolete, is the denier, equivalent to 0.111 TEX.

The technical name of fiberglass follows the rule of ISO 1139:1973(E) and ISO 2078:1993(E) and includes the following members:

- A letter identifying the type of glass used
- A second letter identifying the type of fiber used
  - C (“Continuous”, for filaments)

- D ("Discontinuous", for discontinuous fibers)
- A first number identifying the nominal diameter (in μm) of the filament
- A second number indicating the linear mass of the fiber in TEX
- The direction and value of torsion (Figure 2-4), expressed in rpm (optional)
- The number of wires used to produce the twisted member (optional)
- A manufacturer label containing all the un-coded information necessary for the product characterization (optional)

Negative torsion (S). Positive torsion (Z).

Figure 2-4 – Definition of the two possible directions of torsion.

Examples of labeling are listed in the following:
- EC10 40: continuous filament of E-glass, with a diameter of 10 μm and a linear mass of 40 TEX.
- EC9 34 Z 40: continuous filament of E-glass, with a diameter of 9 μm and a linear mass of 34 TEX, twisted at 40 rpm. The letter Z indicates a torsion defined as positive according to ISO 1139:1973(E) (negative torsion is indicated with the letter S).
- EC9 34 Z 160 x 4 S 150: the letter "x" shows that the material is a wire containing a number of identical filaments. The code preceding the "x" identifies the characteristics of the filaments, while the following number (4) represents the number of filaments and the letter S a negative torsion, accomplished at 150 rpm.
- EC9 x 4 S 150: simplified labelling of the previous filament.

Yarns commonly used for structural composites are referred to as EC5 10 x 2 or SC5 4 x 2, depending whether the material is E-glass or S-glass, respectively. For carbon fibers, yarns are usually classified by the symbol "k," standing for "thousands" [*e.g.*, a 1k yarn is made of 1000 filaments (66.6 Tex), a 3k yarn (200 Tex) has 3000 filaments, and so on]. Typical values are 0.5k, 1k, 3k, 6k, 12k, 18k, 24k, and 48k.

In addition to yarns or rovings, fibers are also commercially available as fabrics. In this case, fibers' dispositions may be such as to provide a quasi-isotropic properties of the fabric. In such materials the main direction is named warp while the orthogonal direction is named weft.

2.2.1.2 Glass fibers

These are fibers commonly used in the naval and industrial fields to produce composites of medium-high performance. Their peculiar characteristic is their high strength. Glass is mainly made of silicon (SiO_2) with a tetrahedral structure (SiO_4). Some aluminium oxides and other metallic ions are then added in various proportions (Table 2-4) to either ease the working operations or modify some properties (*e.g.*, S-glass fibers exhibit a higher tensile strength than E-glass).

Table 2-4 – Typical composition of fiberglass (% in weight).

	E-glass	S-glass
Silicon oxide	54.3	64.20
Aluminium oxide	15.2	24.80
Iron oxide	-	0.21
Calcium oxide	17.2	0.01
Magnesium oxide	4.7	10.27
Sodium oxide	0.6	0.27
Boron oxide	8.0	0.01
Barium oxide	-	0.20
Various	-	0.03

The production technology of fiberglass is essentially based on spinning a batch made of sand, alumina, and limestone. The constituents are dry mixed and brought to melting (about 1260 °C) in a tank. The melted glass is carried directly on platinum bushings and, by gravity, passes through ad hoc holes located on the bottom. The filaments are then grouped to form a strand typically made of 204 filaments. The single filament has an average diameter of 10 μm and is typically covered with a sizing. The yarns are then bundled, in most cases without twisting, in a roving. The typical value of the linear mass for roving to be used in civil engineering applications is larger than 2000 TEX.

Glass fibers are also available as thin sheets, called *mats*. A *mat* may be made of both long continuous or short fibers (*e.g.*, discontinuous fibers with a typical length between 25 and 50 mm), randomly arranged (Figure 2-5) and kept together by a chemical bond. The width of such *mats* is variable between 5 cm and 2 m, their density being roughly 0.5 kg/m^2.

Glass fibers typically have a Young modulus of elasticity (70 GPa for E-glass) lower than carbon or aramid fibers and their abrasion resistance is relatively poor; therefore, caution in their manipulation is required. In addition, they are prone to creep and have a low fatigue strength. To enhance the bond between fibers and matrix, as well as to protect the fibers itself against alkaline agents and moisture, fibers undergo sizing treatments acting as coupling agents. Such treatments are useful to enhance durability and fatigue performance (static and dynamic) of the composite material. FRP composites based on fiberglass are usually denoted as GFRP.

Discontinuous fibers.

Discontinuous fibers *mat*.

Figure 2-5 – Fiberglass *mat*.

2.2.1.3 Carbon fibers

Carbon fibers are used for their high performance and are characterized by high Young modulus of elasticity as well as high strength. They have an intrinsically brittle failure behavior with a relatively low energy absorption; nevertheless, their failure strength are larger compared to glass and aramid fibers. Carbon fibers are less sensitive to creep rupture and fatigue and show a slight reduction of the long-term tensile strength.

The crystalline structure of graphite is hexagonal, with carbon atoms arranged on a basically planar structures, kept together by transverse Van der Waals interaction forces, much weaker than those acting on carbon atoms in the plane (covalent bonds). For such reason, their Young modulus of elasticity and strength are extremely high in the fiber directions and much lower in the transversal direction (anisotropic behavior). The structure of carbon fibers is not as completely crystalline as that of graphite. The term "graphite fibers" is however used in the common language to represent fibers whose carbon content is larger than 99 %. The term "carbon fibers" denotes fibers whose carbon content is between 80 and 95 %. The number of filaments contained in the *tow* may vary from 400 to 160000.

The modern production technology of carbon fibers is essentially based on pyrolysis (*e.g.*, the thermal decomposition in the absence of oxygen of organic substances), named precursors, among which the most frequent are polyacrylonitrile fibers (PAN), and rayon fibers. PAN fibers are first "stabilized," with thermal treatments at 200-240 °C for 24 hrs, so their molecular structure becomes oriented in the direction of the applied load. As a second step, carbonization treatments at 1500 °C in inert atmosphere to remove chemical components other than carbon are performed. The carbonized fibers may then undergo a graphitization treatment in inert atmosphere at 3000 °C, to develop a fully crystalline structure similar to that of graphite. FRP composites based on carbon are usually denoted as CFRP.

2.2.1.4 Aramid fibers

Aramid fibers are organic fibers, made of aromatic polyamides in an extremely oriented form. First introduced in 1971, they are characterized by high toughness. Their Young modulus of elasticity and tensile strength are intermediate between glass and carbon fibers (Figure 2-6 and Figure 2-7). Their compressive strength is typically around 1/8 of their tensile strength. Due to the anisotropy of the fiber structure, compression loads promote a localized yielding of the fibers resulting in fiber instability and formation of kinks. Aramid fibers may degrade after extensive exposure to sunlight, losing up to 50 % of their tensile strength. In addition, they may be sensitive to moisture. Their creep behavior is similar to that of glass fibers, even though their failure strength and fatigue behaviour is higher than GFRP.

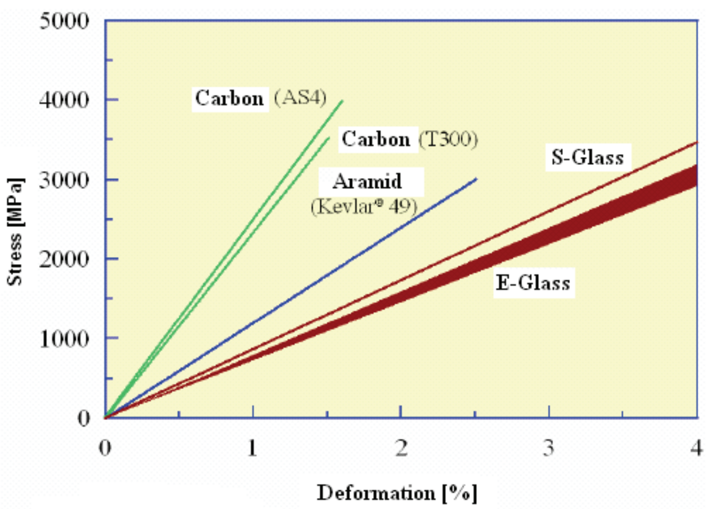


Figure 2-6 – Stress-strain diagram for different reinforcing fibers

The production technology of aramid fibers is based on high-temperature and high-speed extrusion of the polymer in a solution followed by fast cooling and drying. The fibers produced in this way may undergo a hot orientation treatment through winding on fast rotating coils (post-spinning) to improve their mechanical characteristics. Aramid fibers are commercially available as *yarns*, *roving*, or *fabrics*. FRP composites based on aramid fibers are usually denoted as AFRP.

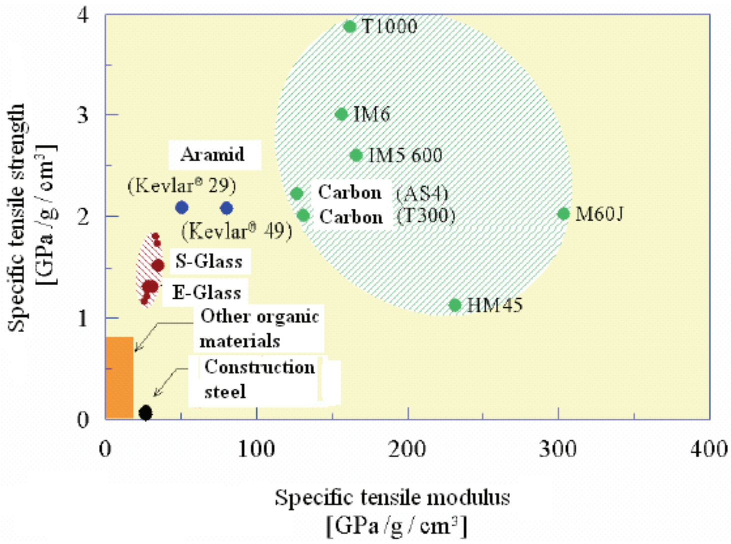


Figure 2-7 – Comparison between FRPs and steel

2.2.1.5 Other types of fibers

Fibers as previously described are the most commonly used for the production of composite materials to be employed for application in the civil engineering field. Alternative fibers such as boron fibers have high Young modulus of elasticity as well as good strength.

In presence of high temperatures, different types of fibers may be used, such as ceramic fibers (*e.g.*, alumina fibers and silicon carbide fibers), whose mechanical characteristics are reported in Table 2-5 along with those of boron fibers.

Table 2-5 – Properties of boron and ceramic fibers.

	Boron fibers	Ceramic fibers		
		Alumina (CFP)*	SiC (CVD)**	SiC (pyrolysis)
Diameter [μm]	16.5	20±5	140	10-20
Density [g/cm^3]	2.63	3.95	3.3	2.6
Failure stress [MPa]	2800	1380	3500	2000
Young's modulus [GPa]	385	379	430	180

(*) *Chemically Formed Processes*
(**) *Chemical Vapour Deposition*

2.2.1.6 Technical characteristics of yarn

Yarns are not available on the market as strengthening materials; instead, they are used as raw material for the production of fabrics. Hereafter, the structure of a typical technical data sheet for yarn is proposed. The international reference standard is ISO 2113:1996(E).

ISO 1889:1997 (E) can be used to determine the count of a yarn. A sample of any given length should be taken from the fabric and should be weighted; the count value is given by the following ratio:

$$T_x = \frac{P \cdot 1000}{L} \tag{2.1}$$

where T_x is the count of the yarn, expressed in Tex $[\mathrm{g/km}]$; P is the weight of the sample, expressed in grams; and L is the length of the sample, expressed in meters.

The area A, in mm^2, of the cross-section of a filament or bundle (*yarn*, *tow*, or *roving*), can be determined using the following equation:

$$A = \frac{T_x}{\rho \cdot 1000} \tag{2.2}$$

where ρ is the yarn density, expressed in $\mathrm{g/cm^3}$; and T_x is the count, expressed in TEX.

The evaluation of such parameters may be useful for production quality control.

TECHNICAL DATA SHEET: yarn

THE MANUFACTURER SHALL REPORT THE STATISTICAL VALUES NEEDED TO EVALUATE THE STRENGTH CHARACTERISTICS (E.G. SAMPLE MEAN, SAMPLE STANDARD DEVIATION, POPULATION, PERCENTILE, CONFIDENCE INTERVAL).

Yarn description

Commercial name, type of yarn, twisting, finishing, and any other information deemed necessary.

Yarn characteristics

property	Measurement unit	Test method Reference standard
fiber diameter	μm	
fiber density	$\mathrm{g/cm^3}$	
no. of filaments		
count	Tex	ISO1889:1997(E)
type of finishing (size)		
finishing content	%	ISO1887:1995(E) ISO10548:2002(E)
Young modulus of elasticity	GPa	ISO10618:1999(E)
tensile strength (average and characteristic value)	MPa	ISO10618:1999(E)
failure strain	%	ISO10618:1999(E)
moisture content	%	ISO3344:1997(E)

Storage conditions

Description

Safety and handling

Description

2.2.2 Non-impregnated fabrics

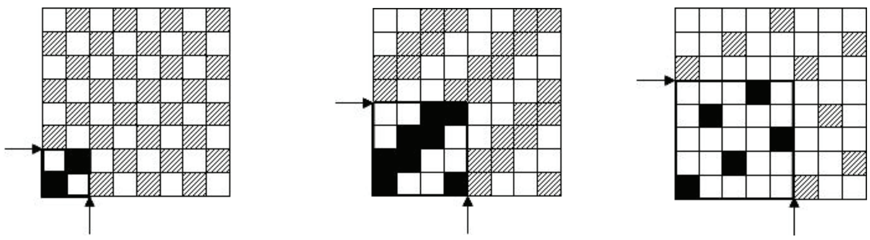
The *fabric* that is not impregnated with resin is named “*dry*.” The simplest fabric is obtained starting from a roving and is named "*woven roving.*" Since the *roving* does not exhibit any twisting, the filament is transversely compressed where weft and warp cross each other. The resulting *fabric* is suitable to realize large products in size and thickness.

Fabrics obtained directly from the weaving of the yarns, being lighter and more compact, can be used for more specific applications that require an optimization of the structural weight. A composite laminate obtained from these fabrics has a lower volumetric fraction of fibers than a laminate made of unidirectional fiber due to the crimp associated to weaving.

The most used types of fabric are plain, twill and satin. Plain fibers exhibit the stiffest and most stable structure. The main disadvantages are the difficulty of impregnation with resin as well as the crimp of weft and warp. This latter characteristic implies a lower strengthening effectiveness on the plane of the laminate. The crimp for such fabrics is about 10 %. Twill fibers and satin fibers are more flexible but relatively prone to be damaged during manipulation. The satin fabric is intrinsically stiffer in the lamination plane, since its has the least crimp of fibers in both directions.

Figure 2-8 shows the geometries of the most used fabrics in current applications. The representation complies with the following assumptions:

- Black or dashed box = weft yarn on top of warp yarn
- White box = weft yarn under warp yarn



Plain *Twill* *Satin*

Figure 2-8 – Fabric examples.

There are also multi-axial fabrics, where the fibers are oriented in more than two directions. They can be made of woven yarns or simply sewn yarns. Finally, three-dimensional fabrics are also available, where the presence of a second weft in a direction orthogonal to the plane provides the product with higher strength and special properties (e.g. the capability to inflate when they are impregnated with resin).

2.2.2.1 Technical characteristics of non-impregnated fabrics

Fabrics for structural strengthening are commonly distributed as a dry product to be impregnated with special resins at the job site. They can be unidirectional, where the fibers are all oriented in the direction of the length and kept together by a light non-structural weft; bi-directional, made of a orthogonal weft-warp weaving, usually balanced (same ratio of fibers in the two directions); and multi-axial, where fibers are oriented in different directions. Dry fiber manufacturers are required to provide material data sheets. The structure of a material data sheet is reported hereafter for mono- and bi-directional fabrics; data sheets of commercially available fabrics may also include other information or parts of those indicated. The suggested structure is exhaustive regarding the type and amount of information provided.

TECHNICAL DATA SHEET: non-impregnated fabric

THE MANUFACTURER SHALL REPORT THE STATISTICAL VALUES NEEDED TO EVALUATE THE STRENGTH CHARACTERISTICS (E.G. SAMPLE MEAN, STANDARD DEVIATION, POPULATION, PERCENTILE, CONFIDENCE INTERVAL).

Fabric description

Type of weave (plain, twill, satin, etc.), type of yarn (weft and warp), characteristics other than weft and warp (finishing, veil, wrapping, etc.), and any other information deemed necessary.

Fabric characteristics

Property	Direction of yarn	Measurement unit	Test method Reference standard
yarn count	warp	Tex	ISO 1889:1997(E)
	weft	Tex	
yarn density		g/cm^3	
no. of yarns/cm	warp	n°/cm	ISO 4602:1997(E)
	weft	n°/cm	
mass (weight)	total	g/m^2	ISO 3374:2000(E)
	warp	g/m^2	
	weft	g/m^2	
Young modulus of elasticity for tensile stress	warp	MPa	
	weft	MPa	
tensile strength (mean and characteristic value)	warp	[N]	ISO 4606:1995(E) (textile glass) ISO 13934-1:1999(E)
	weft	[N]	
failure strain	warp	%	ISO 4606:1995(E) (textile glass) ISO 13934-1:1999(E)
	weft	%	

Characteristics of the yarn

See the yarn technical data sheet.

Storage conditions

Description.

Safety and handling

Description.

Indications for use as strengthening system

The manufacturer may indicate other products to couple with the fabric for the realization of the strengthening systems, such as impregnation resins, possible protective coatings, primer, putty, etc. Such information shall be accompanied by the results of compatibility tests performed on the complete system (see Section 2.5).

The general reference standard is ISO 8099:1980. For multi-axial fabrics, in addition to the general information concerning the type of yarn and other characteristics of the fabric, the orientation of each layer of fibers should be reported as well. In the following, examples concerning the determination of some characteristic parameters of the fabrics used for structural strengthening are illustrated.

In cases where only the yarn count and geometry are provided, the mass of fibers per unit area in a given direction can be determined with the following equation:

$$p_x = \frac{T_x \cdot N_f}{10} \tag{2.3}$$

where p_x is the mass of the fabric in the principal direction, expressed in g/m^2 ; T_x is the *yarn* count referred in the principal direction, expressed in Tex [g/km]; and N_f is the number of *yarns* per unit width in the principal direction [yarns/cm].

For example, given a unidirectional fabric characterized by 3.8 yarns/cm and by a yarn count of 800 Tex, the resulting mass per unit area is:

$$p_x = \frac{800\ [\text{Tex}] \cdot 3.8\ [\text{yarns/cm}]}{10} = 304\ \text{g/m}^2$$

If it is necessary to evaluate the number of yarns arranged in a given direction per unit length in the orthogonal direction, ISO 4602:1997(E) can be applied: the yarns arranged in the orthogonal direction on a given fabric strip (*e.g.*, 10 cm wide) are counted, and the resulting number is varied proportionally to the chosen unit length.

2.2.3 Matrices

Thermoset resins are the most commonly used matrices for 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 resin have several advantages, including low viscosity that allows for a relative easy fiber impregnation, good adhesive properties, room temperature polymerization characteristics, good resistance to chemical agents, absence of melting temperature, etc. Disadvantages are limited range of operating temperatures, with the upper bound limit given by the glass transition temperature, poor toughness with respect to fracture ("brittle" behavior), and sensitivity to moisture during field applications. The most common thermosetting resins for civil engineering are the epoxy resin. Polyester or vinylester resins are also used. Considering that the material is mixed directly at the construction site and obtains its final structural characteristics through a chemical reaction, it should always be handled by specialized personnel.

Fiber-reinforced composite materials with thermoplastic polymeric matrices are also available but require installation techniques different from thermosetting resin. Composite bars with thermoplastic matrix that may be bent at any time by means of special thermal treatment are currently being investigated.

2.2.3.1 Epoxy resins

Epoxy resins are characterized by a good resistance to moisture, chemical agents, and have excellent adhesive properties. They are suitable for production of composite material in the civil engineering field. The maximum operating temperature depends both on formulation and reticulation temperature. For operating temperatures higher than 60 °C, the resin should be suitably selected by taking into account the variations of its mechanical properties. There are usually no significant restrictions for the minimum operating temperature. The main reagent is composed of organic fluids with a low molecular weight, containing a number of epoxy groups, rings composed by a oxygen atom and two carbon atoms:

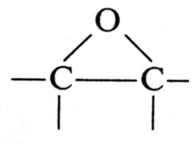


Figure 2-9 – Epoxy group.

Such materials may be produced by the reaction of epichlorohydrin with amino compounds or acid compound of bisphenol A.

The epoxy pre-polymer is usually a viscous fluid, with viscosity depending on the polymerization degree. A reticulating agent (typically an aliphatic amine) is to be added to this mixture in the exact quantity to obtain the correct structure and properties of the crosslinked resin. The reaction is exothermic and does not produce secondary products. It can be carried out at both room and high temperatures, according to the technological requirements and the target final properties. The chemical structure of the resin may be changed on the basis of the chemical composition of the epoxy pre-polymer. The most commonly used epoxy resin in composite materials for civil applications is the diglycidylether of bisphenol A (DGEBA).

2.2.3.2 Polyester resins

Polyester resins have a lower viscosity compared to epoxy resins, are very versatile, and highly reactive. Their mechanical strength and adhesive properties are typically lower than those of epoxy resins.

Unsaturated polyesters are linear polymers with a high molecular weight, containing double C=C bonds capable of producing a chemical reaction. The polymerization degree, and hence the molecule length may be changed; at room temperature the resin is always a solid substance. To be used, polyester resin has to be dissolved in a solvent, typically a reactive monomer, which reduces the resin viscosity and therefore aids the fiber impregnation process. The monomer (typically styrene) shall also contain double C=C bonds, allowing cross-linking bridges between the polyester molecules to be created. The reaction is exothermic and no secondary products are generated. It is usually performed at room temperature, according to technological requirements and target final properties. The chemical structure of polyester resins may be adapted either by changing the acid and the glycol used in the polymer synthesis or by employing a different reactive monomer.

The family of polyester resins for composite materials is typically composed of isophthalic, orthophthalic, and bisphenolic resins. For both high temperatures and chemically aggressive environment applications, vinylester resins are often used; they represent a compromise between the performance of traditional polyester resins and that of epoxy resins.

2.2.3.3 Other types of resins

The intrinsic limitations of thermosetting resins, in particular their poor toughness, their quite low operating temperatures, and their tendency to absorb moisture from the environment, have recently led to the development of composites with a thermo-plastic matrix. Such resins have the capability of flowing after heating at a high enough temperature, specifically, higher than T_g (glass transition temperature) for amorphous materials and higher than T_m (melting temperature) for semi-crystalline materials. The shape of each components may be modified by simply heating the material at a suitable temperature (hot forming). Their use in the civil engineering field is rather limited at present; however, applications of potentially remarkable relevance are currently being developed (*e.g.,* reinforcing bars for concrete). In general, thermoplastic resins are tougher than thermosetting resin, and in some instances have higher operating temperatures. In addition, they have a better resistance to environmental factors. The main limitation for their use is their high viscosity, which makes fiber impregnation difficult, and requires complex and costly working equipment.

Moreover, the use of inorganic matrices (cement-based, metallic, ceramic, etc.) for production of fiber-reinforced composites for construction is rapidly growing. Even though they are not discussed in this document, their use is deemed possible when accompanied by suitable technical documentation and experimental validation to prove their effectiveness.

2.2.3.4 Technical data sheet of the resin

The structure of a typical technical data sheet for resin is reported as an example (technical data sheets available on the market could report other information or only a portion of those indicated here). The suggested structure is exhaustive regarding type and amount of provided information.

TECHNICAL DATA SHEET: resin

Resin description

Commercial name, mono-or bi-component, pasty or fluid consistency, use, and any other necessary information.

Characteristics of unmixed resin

property		Measurement unit	Comp. A	Comp. B	Mixture	Test method Reference standard	Notes
color							
viscosity at 25 °C		Pa·s				ISO 2555:1989(E) ISO 3219:1993(E)	(1)
thyxotropy index						ASTM D2196-99	(1)
density		g/cm^3				ISO 1675:1985(E)	
mixing ratio	volume	%					
	weight						
storage conditions (sealed container)	time	months					
	temperature	° C					

(1) For non thyxotropic resins the Garner viscosimeter can be used; for thyxotropic resins the Brookfield viscosimeter shall be used.

Characteristics of mixed resin

Mixing conditions:
Description

Application conditions:
Description

property		Measurement unit	Test method Reference standard	Notes
pot life (at 35 °C)		Pot life (at 35 °C)	ISO 10364:1993(E)	(2)
gel time	At 5 °C	min	ISO 9396:1997(E) ISO 2535:2001(E) ISO 15040:1999(E)	(3)
	At 20 °C			
	At 35 °C			

%

minimum application temperature		°C		
exothermic peak	Time	min	ISO 12114:1997(E)	
	temperature	°C		
full cure time	At 5 °C	min	ISO 12114:1997(E)	
	At 20 °C			
	At 35 °C			

(2) Pot life (working life) = maximum working time after mixing of all components.
(3) Gel time = time needed from fluid to gel appearance at predefined temperature conditions.

Characteristics of cured resin

property	Meas-urement unit	Test tempera-ture	Value		Test method Reference standard
			Cured 5 days at 22 °C	Cured 1 hour at 70 °C	
volume shrinkage		---			ISO 12114:1997(E)
coefficient of thermal expansion	10^{-6} °C^{-1}	---			ISO 11359-2:1999(E)
glass transition temperature	°C	---			ISO 11357-2:1999(E) (DSC) ISO11359-2:1999(E) (TMA) ASTM E1640 (DMA)
Young modulus of elasticity for tensile stress	GPa				ISO 527:1993(E)
tensile strength	MPa				ISO 527:1993(E)
failure strain	%				ISO 527:1993(E)

Storage conditions
Descriptions

Safety ad handling
Description

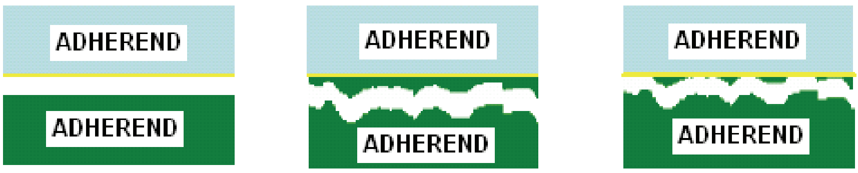
2.2.4 Adhesives and bonding principles

The implementation of FRP-based structural strengthening (*e.g.*, pultruded laminate) requires the use of adhesives. The choice of the most suitable adhesive as well as the type of surface treatment to be carried out prior to FRP application shall be made on the basis of available substrate and properties of the selected FRP system. Technical data sheets for FRP materials usually report the indications of the adhesive to be used as a function of the structure to be strengthened. Even the application of dry fabrics impregnated on-site might be considered as an assembling operation using adhesives. The type of surface treatment to be carried out prior to FRP application is important for the correct use of adhesives. For this reason, the rationale for a suitable substrate preparation that describes physical, chemical, and mechanical mechanisms of adhesion is presented. For a more comprehensive study, the reader is referred to specific literature on the subject.
An adhesive is a material quite often of a polymeric nature capable of creating a link between at least two surfaces and able to share loads. There are many types of natural and synthetic adhesives (elastomers, thermoplastics, and mono- or bi-component thermosetting resins); the most suitable adhesives for composite materials are based on epoxy resins. Epoxy adhesives usually are bi-component viscous mixture; once hardened, through a cross-linking chemical reaction, they become suitable for structural applications.

There are several advantages in the use of adhesive bonding compared to mechanical anchorage. They include the possibility of connecting different materials, providing greater stiffness, uniform distribution of loads, and avoiding holes dangerous for stress concentrations. On the other hand, adhesives are sensitive to environmental conditions, such as moisture, and are not appropriate when exposed to high temperatures (fire resistance).

The following three types of fracture can be identified for adhesive bonding (Figure 2-10).

- <u>Cohesive fracture</u>: it takes place inside one of the materials forming the connection. The same material is therefore on both sides of the fracture surface, which may be either smooth or rough. It is the ideal fracture for adhesives.
- <u>Adhesive fracture</u>: it takes place at the interface between adhesive and support when the adhesive strength is lower than that of the support. The fracture surface is typically smooth. This type of fracture highlights inaccurate applications.
- <u>Mixed fracture</u>: it appears as both cohesive and adhesive failure. The fracture surfaces are very irregular and characterized by coexistence of both materials. It appears for both weak and non-consolidated support (*e.g.*, deteriorated masonry or concrete) and inaccurate adhesive applications.

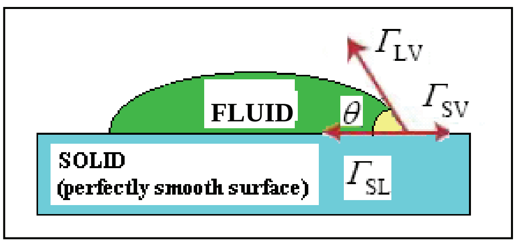


adhesive fracture   cohesive fracture   mixed fracture

Figure 2-10 – Comparison between different types of fracture.

The efficiency of adhesion depends on many factors, such as surface treatment, chemical composition and viscosity of the adhesive, application technique, and hardening or cross-linking process of the adhesive itself. Adhesion mechanisms primary consist of interlocking of the adhesive with the surface of the support with formation of chemical bonds between polymer and support. As a result, adhesive strength is enhanced by surface treatments that improve interfacial properties of the support by increasing the roughness of the surface to be strengthened. Several types of adhesive mechanisms are described in the literature and briefly summarized hereafter.

<u>Physical bond</u>: it involves secondary bonds, such as Van der Waals forces, ionic bonds, and hydrogen bonds between molecules of the adhesive and those of the support. It is based on electrostatic and absorption theory, for which good adhesion is ensured when the adhesive is capable of wetting the support. To this end, the surface energy of the support Γ_{SV} (energy per unit area) shall be higher than that of the adhesive Γ_{LV} (Figure 2-11): for example, wetting properties of epoxy resins bonded to steel support are excellent; they are poor for other materials such as polyethylene.



θ = contact angle

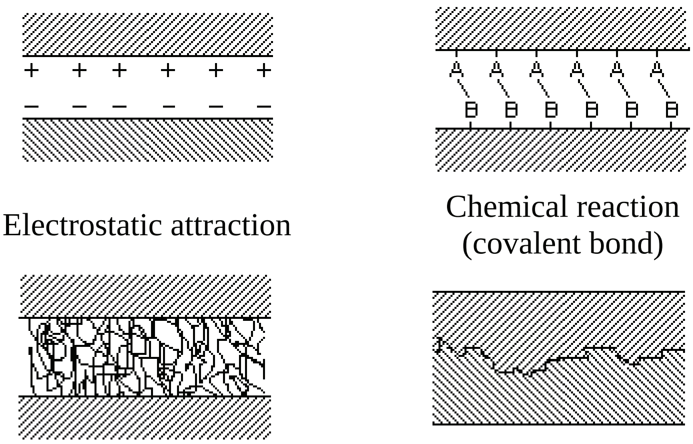
$\theta = 0^\circ$	complete wrapping
$0^\circ < \theta < 180^\circ$	partial wetting
$\theta = 180^\circ$	no wetting

Figure 2-11 – Adsorption theory and contact angle.

Chemical-covalent bond: it involves primary bonds (covalent bonds) between the molecules of the support and the adhesive (Figure 2-12). The fracture implies breaking of such bonds. For this reason, coupling agents are often used particularly in the case of glass fibers. Coupling agents link with the oxides of the surface support to react with the adhesive during cross-linking or causing diffusive phenomena (see the following item).

Diffusive or inter-diffusive phenomena: after diffusion or inter-diffusion of atoms or molecules across the interface, the bond between two surfaces is generated (Figure 2-12). The described mechanism is typical for polymeric matrix composites, where the mobility of polymeric chains makes linking possible; in such a case, time is important to allow the adhesive reaching the final strength.

Mechanical interlock theory: the bond exploits the mutual creep resistance between locally permeated surfaces; therefore, it is important to have irregular surfaces to allow spreading of the adhesive, filling of the support pores, and cracks before solidification takes place (Figure 2-12).



Electrostatic attraction

Chemical reaction (covalent bond)

Inter-diffusion and formation of connections

Mechanical interlocking

Figure 2-12 – Adhesive mechanisms.

The main objective of surface treatment is “cleaning” of the surface by removal of all possible surface contaminations, such as oxides, foreign particles, oil, laitance, dust, moisture, etc. The adopted treatment usually modifies the chemistry of the surface, enhancing the formation of stronger bonds with the adhesive such to resist environmentally aggressive agents, which would degrade the adhesion over time. Finally, treatments should ensure adequate surface roughness.

Other treatments such as solvents and sandblasting can be effectively employed. In some instances, prior to adhesive application, a layer of primer acting as a coupling agent is applied. The use of FRP pultruded laminate requires an additional cleaning of the laminate face prior to bonding. In some cases, laminates have a protective film that prevents external contamination. Such films shall be removed before the laminate is applied. Any surface treatment shall be carried out immediately before FRP application takes place to avoid surface recontamination. Presence of moisture in the support shall be avoided during FRP application; the surfaces of the support shall be perfectly dry prior to application of the adhesive. In the following of this document, recommendations to reduce the risk of failures associated to bonding are presented.

2.2.4.1 Technical data sheet of the adhesive

The most suitable adhesives for composite materials are those based on bi-component epoxy resins. Technical data sheets shall therefore include the chemical-physical properties of each single components as well as the adhesive properties. Because the former have already been listed in 2.2.3.4, the data sheet here reported only refers to the adhesive properties.

TECHNICAL DATA SHEET: adhesive

THE MANUFACTURER SHALL REPORT THE STATISTICAL VALUES NEEDED FOR THE EVALUATION OF CHARACTERISTIC PROPERTIES (E.G. SAMPLE MEAN, STANDARD DEVIATION, POPULATION, PERCENTILE, CONFIDENCE INTERVAL)

Adhesive description

Commercial name, mono- or bi-component, pasty or fluid consistency, use, and any other information deemed useful.

Bonding properties of the resin

property	Meas-urement unit	Test tempera-ture	Value		Test method Reference standard
			Cured 5 days at 22 °C	Cured 1 hours at 70 °C	
shear strength (average and characteristic value)	MPa				*single lap shear* ISO 4587:2003(E)
peeling strength (average and characteristic value)	kN/m				*floating-roller method* ISO 4578:1997(E)

Note: for external strengthening with FRP laminate an ISO standard (TC71/SC6N) "Non-conventional strengthening of concrete - Test methods-Part 2: Fiber strengthened polymer (FRP) sheets" is under preparation, where two new tests are proposed to evaluate the bond on concrete: "Test Method for direct pull-off strength of FRP sheets with concrete" and "Test Method for bond properties of FRP sheets to concrete". A similar pull-off test, "Test method for direct tension pull-off test", is also proposed in the document ACI 440.3R-04 "Guide Test Methods for Fiber-reinforced Polymers for Reinforcing or Strengthening Concrete Structures" by the American Concrete Institute.
Such standards do not propose specific tests for bond on steel. There is however a similar Japanese standard (JSCE-E544-2000 "Test methods for continuous fiber sheets"), proposing also a lap shear strength test for FRP and steel The cited documents include also a shear strength test of the adhesive based on shear lap test.

Storage conditions

Descriptions

Safety and handling

Description

2.3 FRP STRENGTHENING SYSTEMS

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

- Pre-cured systems (2.3.2):
  Manufactured in various shapes by pultrusion or lamination, pre-cured systems are directly bonded to the structural member to be strengthened.
- Wet lay-up systems (2.3.3):
  Manufactured with fibers lying in one or more directions as FRP sheets or fabrics and im-

pregnated with resin at the job site to the support.

- Prepreg systems (2.3.4):
  Manufactured with unidirectional or multidirectional fiber 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.

2.3.1 Mechanical properties of FRP strengthening systems

In FRP materials, fibers provide both loading carrying capacity and stiffness to the composite while the matrix is necessary to ensure sharing of the load among fibers and to protect the fibers themselves from the environment. Most FRP materials are made of fibers with high strength and stiffness, while their strain at failure is lower than that of the matrix.

Figure 2-13 shows the stress-strain relationship for fiber, matrix, and the resulting FRP material. The resulting FRP material has lower stiffness than fibers and fails at the same strain, $\varepsilon_{f,max}$, of the fibers themselves. In fact, beyond such ultimate strain, load sharing from fibers to the matrix is prevented.

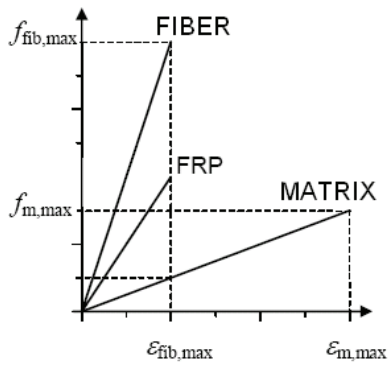


Figure 2-13 – Stress-strain relationship of fibers, matrix and FRP.

Table 2-6 summarizes mechanical properties of a pre-cured laminate compared to the average values of the corresponding fibers. The values of Young modulus of elasticity, E_f, and ultimate strength at failure, f_f, of the laminate are lower than those of the fiber itself, while the ultimate tensile strain is of the same order of magnitude for both materials.

Table 2-6 – Comparison between mechanical properties of a pre-cured laminate and fibers

Pre-cured systems	Modulus of elasticity [GPa]		Ultimate strength [MPa]		Ultimate strain [%]	
	FRP	Fiber	FRP	Fiber	FRP	Fiber
	E_f	E_{fib}	f_f	f_{fib}	ε_{fu}	$\varepsilon_{fib,u}$
CFRP (low modulus)	160	210-230	2800	3500-4800	1.6	1.4-2.0
CFRP (high modulus)	300	350-500	1500	2500-3100	0.5	0.4-0.9

For FRP material made of unidirectional fibers, the mechanical behavior of the composite can be estimated using micro-mechanical models; for example, by using the rule of mixtures [eq. (6.5) in Appendix A]:

$$E_f = V_{fib} \cdot E_{fib} + (1 - V_{fib}) \cdot E_m \tag{2.4}$$

$$f_f \cong V_{fib} \cdot f_{fib} + (1 - V_{fib}) \cdot f_m \tag{2.5}$$

where V_{fib} is the volumetric fraction of fibers (ratio between the volume of fibers and the overall volume of the composite), and E_{fib} and E_m are the Young modules of elasticity of fibers and matrix, respectively. The rule of mixtures is based on the hypothesis of a perfect bond between fibers and matrix; for unidirectional composites it provides accurate assessment of the modulus of elasticity. The same accuracy can not be obtained for ultimate strength. For design purposes, it is always preferable to rely upon experimental determination of the above mentioned values (E_f and f_f), as discussed in the following.

For proper definition of stiffness and strength properties of FRP composites impregnated in-situ, geometrical (volumetric ratio or weight ratio between fibers and matrix) and mechanical characteristics of each components shall be available. For example, a unidirectional 100 mm wide fabric (fibers area $A_{fib} = 70\,\text{mm}^2$) impregnated with variable quantities of resin is considered. By dividing the overall area of the impregnated fabric, A_f (obtained as the sum of resin and fibers area), by the fabric width, the thickness of the composite is obtained. The properties of each components are reported in Table 2-7. The importance of resin content on the mechanical properties in the fiber direction, calculated using Equations (2.4) and (2.5), is summarized in Table 2-8.

Table 2-7 – Properties of components.

Fibers	Matrix
E_{fib}= 220 GPa	E_m= 3 GPa
f_{fib}= 4000 MPa	f_m= 80 MPa

Table 2-8 – Importance of fiber volumetric fraction on the FRP mechanical properties

A_{fib} [mm^2]	A_m [mm^2]	A_f [mm^2]	V_{fib} [%]	E_f [GPa]	f_f [MPa]	ε_{fu} [%]	F_{fu} [kN]	$E_f \cdot A_f$ [kN]
70	0	70	100	220.0	4000	1.81	280.0	15400
70	30	100	70	154.9	2824	1.82	282.4	15490
70	70	140	50	111.5	2040	1.83	285.6	15610
70	163.3	233.3	30	68.1	1256	1.84	293.0	15890

Table 2-8 and Figure 2-14 refer to volumetric fractions of fibers ranging between 30 and 100 %.. Values of fiber stiffness and strength are remarkably larger than those of the matrix (Table 2-7). As a result, mechanical properties of FRP composite materials (E_f and f_f) are mainly controlled by fibers, and the matrix contribution can be neglected.

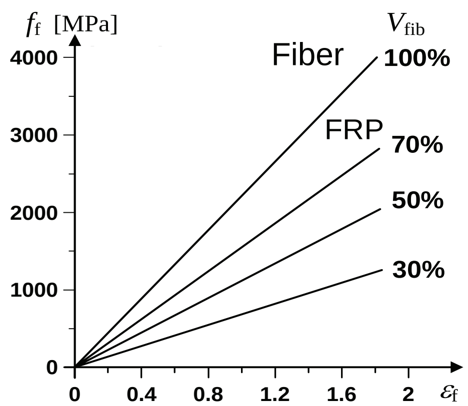


Figure 2-14 – Stress-strain relationship as a function of the volumetric fraction.

When the above mentioned properties are referred to the overall section of the composite, both Young modulus of elasticity and strength at failure decrease as the resin content increases. The same does not apply if one were to look at both ultimate force, F_{fu}, and axial stiffness ($E_f \cdot A_f$) whose variations (3-4 %) are negligible. In fact, the reduction of E_f and f_f are compensated by the increase of the overall cross-sectional area with respect to that of fibers only.

The example shows that when evaluating FRP mechanical properties to be used for design, the resin volumetric fraction needs to be available when referring to stiffness and ultimate strength of the composite. Alternatively, FRP axial stiffness and ultimate force at failure, F_{fu}, calculated neglecting the matrix, shall be available. Such values do not take into account other relevant parameters associated to the composite manufacturing process that significantly affect the mechanism of collapse.

2.3.2 Pre-cured systems

2.3.2.1 Mechanical characteristics

Pre-cured composites are characterized by a unidirectional disposition of fibers that allows the use of the rule of mixtures to determine stiffness and strength of the composite, since the volumetric fractions usually vary between 50 and 70 %. However, such values only represent an estimate (typically an overestimation) because other relevant parameters, such as adhesion properties between fibers and matrix, presence of manufacturing defects, voids, or fibers misalignment, are not taken into account. Reliable values of FRP mechanical properties shall be obtained with experimental testing to ensure determination of appropriate statistical parameters accounting for the adopted manufacturing process as well. The values provided by FRP manufacturers shall be based on criteria similar to those discussed in 2.4. In case of pre-cured systems, manufacturers typically provide mechanical characteristics referred to the laminate cross-section having a well specified size.

2.3.2.2 Technical data sheets for pre-cured systems

Hereafter, the structure of a typical data sheet for pre-cured composites (laminates, bars, grids, etc.) is proposed. As previously seen, technical data sheets available on the market can also include other information or report only a portion of those here indicated. The proposed structure is exhaustive regarding the type and amount of information given.

TECHNICAL DATA SHEET: pre-cured composites
(laminates, bars, grids)

THE MANUFACTURER SHALL REPORT THE STATISTICAL VALUES NEEDED FOR THE EVALUATION OF THE CHARACTERISTIC STRENGTHS (E.G. SAMPLE MEAN, STANDARD DEVIATION, POPULATION, PERCENTILE, CONFIDENCE INTERVAL)

Description
Commercial name, type of fiber, type of resin, manufacturing technology (pultrusion, lamination, etc.), and any other information deemed useful.

Geometrical and physical properties

property	Measurement unit	Test method Reference standard	Notes
thickness (laminate)	mm		
width	mm		
length	mm		

%

section geometry (bars, grids)				
nominal area (bars, grids)		mm^2		
nominal perimeter (bars, grids)		mm		(1)
color				
density	fiber	g/cm^3	ISO 1183-1:2004(E)	(2)
	matrix	g/cm^3		
fiber content	weight	%	ISO 11667:1997(E)	
	volume	%		
glass transition temperature of resin (T_g)		°C	ISO 11357-2:1999(E) (DSC) ISO11359-2:1999(E) (TMA) ASTM E1640 (DMA)	
maximum operating temperature		°C		
electrical conductivity		S/m		

(1) Value to be used in case of bars and grids of non circular cross section to compute bond length.
(2) Value to be used to compute the weight fraction of fiber when the volume fraction is known and vice versa.

Properties

property	Measurement unit	Test method Reference standard	Notes
tensile Young modulus of elasticity	GPa	ISO 527-4,5:1997(E)	
tensile strength (average and characteristic value)	MPa	ISO 527-4,5:1997(E)	
strain at failure	%	ISO 527-4,5:1997(E)	
compression Young modulus of elasticity (bars)	GPa	ISO 14126:1999(E)	
compressive strength (bars) (average and characteristic value)	MPa	ISO 14126:1999(E)	
compressive strain at failure (bars)	%	ISO 14126:1999(E)	
creep		ISO 899-1:2003(E)	(3)
relaxation (bars, grids)			(4)
bond: shear stress (bars, grids)		Pull-out test	(4)

(3) The ISO 899-1:2003(E) standard is the general reference standard for creep behavior of FRP materials, while the ISO standard (TC71/SC6N) "Non-conventional strengthening of concrete - Test methods-Part 1: Fiber strengthened polymer (FRP) bars and grids" is under way for reinforcing FRP bars and prestressed grids, where a specific test is proposed for FRP bars ("Test Method for creep failure"). As an alternative, there is a test proposed in the document ACI 440.3R-04 "Guide Test Methods for Fiber-reinforced Polymers for Reinforcing or Strengthening Concrete Structures" named "Test Method for creep rupture of FRP bars".
(4) In the ISO standard (TC71/SC6N) concerning FRP bars and grids the following two tests are proposed: "Test method for bond strength by pull-out testing" for bond, and "Test Method for long-term relaxation" for relaxation. Similar tests are included in the ACI 440.3R-04 document.

Storage conditions
Description

Safety and handling
Description

Indications for use as strengthening system
The manufacturer may indicate other products to couple with the pre-cured composite for the realization of the strengthening system, such as adhesives, possible protective coatings, primer, putty, etc. Such information shall be accompanied by compatibility test results performed on the as-proposed system.

2.3.3 Wet lay-up systems

In case of wet lay-up systems, final thickness of the FRP laminate can not be estimated in a deterministic fashion. Therefore, it is recommended to refer to both mechanical and geometrical properties of dry fabric according to the technical data sheets provided by FRP manufacturer.

2.3.3.1 Determination of laminate cross sectional area

For the laminate cross sectional area, A_{rt}, reference shall be made to the technical data sheet provided by FRP manufacturer. Laminate cross sectional area per unit width can be expressed as follows:

$$A_{\mathrm{rt}} = \frac{T_{\mathrm{x}} \cdot N_{\mathrm{f}}}{10 \cdot \rho_{\mathrm{fib}}} \tag{2.6}$$

where A_{rt} is expressed in $\mathrm{mm^2/m}$, T_{x} is the yarn count referring to the principal direction expressed in TEX [g/km], N_{f} is the number of yarns per unit width referring to the principal direction expressed in [yarns/cm], and ρ_{fib} is the fiber density [$\mathrm{g/cm^3}$].

In case of fabrics with the same number of fibers in two orthogonal directions (balanced fabrics), laminate cross sectional area, A_{rt}, may also be calculated as follows:

$$A_{\mathrm{rt}} = \frac{p_{\mathrm{t}}}{2 \cdot \rho_{\mathrm{fib}}} \tag{2.7}$$

where p_{t} is the fabric mass per unit area, expressed in $\mathrm{g/m^2}$.

In case of a unidirectional fabric, A_{rt}, may be evaluated as follows:

$$A_{\mathrm{rt}} = \frac{p_{\mathrm{t}}}{\rho_{\mathrm{fib}}} \tag{2.8}$$

Sometimes it is common to refer A_{rt} to the thickness of an equivalent plate made of fiber only. The equivalent thickness, t_{eq}, expressed in mm, can be obtained as follows:

$$t_{\mathrm{eq}} = \frac{A_{\mathrm{rt}}}{1000} \tag{2.9}$$

A disadvantage of this method is that different thickness values of the equivalent plate would be associated to the same unbalanced fabric (*e.g.,* a fabric having different number of fibers in two orthogonal directions).

Table 2-9 summarizes the parameters deemed necessary for the determination of the laminate cross sectional area for three different fabrics as follows: 1) unbalanced plain weave fabric (fabric A), 2) balanced plain weave fabric (fabric B), and 3) unidirectional fabric (fabric C).

Table 2-9

Property		Measurement unit	Fabric A	Fabric B	Fabric C
Fabric mass		$\mathrm{g/m^2}$	187	286	304
Fiber density		$\mathrm{g/cm^3}$	1.76	1.76	1.8
No. of yarns/cm	weft	no/cm	4	6	-
	warp	no/cm	8	6	3.8
Count	weft	Tex	67	200	-
	warp	Tex	200	200	800

In case of unbalanced fabric (fabric A), Equation (2.6) yields:

$$A_{\mathrm{rt}}^{\mathrm{weft}} = \frac{67[\mathrm{Tex}]\cdot 4[\mathrm{yarns/cm}]}{10\cdot 1.76[\mathrm{g/cm^3}]} = 15.2\,\frac{\mathrm{mm^2}}{\mathrm{m}} \qquad \text{(area in weft direction)}$$

$$A_{\mathrm{rt}}^{\mathrm{warp}} = \frac{200[\mathrm{Tex}]\cdot 8[\mathrm{yarns/cm}]}{10\cdot 1.76[\mathrm{g/cm^3}]} = 90.9\,\frac{\mathrm{mm^2}}{\mathrm{m}} \qquad \text{(area in warp direction)}$$

For case B fabric, cross sectional area can be written as follows (both directions):

$$A_{\mathrm{rt}} = \frac{200\ [\mathrm{Tex}]\cdot 6\ [\mathrm{fili/cm}]}{10\cdot 1.76\ [\mathrm{g/cm^3}]} = 68.2\ \frac{\mathrm{mm^2}}{\mathrm{m}}$$

Alternatively, the same result can be obtained from Equation (2.7) as follows:

$$A_{\mathrm{rt}} = \frac{p_{\mathrm{t}}\ \left[\mathrm{g/m^2}\right]}{2\cdot\rho_{\mathrm{fib}}\ \left[\mathrm{g/cm^3}\right]} = \frac{240}{2\cdot 1.76}\ \frac{\mathrm{mm^2}}{\mathrm{m}} = 68.2\ \frac{\mathrm{mm^2}}{\mathrm{m}}$$

Finally, for case C fabric, Equation (2.8) provides:

$$A_{\mathrm{rt}} = \frac{p_{\mathrm{t}}\ \left[\mathrm{g/m^2}\right]}{\rho_{\mathrm{fib}}\ \left[\mathrm{g/cm^3}\right]} = \frac{304}{1.80}\frac{\mathrm{mm^2}}{\mathrm{m}} = 168.9\,\frac{\mathrm{mm^2}}{\mathrm{m}}$$

2.3.3.2 Mechanical characteristics

Mechanical properties of wet lay-up systems cannot be determined by multiplying the cross sectional area, A_{rt}, computed according to Equation (2.6), by both Young modulus of elasticity and ultimate fiber or fabric strength. In fact, values referring to fibers do not take into account the real geometry of the fabric (weaving, weft-warp) that considerably affect the fabric mechanical properties. Similarly, values of stiffness and strength referred to dry fabric, when available, cannot be used directly for the determination of the same properties referring to the final composite.

As a result, mechanical properties of wet lay-up systems can be determined using two different methods; both of them require information that shall be provided by FRP manufacturer/supplier.

Mode 1

Safety factors for both stiffness, α_{fE}, and strength, α_{ff}, can be introduced as follows:

$$A_f \cdot E_f = \alpha_{fE} \cdot A_{fib} \cdot E_{fib} \tag{2.10}$$

where $A_{fib} = A_{rt}$ is calculated according to 2.3.3.1, and E_{fib} is the Young modulus of elasticity for fiber only, while the product $A_f \cdot E_f$ refers to the axial stiffness of the FRP after impregnation. The safety factor, α_{fE}, shall be estimated by the FRP manufacturer/supplier and shall be determined on the basis of experimental tests carried out on composite samples corresponding to well-defined volumetric fractions. Such safety factors may take into account both resin type and strengthening geometry; however, quality of installation and nature of support can not be considered for the calibration of α_{fE}. Similarly:

$$A_f \cdot f_f = \alpha_{ff} \cdot A_{fib} \cdot f_{fib} \tag{2.11}$$

Mode 2

Alternatively, FRP manufacturer/supplier may specify characteristic values of the mechanical properties of the system considered applied to the structure to be strengthened, based on experimental investigations carried out on the final system. In this case, it is possible to take into account all the parameters affecting the behavior of the final system, including nature and geometry of the support.

2.3.3.3 Technical data sheets for wet lay-up systems

A specific technical data sheet can not be formulated for wet lay-up systems; therefore, it is necessary to refer to properties of dry fiber. The manufacturers/suppliers shall, however, provide the values of the safety factors, α_{fE} and α_{ff}, with detailed information on the performed experimental testing.

2.3.4 Pre-impregnated systems

Pre-impregnated (*prepreg*) systems are impregnated directly at the manufacturer plant and delivered in rolls. Resin may receive pre-polymerization treatments. A pre-impregnated system is a thin sheet (0.15 mm typical thickness), flexible and moderately sticky, with detaching film (silicon paper or similar) applied on the surfaces to preserve the system itself from external contamination. Storing shall be performed under controlled moisture and temperature conditions; system cross linking shall occur at the time of application by means of thermal treatments. Mechanical characteristics and technical data sheets are identical to wet lay-up systems (2.3.3).

2.4 QUALITY CONTROL

The qualification process of FRP systems and the necessary experimental tests developed by the manufacturer shall be aimed to complete the following:

- Ensure quality of products and compliance with published specified values.
- Provide a statistically significant number of experimental results for physical and mechanical characteristics to be used for design.
- Provide, when possible, data on experimental tests related to long-term behavior of the FRP system.

Qualification tests regard physical and mechanical properties (stiffness and strength) of composite materials, regardless of their particular application. Quality of installation as well as FRP monitoring over time is reported elsewhere.

Two qualification levels for FRP systems can be identified:

- Level 1: where physical and mechanical properties of composite are defined using statistical analysis on a large series of tests.
- Level 2: where long-term physical and mechanical properties of composite are defined.

Both mechanical and physical qualification tests shall be carried out by a certified laboratory provided with the necessary equipment and experience in the characterization of composite materials. Each manufacturer shall provide the mechanical characteristics obtained with statistical analysis, including the characteristic values, percentile, sample mean, sample standard deviations, confidence intervals, and number of samples tested. Suitable safety factors should be employed on the basis of the adopted manufacturing technique.

2.4.1 Level 1: Physical-mechanical properties

Table 2-10 summarizes the most common tests performed on composite materials and the corresponding technical standards (ISO and ASTM).

Table 2-10 – Mechanical and physical characteristics of composites.

Property	Reference standard	
	ISO	ASTM
Mechanical properties		
Tensile Young modulus of elasticity	527-4, 5:1993(E)	D3039-00, D5083-02, D3916-02
Tensile strength	527-1, 4, 5:1993(E)	D3039-00, D5083-02, D3916-02
Tensile strain at failure	527-1, 4, 5:1993(E)	D3039-00, D5083-02, D3916-02
Compression Young modulus of elasticity	14126:1999(E)	D3410
Compressive strength	14126:1999(E)	D3410
Compressive strain at failure	14126:1999(E)	D3410
Creep	899-1:2003(E)	D2990-01
Physical properties		
Density	1183-1:2004(E)	D792-00
Coefficient of thermal expansion	11359-2:1999(E)	E831, D696
Glass transition temperature (matrix)	11357-2:1999(E) (DSC) 11359-2:1999(E) (TMA)	E1356, E1640
Fiber content	11667:1997(E)	D3171, D2584

Samples shall be obtained either from the manufacturing plant or realized in the laboratory using the same technique that will be used in the field. The minimum number of samples shall be at least five for each test to be carried out. Each FRP component may be qualified according to Table 2-11 (fabrics) and Table 2-12 (resins).

Table 2-11 – Mechanical characteristics of fabrics.

Property	ISO Reference standard	Product type
Mechanical properties		
Tensile Young modulus of elasticity	4606:1995(E), 13934-1:1999(E)	fabric
Tensile strength	4606:1995(E), 13934-1:1999(E)	fabric
Tensile strain at failure	4606:1995(E), 13934-1:1999(E)	fabric

Table 2-12 – Mechanical and physical characteristics of resins (matrix and adhesive).

Property	Reference standard		Product type
	ISO	ASTM	
Mechanical properties			
Tensile Young modulus of elasticity	527-1:1993 (E)	D638-02	resin
Tensile strength	527-1:1993 (E)	D638-02	resin
Tensile strain at failure	527-1:1993 (E)	D638-02	resin
Compression Young modulus of elasticity	604:2002(E)	D695	resin
Compressive strength	604:2002(E)	D695	resin
Compressive strain at failure	604:2002(E)	D695	resin
Shear strength	4587:2003(E)	D3163-01	adhesive
Peeling strength	4578:1997(E)	D3167-03	adhesive
Physical properties			
Viscosity	2555:1989(E), 3219:1993(E)	D2196-99	resin
Thyxotropy index		D2196-99	resin
Density	1675:1985(E)	D1217-93	resin
Gel time	9396:1997(E), 2535:2001(E) 15040:1999(E)	D2471-99	resin
Pot life	10364:1993(E)	D1338-99	resin, adhesive
Exothermic peak	12114:1997(E)	D2471-99	resin
Time to full cross-linking	12114:1997(E)	D4473-03	resin
Volumetric shrinkage	12114:1997(E)	D6289-03	resin
Coefficient of thermal expansion	11359-2:1999(E)	E831, D696	resin
Glass transition temperature	11357-2:1999(E) (DSC) 11359-2:1999(E) (TMA)	E1356, E1640	resin

2.4.2 Level 2: Long-term properties

Three different behaviors can be identified over time, according to the following:

- Chemical degradation phenomena.
- Environmental factors (e.g. freeze-thaw cycles).
- Load application: constant (creep) or variable (fatigue).

Tests shall be performed on samples of suitable geometry depending upon the particular test to be carried out. For chemical degradation phenomena and environmental factors, tests are carried out on specimens that have been treated with thermal and/or environmental conditioning for a suitable time. Once conditioning is completed, mechanical and/or physical properties can be determined according to the provisions of ISO (or ASTM) standards (Table 2-10, 2-8, 2-9).

If the variation over time of a particular property is related to the chemical degradation of the material, its value at any given time may be estimated by applying the Arrhenius' procedure (*e.g.*, extrapolating the results of short-term high-temperature tests). It shall be noticed that the results obtained in such a way do not take into account the effect of stresses or environmental factors, such as extended exposure to ultraviolet rays, acids, alkali, salts etc.

To evaluate the strength of FRP systems for particular environmental conditions, specific standards must be referenced. In some instances, a standard developed for the qualification of other materials can be used (*e.g.*, the effect of freeze-thaw cycles on the properties of the composite material for which the same conditioning procedure used for concrete may be adopted). After sample conditioning, the value of the property of interest can be obtained according to the standard reference (Table 2-10, 2-8, 2-9).

It is worth noting that the ISO EN 13687-(1÷5) are particularly useful for aging cycles as well as for the evaluation of the long-term performance of adhesive bond. Creep tests are deemed necessary for long-term behavior of FRP material subjected to a constant load according to ISO 899-1:2003(E); alternatively, the ASTM D2990-01 can be used. Performance under fatigue shall be referred to ISO 13003-2003(E) or ASSTM D 3479-02.

2.5 ACCEPTANCE CRITERIA

FRP materials to be used for structural strengthening shall undergo a series of controls to ensure appropriateness level of mechanical and physical characteristics. For construction materials, specific standards are available for the determination of minimum values of physical and mechanical properties, test procedures, as well as acceptance criteria (6.3). Few items concerning the responsibility of manufacturers, designers, contractors, etc., are briefly reported in the following.

2.5.1 Selection and testing of materials: tasks and responsibilities of professionals

<u>Manufacturers/suppliers</u>:

- Production of FRP materials shall be subjected to a quality control program including fibers, matrices, adhesives, pre-cured composites and other constituents.
- Manufacturers shall provide test certificates for each production lot in compliance with their published literature.
- Whenever possible, each production lot shall be marked to ensure traceability. If this is not the case, each lot shall be accompanied by labels or tags reporting all the information needed for their traceability.
- In addition to mechanical and physical characteristics, manufacturers/suppliers producing complete strengthening systems (fibers, resins, pre-cured or prepreg systems, adhesives and other components) can provide the mechanical properties of the complete system by indicating the type of substrate used. Such values shall be the results of experimental tests carried out either in the laboratory or in-situ.
- Complete strengthening systems certified according to the previous item shall be indicated as <u>Type-A applications</u>, as opposed to <u>Type-B applications</u>:

Type-A applications	Strengthening system with certification of each component as well as the final product to be applied to a given support.
Type-B applications	Strengthening systems certified for each component only.

<u>Designer</u>:

- Designers shall clearly state the quality and characteristics (geometrical, mechanical, and

physical) of the each constituent of the selected strengthening system, specifying, if necessary, the minimum acceptance requirements.
- Designers shall specify acceptance criteria for both selected strengthening systems and field application. In the first case, the designer indicates specimens to be taken from the job site as well as the tests to be performed. In the second case, the designer may indicate quality tests for FRP installation as suggested in 4.8.3 and 5.8.3 for reinforced concrete and masonry structures, respectively.

Contractor/subcontractors:
- Shall obtain the material indicated by the designer through suppliers/manufacturers who guarantee the quality of their products.
- Shall make sure that the products are accompanied by technical data sheets, reporting both mechanical and physical characteristics, and possibly by laboratory test certificates.
- Shall make sure that the products comply with the provisions indicated by the designer; if the material with the indicated requirements is not available, they shall agree with the designer upon viable alternatives.

Construction manager:
- Shall make decisions as to the acceptance of products.
- Shall check the compliance of the material with the designer’s provisions.
- Shall check the origin of the supplied material. Pultruded materials are typically marked by the manufacturer for their identification. Other materials shall have labels or tags with the necessary information for their traceability.
- Shall check the mechanical and physical characteristics of products using the test certificates provided by the manufacturer.
- Based upon the importance of the application, it may require experimental tests to evaluate both quality of materials and compliance with the values provided by the manufacturer. Such tests shall be carried out in laboratories of proven experience and appropriately equipped to characterize FRP materials. Acceptance criteria may be based on the maximum acceptable deviation of results from the values obtained during production. In such a case, it is necessary to ensure that the test procedures are the same and that samples are obtained with the same production techniques. In some cases, tests may be required to evaluate both mechanical and physical properties of un-conditioned and conditioned specimens to take into account temperature and moisture variation.
- For Type-A applications, it is the decision of the construction manager to require acceptance tests for the installed system. For Type-B applications, the construction manager shall require a number of tests to ensure proper quality of both strengthened system and installation procedures as suggested in 4.8.3 and 5.8.3 for reinforced concrete and masonry structures, respectively.

Test laboratories:
- Shall demonstrate experience in the experimental characterization of FRP materials.
- Shall be equipped with appropriate measurement devices and tests instrumentation.
- Shall carry out the experimental tests according to the procedures indicated in the specific standards for FRP materials.
- Shall provide detailed test reports on test instrumentation and results.
- Shall own a quality manual and carry out experimental activities according to EN-ISO17025 “General requirements for the competence of testing and calibration laboratories”.

Inspector:
If the FRP strengthened structure has to be tested, the inspector shall do the following:
- Check the quality of the materials to be in compliance with the manufacturer specifications.
- Verify that all materials used have been accepted by the construction manager.
- Check the results of experimental tests required by the construction manager, if available.

2.6 TRANSPORTATION, STORAGE AND HANDLING

Transportation, storage, and handling of FRP material is fundamental to ensure that properties of each components are not altered and compliance with safety laws and regulations is met.
- Transportation. Each component of the selected FRP system shall be suitably packaged and transported according to safety laws and regulations.
- Storage. To preserve properties of FRP material and ensure compliance with safety laws and regulations, FRP material shall be stored according to the recommendations provided by the supplier/manufacturer. To preserve properties of fibers and resins, storage shall be performed under suitable temperature conditions (suggested range is 10-24 °C), in a dry environment (moisture less than 20 %), unless otherwise suggested by the manufacturer. Laminate and other preformed material may be damaged due to bending or improper stacking. Due to safety reasons, some constituents such as reactive reticulating agents, initiators, catalysts, solvents for surface cleaning, etc., shall be stored according to manufacturer requirements or official standards. Catalysts and initiators (typically peroxides) shall be stored separately from other reagents to avoid any accidental contact leading to premature polymerization. The properties of non-polymerized resins can change over time and are affected by moisture and temperature conditions. The latter can also affect the mixture reactivity and the properties of polymerized resin. Manufacturers shall indicate the storage time (shelf life) that ensures that the properties of thermo-setting resins are maintained. Constituents exceeding their shelf time or suffering degradation or contamination shall not be used. All the constituents deemed unusable shall be disposed of according to the manufacturer specifications as well as the provisions of safety laws and regulations.
- Handling. The manufacturer shall provide the technical data sheet reporting all information relevant to safety (MSDS – Materials Safety Data Sheet) for all the constituents of FRP material. Substances used in combination with thermoset resins are typically hardeners, cross-linkers, initiators (peroxides), and fillers. Some potential dangers when using thermoset resins include:
    - Skin irritation and sensitization.
    - Inhalation of vapours of solvents, diluents, and monomers.
    - Fire or blast risk due to large concentrations of flammable substances in the air or contact with flames or sparks (including cigarettes).
    - Exothermal reactions between reagents that may cause fires or accidents to the personnel involved.
    - Presence of dusts from working or handling FRP material.

  It is therefore necessary to adopt precautions when working with such products or with their constituents. The complexity of thermosetting resins and the associated materials require to all personnel a careful read of labels and MSDS to avoid risks of accidents. For handling of fibers or resins, the use of disposable gloves, work-suits, and protection glasses is suggested. Rubber or plastic gloves shall be solvent-resistant. In the presence of fiber fragments, dusts or solvent vapours, or when mixing and applying resins, respiratory protection devices are needed, as specifically required by FRP manufacturers. The working site shall always be properly ventilated.

3 BASIS OF DESIGN FOR FRP STRENGTHENING

(1)P The subject of this chapter regards FRP strengthening of existing reinforced and prestressed structures as well as masonry structures for which building code requirements are not met. The same principles also apply to existing structures made out of steel and timber, not included in this document.

(2)P The following is assumed:

- The choice and the design of the strengthening system are made by an appropriately qualified and experienced engineer.
- The installation phase is carried out by personnel having the appropriate skills and experience.
- Proper supervision and quality control is provided during installation.
- Construction materials are used as specified in the following.

(3)P The FRP strengthening system shall be designed to have appropriate strength, and meet serviceability and durability requirements. In case of fire, the strength of the selected FRP system shall be adequate to the required period of time.

(4)P The FRP strengthening system shall be located in areas where tensile stresses are to be carried out. FRP composites shall not be relied upon to carry compressive stresses.

3.1 BASIC REQUIREMENTS

(1)P Design of FRP strengthening system shall be performed in compliance with the following principles:

- The risks to which the structure can be subjected shall be accurately identified, removed or attenuated.
- The strengthening configuration shall not be very sensitive to the above risks.
- Strengthening systems shall survive the occurrence of acceptable localized damages.
- Strengthening systems collapsing without warning shall be avoided.

(2)P The above defined basic requirements can be considered met if the following are satisfied:

- Suitable materials are chosen.
- Design is properly performed, with a careful choice of the construction details.
- Quality control procedures are defined for design and construction relevant to the particular project.

(3)P If FRP strengthening concerns structures of historical and monumental interest, a critical evaluation of the strengthening technique is required with respect to the standards for preservation and restoration. The actual effectiveness of the strengthening technique shall be objectively proven, and the adopted solution shall guarantee compatibility, durability, and reversibility.

3.2 DURABILITY REQUIREMENTS

(1)P A strengthening application shall be designed such that deterioration over the design service life of the strengthened structure does not impair its performance below the intended level. Environmental conditions as well as the expected maintenance program need to be carefully addressed. Durability is of fundamental relevance and all the operators involved in the FRP-based strengthening processes shall pursue such requirement (2.5.1).

(2) To ensure durability to FRP strengthened members the following shall be taken into account:

- Intended use of the strengthened structure.
- Expected environmental conditions.
- Composition, properties, and performance of existing and new materials.
- Choice of the strengthening system, its configuration, and construction details.
- Quality of workmanship and the level of control.
- Particular protective measures (*e.g.*, fire or impact).
- Intended maintenance program during the life of the strengthened structure.

(3)P Special design problems (regarding environmental issues, loading, etc.) shall be identified at the design stage to evaluate their relevance for a durability point of view, assign proper values of the conversion factors (3.5), and take the necessary provisions for protection of the adopted FRP system.

(4) When conversion factors for a particular FRP system are not available, any possible reason of degradation of the adopted strengthening configuration shall be accurately estimated. Such estimation can be accomplished through theoretical models, experimental investigations, experience on previous applications, or any combination of the above.

3.3 GENERAL PRINCIPLES OF THE STRENGTHENING DESIGN

3.3.1 General

(1)P Design with FRP composites shall be carried out both in terms of serviceability limit state (SLS) and ultimate limit state (ULS), as defined by the current building code.

(2) Verification of one limit state may be omitted provided that sufficient information is available to prove that it is satisfied by another one.

(3)P Structures and structural members strengthened with FRP shall be designed to have design strength, R_d, at all sections at least equal to the required strength, E_d, calculated for the factored load and forces in such combinations as stipulated in the current building code. The following inequation shall be met:

$$E_d \leq R_d \tag{3.1}$$

(4) The design values are obtained from the characteristic values through appropriate partial factors different for each limit state as indicated in the current building code. Specific partial factors for FRP materials are indicated in this document.

(5) For structural applications using FRP to be carried out on single structural members, when conditions for upgrading are not met, it shall be proven that the adopted strengthening method is such to provide the structure with a significant level of safety with respect to the applied loads. Such applications are defined as "ameliorations." For ameliorations, the level of safety shall be computed for each limit state both prior to and after FRP application.

3.3.2 Partial factors and design loads

(1)P When designing FRP strengthened members, the structure's service life shall be in compliance with the requirements of the current building code. Therefore, the same partial factors for existing materials and the same design loads prescribed by the current building code for new constructions shall be adopted.

3.3.3 Properties of FRP materials

(1)P The properties of FRP materials to be used for strengthening existing structures shall be determined through standardized laboratory tests, such as those indicated in the chapter on materials.

(2)P Properties of the existing materials in the structure to be strengthened shall be obtained from both on-site or laboratory tests and, when available, from any additional source of information (original documents of the project, further documentation obtained subsequently, etc)

(3) Strength and strain properties of FRP materials used for strengthening, as well as those of existing materials (unless otherwise indicated in the current building code) are described by the corresponding characteristic values. The derivation of the characteristic value of a mechanical property through on-site tests shall take into account dispersion of test results, statistical uncertainty associated to the number of performed tests, and previous statistical knowledge.

(4) Only the stiffness parameters (Young modulus of elasticity) of FRP materials as well as those of the existing materials are evaluated through the corresponding average values.

(5) In case of FRP wet lay-up systems, the coefficients α_{fE} and α_{ff} (Section 2.3.3.2), for FRP Young modulus of elasticity and FRP tensile strength shall not be larger than 0.90.

(6) For the generic property of a FRP material, the design value, X_{d}, can be expressed as follows:

$$X_{\mathrm{d}} = \eta \cdot \frac{X_{\mathrm{k}}}{\gamma_{\mathrm{m}}} \tag{3.2}$$

where η is a conversion factor accounting for special design problems (3.5), X_{k} is the characteristic value of the property being considered, and γ_{m} is the partial factor of the material that takes into account the type of application (Table 3-2).

(7) For the generic property of the existing material belonging to the structure to be strengthened, the design value X_{d} can be determined through a series of on-site tests, according to the current building code (dividing the average value by a confidence factor); alternatively, the following equation can be used:

$$X_{\mathrm{d}} = \eta \cdot \frac{X_{\mathrm{k(n)}}}{\gamma_{\mathrm{m}}} = \frac{\eta}{\gamma_{\mathrm{m}}} \cdot m_{\mathrm{X}} \cdot (1 - k_{\mathrm{n}} \cdot V_{\mathrm{X}}) \tag{3.3}$$

where $X_{\mathrm{k(n)}}$ is the characteristic value of the property X resulting from the number n of on-site tests, and m_{X} is the average value of the property X resulting from the number n of tests. The value of k_{n} is provided in Table 3-1 as a function of the number n, while the coefficient of variation V_{X} shall be assumed equal to 0.10 for steel, 0.20 for concrete, and 0.30 for masonry and timber. The value of the conversion factor η is typically assumed equal to 0.85 for concrete and 1.00 for steel, masonry, and timber.

Table 3-1 – Values of k_n for the determination of the characteristic value.

n	1	2	3	4	5	6	8	10	20	30	∞
k_n	2.31	2.01	1.89	1.83	1.80	1.77	1.74	1.72	1.68	1.67	1.64

3.3.4 Design capacity

(1) The design strength, R_d, can be expressed as follows:

$$R_d = \frac{1}{\gamma_{Rd}} \cdot R\left\{X_{d,i}; a_{d,i}\right\} \tag{3.4}$$

In Equation (3.4), $R\{\cdot\}$ is a suitable function for the specific mechanical model being considered (*e.g.*, flexure, shear, etc), and γ_{Rd} is a partial factor covering uncertainties of the assumed model. The arguments of the function $R\{\cdot\}$ are typically the design values $X_{d,i}$ of the materials used for strengthening, or the existing materials, and the nominal values $a_{d,i}$ of the geometrical parameters involved in the model.

(2)P As a rule, the FRP contribution to the strengthened member can not increase its capacity for more than 60 % of that of the unstrengthened member. Such limitation does not apply to exceptional or seismic loads.

3.4 PARTIAL FACTORS

3.4.1 Partial factors γ_m for FRP materials

(1) For ultimate limit states, values to be assigned to the partial factors, γ_m, indicated by γ_f for FRP materials, are suggested in Table 3-2, as a function of the FRP failure mode:

Table 3-2 – Partial factors, γ_m, for materials and products.

Failure mode	Partial factor	Type-A application (1)	Type-B application (2)
FRP rupture	γ_f	1.10	1.25
FRP debonding	$\gamma_{f,d}$	1.20	1.50

(1) Strengthening systems certified according to the indications of Chapter 2 of (Section 2.5).
(2) Strengthening systems uncertified according to the indications of Chapter 2 (Section 2.5).

(2) For serviceability limit state, a value of $\gamma_m = \gamma_f = 1.0$ is assigned to all partial factors, except where otherwise indicated.

3.4.2 Partial factors γ_{Rd} for resistance models

(1) For ULS, values to be assigned to the partial factors γ_{Rd} are reported in Table 3-3.

Table 3-3 – Partial factors γ_{Rd}.

Resistance model	γ_{Rd}
Bending/Combined bending and axial load	1.00
Shear/Torsion	1.20
Confinement	1.10

3.5 SPECIAL DESIGN PROBLEMS AND RELEVANT CONVERSION FACTORS

(1) Hereafter, some reference values to be assigned to the conversion factor η, (3.3.3(6)) that affects both durability and behavior of FRP materials are reported.

3.5.1 Environmental conversion factor η_a

(1)P Mechanical properties (*e.g.*, tensile strength, ultimate strain, and Young modulus of elasticity) of FRP systems degrade under specific environmental conditions such as alkaline environment, moisture, extreme temperatures, thermal cycles, freeze and thaw cycles, and ultraviolet radiations (UV).

(2) Effects of alkaline environment. The water contained in the pores of concrete may cause degradation of the resin and the interface between FRP and support. The damage of the resin due to alkaline environment is typically more dangerous than that due to moisture. The resin shall complete its curing process prior to being exposed to alkaline environment.

(3) Effects of moisture. The main effects of moisture absorption concern the resin; they can be summarized as follows: plasticization, reduction of glass transition temperature, and strength and stiffness (the latter less significant). The absorption of moisture depends on the type of resin, the composition and quality of the laminate, the thickness, the curing conditions, the resin-fiber interface, and the working conditions. In a marine environment, where osmotic effects may cause the presence of air pockets in the resin, it is suggested to use protective coatings.

(4) Effects of extreme temperatures and thermal cycles. The primary effects of temperature concern the viscous response of both resin and composite. As the temperature rises, the Young modulus of elasticity of the resin lowers. If the temperature exceeds the glass transition temperature, the performance of FRP materials significantly decreases. In general, thermal cycles do not have harmful effects on FRP; however, they may cause micro-fractures in systems with high modulus resins. For typical temperature in civil infrastructures, undesired performance can be avoided by choosing a system where the glass transition temperature is always higher than the maximum operating temperature of the structure or component being strengthened.

(5) Effects of freeze and thaw cycles. In general, exposure to freeze and thaw cycles does not have an impact on FRP performance, whereas it lowers the performance of the resin as well as the fiber-resin interface. For temperatures below 0 °C, polymeric-based resin systems may improve their performance by developing higher strength and stiffness. The effects of the degradation induced by freeze and thaw cycles may be magnified by the presence of moisture.

(6) Effects of ultraviolet radiations (UV). Ultraviolet radiations rarely degrade the mechanical performance of FRP-based systems, although this may cause some resins to have a certain degree of brittleness and surface erosion. In general, the most harmful effect linked to UV exposure is the penetration of moisture and other aggressive agents through the damaged surface. FRP-based systems may be protected from such damages by adding fillers to the resin or by providing appropriate coatings.

(7) Table 3-4 summarizes the values to be assigned to the environmental conversion factor η_a, depending upon fiber/resin type and exposure conditions. Such values represent conservative estimates based on the durability of different fiber types. Values as reported in the table may be increased by 10 % (however, $\eta_a \leq 1$ shall always be satisfied) whenever protective coatings are used. Such coatings need to be maintained on the strengthened structure for its entire life and need to be

experimentally tested and proven to be effective in protecting the FRP system from the environmental exposure.

Table 3-4 – Environmental conversion factor η_a for different exposure conditions or FRP systems.

Exposure conditions	Type of fiber/resin	η_a
Internal	Glass/Epoxy	0.75
	Aramid/Epoxy	0.85
	Carbon/Epoxy	0.95
External	Glass/Epoxy	0.65
	Aramid/Epoxy	0.75
	Carbon/Epoxy	0.85
Aggressive environment	Glass/Epoxy	0.50
	Aramid/Epoxy	0.70
	Carbon/Epoxy	0.85

3.5.2 Conversion factors for long-term effects η_l

(1)P Mechanical properties (*e.g.*, tensile strength, ultimate strain, and Young modulus of elasticity) of FRP-based systems degrade due to creep, relaxation, and fatigue.

(2) Effects of creep and relaxation. For FRP-based systems, creep and relaxation depend on both properties of resins and fibers. Typically, thermosetting resins (unsaturated polyesters, vinyl esters, epoxy, and phenolic resins) are less viscous than thermo-plastic resins (polypropylenes, nylon, polycarbonates, etc.). Since the presence of fibers lowers the resin creep, such phenomena are more pronounced when the load is applied transversely to the fibers or when the composite has a low volume ratio of fibers. Creep may be reduced by ensuring low serviceability stresses. CFRP, AFRP, and GFRP systems are the least, moderately, and most prone to creep rupture, respectively.

(3) Fatigue effects. The performance of FRP systems under fatigue conditions need to be taken into consideration as well. Such performance depends on the matrix composition and, moderately, on the type of fiber. In unidirectional composites, fibers usually have few defects; therefore, they can effectively delay the formation of cracks. The propagation of cracks is also prevented by the action of adjacent fibers.

(4) To avoid failure of FRP strengthened members under continuous stress or cyclic loading, values of the conversion factor for long term effects, η_l, are suggested in Table 3-5. In case of combined continuous or cyclic loading, the overall conversion factor may be obtained as the product of the pertaining conversion factors.

Table 3-5 – Conversion factor for long-term effects η_l for several FRP systems at SLS.

Loading mode	Type of fiber/resin	η_l
Continuous (creep and relaxation)	Glass/Epoxy	0.30
	Aramid/Epoxy	0.50
	Carbon/Epoxy	0.80
Cyclic (fatigue)	All	0.50

3.5.3 Impact and explosive loading

(1) The behavior of FRP systems subjected to impact or explosive loading is not completely understood yet. First indications suggest choosing AFRP (more resistant to impact) and/or GFRP systems rather than CFRP.

3.5.4 Vandalism

(1)P FRP composite materials are particularly sensitive to cuts and incisions produced by cutting tools.

(2) Particular protection systems need to be carried out for FRP strengthened members open to the public where vandalism could be an issue. The safety of the structural member shall be checked, assuming that the FRP system is no longer in place. Design shall be verified using the combination for quasi-permanent loads while the material partial factors at ULS shall be considered for exceptional loading.

3.6 STRENGTHENING LIMITATIONS IN CASE OF FIRE

(1)P FRP materials are particularly sensitive to high temperatures that may take place during fire. When the room temperature exceeds the glass transition temperature of the resin (or the melting temperature in the case of semi-crystalline materials), both strength and stiffness of the installed FRP system are reduced. In case of FRP applied as external reinforcement to concrete or masonry members, exposure to high temperature produces a fast degradation of the bond between the FRP system and the support. As a result, degradation of the strengthening effectiveness and debonding of FRP composite may take place.

(2) With regard to fire exposure, mechanical properties of FRP strengthened members may be improved by increasing the thickness of protective coatings. It is suggested to employ coating capable of reducing the spreading of flames as well as smoke production. It is also recommended to employ protective coating systems provided with official certificates. Further specifications on the application of protective coating systems are reported in sections 4.8.2.3 and 5.8.2.3.

(3)P In order to prevent collapse of the FRP strengthened structure, as long as further information on the actual behaviour of coatings and resins under fire exposure is not available, it is recommended to keep low the FRP contribution to the member capacity.

(4) It is suggested that the combination for exceptional loading (fire), as defined by the current building code, takes as a reference the situations hereafter listed, where the design value of the effect of indirect thermal loading is indicated by the symbol E_{d}.

- Exceptional loading with FRP strengthening still in place ($E_{\mathrm{d}} \neq 0$), when the strengthening system has been designed to withstand fire exposure. Applied loads need to be considered at SLS and load factors in compliance with frequent loading conditions. In this case, all loads acting on the structure for the frequent combination are to be considered. The member capacity, reduced to take into account the duration of fire exposure, shall be computed with the partial factors pertaining to exceptional situations, according to the current building code (for fiber-reinforced composites $\gamma_{\mathrm{f}} = 1$).
- Situation following an exceptional event ($E_{\mathrm{d}} = 0$), when the strengthening system is no longer in place. Applied loads need to be considered for quasi-permanent loading conditions. The member capacity, reduced to take into account the duration of fire exposure, shall be computed with the partial factors pertaining to exceptional situations.

4 STRENGTHENING OF REINFORCED AND PRESTRESSED CONCRETE STRUCTURES

4.1 DEBONDING MECHANISMS

4.1.1 Failure mechanisms due to debonding

(1)P When strengthening reinforced concrete members with FRP composites, the role of bond between concrete and FRP is of great relevance due to the brittleness of the failure mechanism by debonding (loss of adhesion). According to the capacity design criterion, such a failure mechanism shall not precede flexural or shear failure of the strengthened member.

(2)P The loss of adhesion between FRP and concrete may concern both laminates or sheets applied to reinforced concrete beams as flexural and/or shear strengthening. As shown in Figure 4-1, debonding may take place within the adhesive, between concrete and adhesive, in the concrete itself, or within the FRP reinforcement (e.g. at the inteface between two adjacent layers bonded each other) with different fiber inclination angles. When proper installation is performed, because the adhesive strength is typically much higher than the concrete tensile strength, debonding always takes place within the concrete itself with the removal of a layer of material, whose thickness may range from few millimeters to the whole concrete cover.

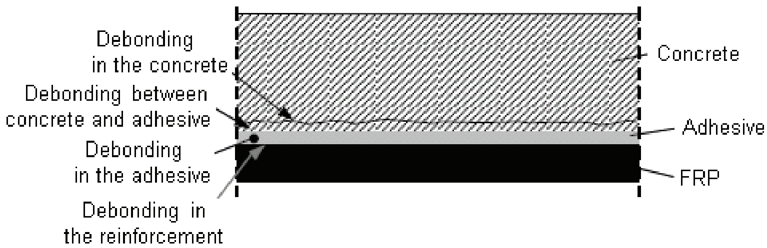


Figure 4-1 – Debonding between FRP and concrete**.**

(3)P Debonding failure modes for laminates or sheets used for flexural strengthening may be classified in the following four categories, schematically represented in Figure 4-2.

- 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)

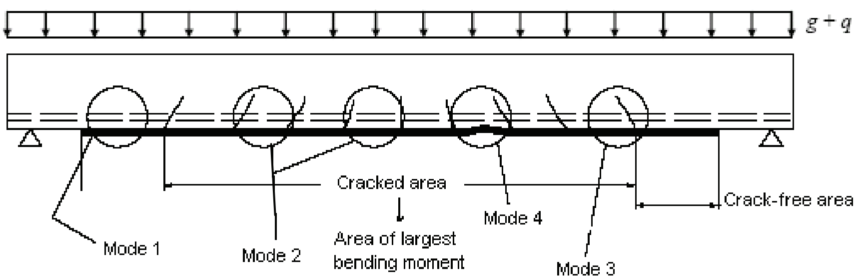


Figure 4-2 – FRP flexural strengthening: debonding failure modes

(4) In the following, reference is made to Modes 1 and 2 only, since they are the most frequent in ordinary situations. To mitigate the risk of occurrence of the remaining failure modes, recom-

mendations on both support control and preparation, as reported in this document (Section 4.8), can be followed.

(5) Further details on failure modes due to debonding and criteria for design are provided in Appendix B.

4.1.2 Fracture energy

(1)P Before any flexural and shear design can take place, the evaluation of the maximum force that may be transferred from the concrete to the FRP, as well as the evaluation of shear and normal stresses at the concrete-FRP interface is required. The former is necessary when designing at ULS; the latter when designing at SLS.

(2)P With reference to a typical bond test as represented in Figure 4-3, 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 bonded length, l_e, is defined as the length that, if exceeded, there would be no increase in the force transferred between concrete and FRP.

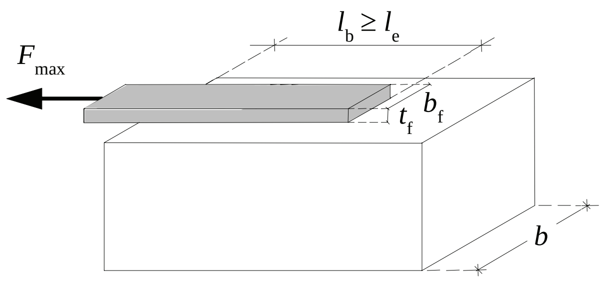


Figure 4-3 – Maximum force transferred between FRP and concrete.

(3) The optimal bonded length, l_e, may be estimated as follows:

$$l_e = \sqrt{\frac{E_f \cdot t_f}{2 \cdot f_{ctm}}} \quad \text{[length in mm]} \tag{4.1}$$

where E_f and t_f are Young modulus of elasticity and thickness of FRP, respectively, and f_{ctm} is the average tensile strength of the concrete.

(4) The specific fracture energy, Γ_{Fk}, of the FRP–concrete interface may be expressed as follows:

$$\Gamma_{Fk} = 0.03 \cdot k_b \cdot \sqrt{f_{ck} \cdot f_{ctm}} \quad \text{[forces in N, lengths in mm]} \tag{4.2}$$

where f_{ck} is the characteristic strength of concrete.

The value provided in Equation (4.2) is intended as a characteristic value (5th percentile); k_b is a geometric coefficient depending on both width, *b*, of the strengthened beam and width, b_f, of the FRP system; and k_b can be written as follows:

$$k_{\mathrm{b}} = \sqrt{\frac{2 - \frac{b_{\mathrm{f}}}{b}}{1 + \frac{b_{\mathrm{f}}}{400}}} \geq 1 \qquad \text{[length in mm]} \tag{4.3}$$

where $b_{\mathrm{f}}/b \geq 0.33$ (if $b_{\mathrm{f}}/b < 0.33$, the value for k_{b} corresponding to $b_{\mathrm{f}}/b = 0.33$ is adopted).

4.1.3 Ultimate design strength for laminate/sheet end debonding (mode 1)

(1) For laminate/sheet end debonding assuming that the provided bond length is equal to or larger than the optimal bonded length, the ultimate design strength, f_{fdd}, can be calculated as follows:

$$f_{\mathrm{fdd}} = \frac{1}{\gamma_{\mathrm{f,d}} \cdot \sqrt{\gamma_{\mathrm{c}}}} \cdot \sqrt{\frac{2 \cdot E_{\mathrm{f}} \cdot \Gamma_{\mathrm{Fk}}}{t_{\mathrm{f}}}} \tag{4.4}$$

where $\gamma_{\mathrm{f,d}}$ is the partial factor indicated in Table 3-2 (Section 3.4.1) and γ_{c} is the partial factor for concrete.
Equation (4.4) may be used for:

- Flexural strengthening (4.2.2.5)
- Shear strengthening (4.3.3.2)

(2) For bond lengths, l_{b}, shorter than the optimal bonded length, l_{e}, the ultimate design strength shall be reduced according to the following equation:

$$f_{\mathrm{fdd,rid}} = f_{\mathrm{fdd}} \cdot \frac{l_{\mathrm{b}}}{l_{\mathrm{e}}} \cdot \left(2 - \frac{l_{\mathrm{b}}}{l_{\mathrm{e}}}\right) \tag{4.5}$$

(3) When special anchoring devices are used (FRP transverse bars, U-wrap with FRP sheets, etc.), the maximum design strength must be evaluated directly with ad-hoc experimental tests.

4.1.4 Ultimate design strength for intermediate debonding (mode 2)

(1)P To prevent failure from intermediate debonding mechanism, the stress variation $\Delta\sigma_{\mathrm{f}}$ in the FRP system between two subsequent cracks should not exceed the limit $\Delta\sigma_{\mathrm{R}}$. The latter value typically depends on the characteristics of bond between concrete and FRP (as defined in Appendix B), the distance between transverse cracks in the concrete, and the level of stress, σ_{f}, in the FRP reinforcement.

(2) Alternatively, a simplified procedure may be used. The maximum strength calculated in the FRP system at ULS shall be less than $f_{\mathrm{fdd,2}}$:

$$f_{\mathrm{fdd,2}} = k_{\mathrm{cr}} \cdot f_{\mathrm{fdd}} = \frac{k_{\mathrm{cr}}}{\gamma_{\mathrm{f,d}} \cdot \sqrt{\gamma_{\mathrm{c}}}} \cdot \sqrt{\frac{2 \cdot E_{\mathrm{f}} \cdot \Gamma_{\mathrm{Fk}}}{t_{\mathrm{f}}}} \tag{4.6}$$

where k_{cr} can be taken equal to 3.0 if specific data are not available.

The corresponding value of the design strain, $\varepsilon_{\mathrm{fdd}}$, in the FRP system can be calculated as follows:

$$\varepsilon_{\mathrm{fdd}} = \frac{f_{\mathrm{fdd,2}}}{E_{\mathrm{f}}} \tag{4.7}$$

4.1.5 Interfacial stress for serviceability limit state

(1)P For FRP-strengthened beams, stress concentrations (shear and normal stresses) take place at the interface between concrete and FRP close to transverse cracks in the concrete or to the ends of FRP reinforcement. Stress concentrations may cause cracks at the interface.

(2)P Under service conditions, interfacial cracks need to be avoided, especially when the strengthened member can be subjected to fatigue and freeze/thaw cycles. For the analysis, a linear elastic behavior for both concrete and steel can be considered.

(3) For rare or frequent loading conditions, the "equivalent" shear stress, $\tau_{\mathrm{b,e}}$, at the adhesive-concrete interface, shall be smaller than the design bond strength, f_{bd}, between FRP reinforcement and concrete according to the following equation:

$$\tau_{\mathrm{b,e}} \leq f_{\mathrm{bd}} \tag{4.8}$$

(4) The "equivalent" shear stress, $\tau_{\mathrm{b,e}}$, may be defined as follows:

$$\tau_{\mathrm{b,e}} = k_{\mathrm{id}} \cdot \tau_{\mathrm{m}} \tag{4.9}$$

where:

- k_{id} represents a coefficient (≥ 1) accounting for shear and normal stresses close to the anchorage ends (Appendix B):

$$k_{\mathrm{id}} = \left(k_{\sigma}^{1.5} + 1.15 \cdot k_{\tau}^{1.5}\right)^{2/3} \tag{4.10}$$

- and the coefficients k_{σ} and k_{τ} can be calculated as follows:

$$k_{\sigma} = k_{\tau} \cdot \beta \cdot t_{\mathrm{f}} \tag{4.11}$$

$$k_{\tau} = 1 + \alpha \cdot a \cdot \frac{M_{(\mathrm{z=a})}}{V_{(\mathrm{z=a})} \cdot a} \tag{4.12}$$

- $M_{(\mathrm{z=a})}$ is the moment acting on the section where FRP strengthening ends (Figure 4-4).

- $V_{(\mathrm{z=a})}$ is the shear force acting on the section where FRP strengthening ends.

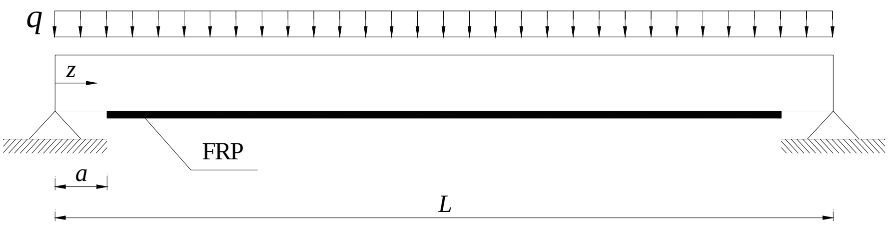


Figure 4-4 – Definition of beam geometrical parameters.

- α and β are two elastic parameters depending on the characteristics of the interface and the FRP:

$$\alpha = \sqrt{\frac{K_1}{E_f \cdot t_f}} \tag{4.13}$$

$$\beta = \left(\frac{b_f \cdot 2.30 \cdot K_1}{4 \cdot E_f \cdot I_f}\right)^{1/4} \tag{4.14}$$

where E_f, t_f, b_f, and I_f, represent Young modulus of elasticity, thickness, width, and moment of inertia (with respect to its centroidal axis parallel to b_f) of the FRP reinforcement, respectively, and K_1 is the slope of the initial linear branch of τ-s diagram (Appendix B). K_1 is assumed to be equal to:

$$K_1 = \frac{1}{t_a/G_a + t_c/G_c} \tag{4.15}$$

where G_a and G_c are the adhesive and concrete shear modulus, respectively, t_a is the nominal thickness of the adhesive, and t_c is the effective thickness of the concrete (typical values for t_c are $20 \div 30$ mm).

- τ_m is the average shear stress according to the Jourawski theory:

$$\tau_m = \frac{V_{(z=a)} \cdot t_f \cdot (h - x_e)}{I_c / n_f} \tag{4.16}$$

- x_e and I_c are the distances from the extreme compression fiber to the neutral axis and the moment of inertia of the transformed section, respectively.

- $n_f = E_f / E_c$ is the modular ratio (E_c is the Young modulus of elasticity for the concrete corresponding to the considered load combination).

(5) If end anchorage is provided using U-wrap, the effect of normal stresses can be neglected and the coefficient k_σ in Equation (4.11) can be assumed equal to zero.

(6) The design bond strength, f_{bd}, is a function of the characteristic tensile strength of the concrete, f_{ctk}, as follows:

$$f_{\mathrm{bd}} = k_{\mathrm{b}} \cdot \frac{f_{\mathrm{ctk}}}{\gamma_{\mathrm{b}}} \tag{4.17}$$

where the partial factor γ_{b} is taken equal to 1.0 for rare loading combinations, 1.2 for frequent loading combinations, and the geometric coefficient $k_{\mathrm{b}} \geq 1$ can be obtained from Equation (4.3).

(7) When computing anchorage stresses at SLS, reference can be made to the stress corresponding to the increased load following the application of FRP reinforcement.

4.2 FLEXURAL STRENGTHENING

4.2.1 Introduction

(1)P Flexural strengthening is necessary for structural members subjected to a bending moment larger than the corresponding flexural capacity. Only the case of uniaxial bending (*e.g.*, when the moment axis coincides with a principal axis of inertia of the cross-section, in particular a symmetry axis) is addressed here.

(2)P Flexural strengthening with FRP materials may be carried out by applying one or more laminates or one or more sheets to the tension side of the member to be strengthened.

4.2.2 Analysis at ultimate limit state

4.2.2.1 Introduction

(1)P Flexural design at ULS of FRP strengthened members requires that both flexural capacity, M_{Rd}, and factored ultimate moment, M_{Sd}, satisfy the following inequation:

$$M_{\mathrm{Sd}} \leq M_{\mathrm{Rd}} \tag{4.18}$$

(2)P ULS analysis of RC members strengthened with FRP relies on the following fundamental hypotheses:

- Cross-beam sections, perpendicular to the beam axis prior to deflection, remain still plane and perpendicular to the beam axis after deflection.
- Perfect bond exists between FRP and concrete, and steel and concrete.
- Concrete does not react in tension.
- Constitutive laws for concrete and steel are accounted for according to the current building code.
- FRP is considered a linear-elastic material up to failure.

(3) FRP strengthening is effective for low steel reinforcement ratios (*e.g.*, steel yielded at ultimate); the rules hereafter reported refer exclusively to this situation.

(4)P It is assumed that flexural failure takes place when one of the following conditions is met:

- The maximum concrete compressive strain, $\varepsilon_{\mathrm{cu}}$, as defined by the current building code is reached.
- The maximum FRP tensile strain, $\varepsilon_{\mathrm{fd}}$, is reached; $\varepsilon_{\mathrm{fd}}$ can be calculated as follows:

$$\varepsilon_{fd} = \min\left\{\eta_a \cdot \frac{\varepsilon_{fk}}{\gamma_f}, \varepsilon_{fdd}\right\} \tag{4.19}$$

where ε_{fk} is the characteristic strain at failure of the adopted strengthening system; γ_f and η_a are the coefficients defined in Table 3-2 and in Table 3-4, respectively; ε_{fdd} is the maximum strain due to intermediate debonding as defined in Section 4.1.4; and E_f is the FRP Young modulus of elasticity (generally the minimum value in Equation (4.19) corresponds to ε_{fdd}).

(5)P The shear capacity of the strengthened member shall be larger than the shear demand corresponding to the examined case. If deemed necessary, shear capacity shall be increased according to the provisions of Section 4.3.

(6)P Because a member strengthened with FRP is generally loaded at the time of FRP application, the existing strain in the structure before FRP strengthening takes place shall be taken into account.

4.2.2.2 Strain in the structure prior to FRP strengthening

(1)P When FRP strengthening is applied to a member subjected to existing loads, the initial strain shall be evaluated when the applied moment due to the existing loads, M_o, is larger than the member cracking moment. If this is not the case, initial strain can usually be neglected.

(2)P Design is carried out assuming a linear elastic behavior for all materials of the member being analyzed.

(3)P Strains to be taken into account are those on compression, ε_{co}, and tension side, ε_o, where the FRP strengthening system is applied. They can be obtained on the basis of the linearity of the strain diagram as a function of the mechanical and geometrical properties of the cross section.

4.2.2.3 Flexural capacity of FRP-strengthened members

(1)P Member flexural capacity is carried out according to Section 4.2.2.1.

(2) The flexural analysis of FRP strengthened members can be carried out by using strain compatibility and force equilibrium. The stress at any point in a member must correspond to the strain at that point; the internal forces must balance the external load effects.

(3) With reference to the simple situation shown in Figure 4-5, two types of failure can be observed, depending on whether the ultimate FRP strain (region 1) or the ultimate concrete compressive strain (region 2) is reached.

(4) When design falls in region 1, failure is due to the rupture of the FRP system. Any strain diagram corresponding to such a failure mode has its fixed point at the value of the FRP strain, ε_{fd}, as defined in Equation (4.19).
The distribution of strains over the depth of the member must be linear to satisfy the fundamental hypotheses presented earlier in this chapter. They shall be calculated as follows:

- (FRP) $\varepsilon_f = \varepsilon_{fd}$

- (concrete in compression) $\varepsilon_{\mathrm{c}} = (\varepsilon_{\mathrm{fd}} + \varepsilon_{\mathrm{o}}) \cdot \dfrac{x}{(h-x)} \leq \varepsilon_{\mathrm{cu}}$
- (steel in compression) $\varepsilon_{\mathrm{s2}} = (\varepsilon_{\mathrm{fd}} + \varepsilon_{\mathrm{o}}) \cdot \dfrac{x-d_2}{(h-x)}$
- (steel in tension) $\varepsilon_{\mathrm{s1}} = (\varepsilon_{\mathrm{fd}} + \varepsilon_{\mathrm{o}}) \cdot \dfrac{d-x}{(h-x)}$

where all the symbols are shown in Figure 4-5, and $\varepsilon_{\mathrm{cu}}$ represents the ultimate concrete compressive strain. In particular, the position x of the neutral axis is identified by its distance from the extreme compression fiber of the member cross-section.

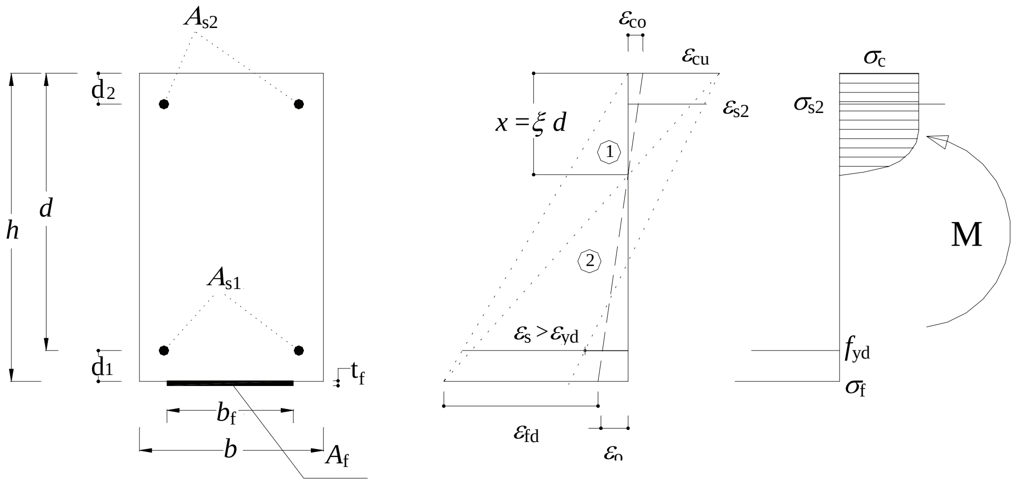


Figure 4-5 – Failure mode of a RC member Strengthened with FRP

Checking the value of the steel strain at ultimate is usually unnecessary because typical FRP systems present values of ultimate strain significantly smaller compared to that of steel. If the ultimate strain for steel according to the current building code is exceeded, this shall be taken into account when computing the position of the neutral axis and therefore the member flexural capacity.

(5) When design falls in region 2, failure is due to concrete crushing (strain equal to $\varepsilon_{\mathrm{cu}}$) with yielding of steel in traction, while FRP strain has not reached its ultimate value. The distribution of strains over the depth of the member must be linear to satisfy the fundamental hypotheses presented earlier in this chapter. They shall be calculated as follows:

- (FRP) $\varepsilon_{\mathrm{f}} = \dfrac{\varepsilon_{\mathrm{cu}}}{x} \cdot \left(h - x\right) - \varepsilon_0 \leq \varepsilon_{\mathrm{fd}}$
- (concrete in compression) $\varepsilon_{\mathrm{c}} = \varepsilon_{\mathrm{cu}}$
- (steel in compression) $\varepsilon_{\mathrm{s2}} = \varepsilon_{\mathrm{cu}} \cdot \dfrac{x-d_2}{x}$
- (steel in tension) $\varepsilon_{\mathrm{s1}} = \varepsilon_{\mathrm{cu}} \cdot \dfrac{d-x}{x}$

All symbols used have been previously shown in Figure 4-5.

(6) For both failure modes, the position x of the neutral axis is computed by means of the translational equilibrium equation along the beam axis as follows:

$$0 = \psi \cdot b \cdot x \cdot \overline{f_{cd}} + A_{s2} \cdot \sigma_{s2} - A_{s1} \cdot f_{yd} - A_f \cdot \sigma_f \tag{4.20}$$

where $\overline{f_{cd}}$ is equal to the design concrete compressive strength, f_{cd}, suitably reduced if it is necessary.
The flexural capacity, M_{Rd}, of the strengthened member can be calculated using the rotational equilibrium equation as follows:

$$M_{Rd} = \frac{1}{\gamma_{Rd}} \cdot \left[\psi \cdot b \cdot x \cdot \overline{f_{cd}} \cdot (d - \lambda \cdot x) + A_{s2} \cdot \sigma_{s2} \cdot (d - d_2) + A_f \cdot \sigma_f \cdot d_1 \right] \tag{4.21}$$

where the partial factor, γ_{Rd}, should be assumed equal to 1.00 (Table 3-3, Section 3.4.2).

In Equations (4.20) and (4.21), the non-dimensional coefficients ψ and λ represent the resultant of the compression stresses and its distance from the extreme compression fiber, respectively, divided by $b \cdot x \cdot \overline{f_{cd}}$ and by *x*, respectively.

(7) If the steel is in its elastic phase, stresses may be obtained by multiplying the corresponding strain by the Young modulus of elasticity; otherwise they can be assumed equal to the yielding stress, f_{yd}. In both regions 1 and 2, the strain exhibited by tension steel bars is always larger than ε_{yd}.

(8) Because FRP materials have a linear elastic behavior up to failure, their stress shall be taken as the product of the Young modulus of elasticity times the calculated strain.

(9) To avoid the occurrence that existing steel reinforcing bars remain elastic at ultimate, the non-dimensional coefficient $\xi = x/d$ shall not exceed the limit value, ξ_{lim}, provided as follows:

$$\xi_{lim} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{yd}} \tag{4.22}$$

4.2.2.4 Flexural capacity of FRP-strengthened members subjected to bending moment and axial force

(1)P The principles introduced in Section 4.2.2.1, from (1)P through (5)P, still apply. However, the presence of the axial force, N_{Sd}, needs to be taken into account when determining the member flexural capacity, M_{Rd}.

(2)P Strengthening effectiveness close to the beam-column intersections shall be ensured with suitable construction details. In addition, longitudinal fibers subjected to combined axial load and bending shall be properly anchored to avoid fiber delamination and spalling of concrete. When evaluating the maximum strain using Equation (4.19), the value corresponding to the first term in brackets shall be considered for design.

(3) Items (2) to (9) of Section 4.2.2.3 still apply to this case; the left-hand side term of Equation (4.20) is no longer equal to zero and shall be considered equal to the design factored axial load, N_{Sd}.

(4) Alternatively, it shall be possible to evaluate the member flexural capacity due to combined axial load and bending according to the provisions of Appendix C.

4.2.2.5 Failure by laminate/sheet end debonding

(1)P Laminate/sheet end debonding depends on a number of parameters such as location and type of cracks (shear or flexural cracks), uneven concrete surface, as well as stress concentration close to the anchorage zone.

(2) The maximum distance *a* (Figure 4-6) to avoid debonding shall be computed by equating the ultimate design strength from Equation (4.4), for $l_\mathrm{b} \geq l_\mathrm{e}$, to the stress calculated at ULS in the FRP system, at a distance $a + l_\mathrm{b}$ from the support. If the available bond length is $l_\mathrm{b} < l_\mathrm{e}$, Equation (4.4) shall be replaced with Equation (4.5).

(3) When the end of the FRP system is close to the member supports, where shear forces may induce inclined cracking, the moment to be taken into account in item (2) shall be evaluated by increasing the design moment as follows:

$$M = V_\mathrm{Sd} \cdot a_1 \tag{4.23}$$

where V_Sd is the factored shear force, $a_1 = 0.9 \cdot d \cdot (1 - \cot\alpha)$, α is the angle of existing transverse steel reinforcement, and *d* is the member effective depth (Figure 4-6).

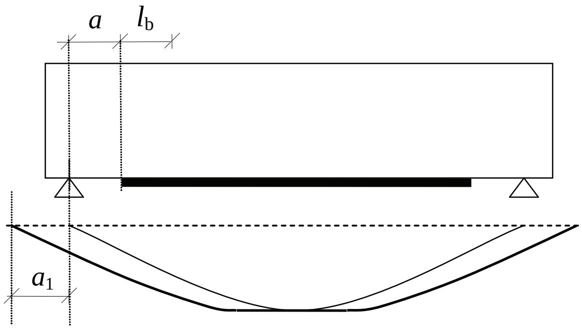


Figure 4-6 – Shifting of bending moment diagram.

(4) When special anchoring devices used to avoid FRP debonding at the termination points are employed, it shall be permitted to neglect provisions of 4.1.3. Such anchoring devices need to be guaranteed based on proper experimental tests. Experimental tests need to be conducted for the material intended for such application (adhesives and reinforcing fibers), for the specific system used (transverse bars embedded in concrete, U-wrap with FRP sheets, etc.), for construction procedures as recommended by the manufacturer, for surface preparation, and for the expected environmental conditions.

4.2.3 Analysis at serviceability limit state

4.2.3.1 Design assumptions

(1)P This section deals with the following serviceability limit states:

- Stress limitation (Section 4.2.3.2)

- Deflection control (Section 4.2.3.3)
- Crack control (Section 4.2.3.4)

Other serviceability limit states may be relevant in particular situations, even though they are not listed in this document.

(2)P At SLS the following items need to be checked:

- Stresses need to be controlled to avoid yielding of tensile steel and creep phenomena in both concrete and FRP.
- Deflections should not attain excessive values such as to prevent the normal use of the structure, induce damage to non-structural members, and cause psychological disturbance to the users.
- Excessive cracking could significantly reduce the durability of structures, their functionality, their aspect, and decrease bond performance of the FRP-concrete interface.

(3)P Design at SLS can be carried out considering all materials having a linear-elastic behavior for both uncracked and cracked transformed section conditions. Existing strain at the time of FRP installation shall be accounted for. The principle of superposition can be used for design. Design assumptions are as follows:

- Linear-elastic behavior of all materials.
- Cross-beam sections, perpendicular to the beam axis prior to deflection, remain still plane and perpendicular to the beam axis after deflection.
- Perfect bond exists between steel and concrete, and concrete and FRP.

(4)P The first assumption allows one to assume a constant value for the Young modulus of elasticity of each material. The second implies the linearity of the strain diagram. The third, along with the first, allows the definition of the modular ratios $n_s = \sigma_s / \sigma_c = E_s / E_c$ and $n_f = \sigma_f / \sigma_c = E_f / E_c$. Such modular ratios are used to transform the actual beam into an all-concrete beam. The stesses in the concrete can be deduced by the strains multiplying these ones by the Young modulus of elasticity, E_c.The actual values of stresses in the steel and in the FRP are n_s or n_f times, respectively, those evaluated in the concrete adjacent fibers.
The values of the modular ratios shall be set to account for creep as well as short and long-term conditions.

(5)P When designing at SLS it is necessary to evaluate the position of the neutral axis as well as the value of the moment of inertia for both cracked and uncracked conditions prior to and after the installation of the FRP strengthening system.

(6)P If deemed necessary, service stress due to thermal loads, creep, shrinkage, etc., shall be added to the stress given by the applied loads.

4.2.3.2 Stress limitation

(1)P Stress at service in the FRP system, computed for the quasi-permanent loading condition, shall satisfy the limitation $\sigma_f \leq \eta \cdot f_{fk}$, where f_{fk} is the FRP characteristic strength at failure and η is the conversion factor as suggested in Section 3.5. Service stress in concrete and steel shall be limited according to the current building code.

(2) Assuming that M_o is the bending moment acting on the member prior to FRP strengthening, and assuming that M_1 is the bending moment acting after FRP strengthening, the stress due to the

combined moment $M = M_o + M_1$ can be evaluated as follows:

- Stress in the concrete $\sigma_c = \sigma_{c0} + \sigma_{c1}$ $\sigma_{c0} = M_0 / W_{0,c}^{s}$ $\sigma_{c1} = M_1 / W_{1,c}^{s}$
- Stress in the steel: $\sigma_s = \sigma_{s0} + \sigma_{s1}$ $\sigma_{s0} = n_s M_0 / W_{0,s}^{i}$ $\sigma_{s1} = n_s M_1 / W_{1,s}^{i}$
- Stress in the FRP: $\sigma_f = n_f M_1 / W_{1,f}^{i}$

where (see Figure 4-5):

- $W_{0,c}^{s} = I_0 / x_0$: modulus of resistance of RC members related to extreme concrete compression fiber
- $W_{0,s}^{i} = I_0 / (d - x_0)$: modulus of resistance of RC members related to tension steel
- $W_{1,c}^{s} = I_1 / x_1$: modulus of resistance of RC strengthened members related to extreme concrete compression fiber
- $W_{1,s}^{i} = I_1 / (d - x_1)$: modulus of resistance of RC strengthened members related to tension steel
- $W_{1,f}^{i} = I_1 / (H - x_1)$: modulus of resistance of RC strengthened members related to FRP system

When the existing applied moment, M_0, is such to produce cracking in the concrete member, neutral axis determination as well as values of the moment of inertia I_0 and I_1 shall be calculated with reference to cracked transformed section for both unstrengthened and strengthened conditions. Modular ratios used for design shall also take into account shrinkage and creep of concrete depending upon the performed short or long term analysis.

4.2.3.3 Deflection control

(1)P Deflections exhibited by FRP strengthened structures shall comply with current building code requirements.

(2)P The adopted deflection model shall simulate the real behavior of the structure. If deemed necessary, cracking shall be accounted for.

(3)P The adopted deflection model should take into account the following:

- Creep and shrinkage of concrete.
- Concrete stiffening between cracks.
- Existing cracks prior to FRP strengthening.
- Thermal loads.
- Static and/or dynamic loads.
- Appropriate concrete Young modulus of elasticity depending upon aggregate type and concrete curing at the time of loading.

(4)P The superposition principle shall not be applied if deflections are computed using non-linear analysis.

(5) Deflections for FRP strengthened beams can be performed by integration of the curvature diagrams. Such diagrams can be computed with non-linear analyses by taking into account tension stiffening of concrete. Alternatively, simplified analyses are possible similarly to what used for traditional RC beams, provided that they are supported by suitable experimental basis.

(6)P Existing strain prior to FRP strengthening can be accounted for by adding the deflections related to different phases.

4.2.3.4 Crack control

(1)P At SLS, crack width shall be checked to guarantee a proper use of the structure and to protect steel internal reinforcement.

(2)P Crack width limitations for FRP strengthened structures shall satisfy the requirements of the current building code.

(3) At present no accurate and completely reliable models are available for crack width computation of FRP strengthened concrete structures. Several experiment-based formulations are available in the literature. Such formulations modify the expressions used for traditional RC sections to take into account the presence of the external strengthening. Experimental evidences show that members strengthened with FRP have smaller but closer cracks.

(4)P More refined and accurate models can be adopted when supported by ad-hoc experimental results.

4.2.4 Ductility

(1)P For flexural members, ductility is a measure of the member capability of evolving in the plastic range; it depends on both section behavior and the actual failure modes of the overall structural member.

(2)P For FRP strengthened members, greater ductility is ensured when failure takes place due to crushing of concrete. Collapse due to FRP rupture leads to brittle failures.

(3)P Regardless of the type of cross section, ductility is mainly controlled by the member failure mode. It can be considered totally absent if debonding starts prior to any other failure mechanism.

4.3 SHEAR STRENGTHENING

4.3.1 Introduction

(1)P Shear strengthening is deemed necessary when the applied factored shear force is greater than the corresponding member shear capacity. The latter shall be determined considering the contributions of both concrete and steel transverse reinforcing bars when available.

(2)P Shear strengthening shall be verified at ULS only.

(3)P In the following of this document some specific configurations of FRP shear strengthening are considered. Other solutions are also possible, provided that their effectiveness is proven and their contribution to the shear capacity is quantified.

4.3.2 Strengthening configurations

(1) Shear strengthening is realized by applying one or more layers of FRP material externally bonded to the surface of the member to be strengthened (Figure 4-7). External FRP reinforcement can be applied in a discontinuous fashion, with gaps between following strips, or continuously, with strips next to each other.

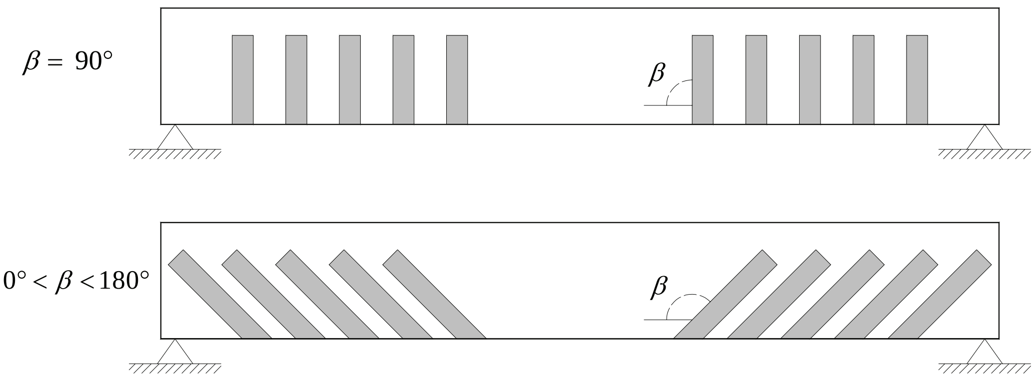


Figure 4-7 – Lateral view of FRP shear strengthening.

(2) Design of FRP strengthening depends on both geometry (FRP thickness, width, and spacing) and the fiber's angle with respect to the longitudinal axis of the member.

(3) Figure 4-8 shows three FRP strengthening configurations: side bonding, U-wrapped, and completely wrapped beams.

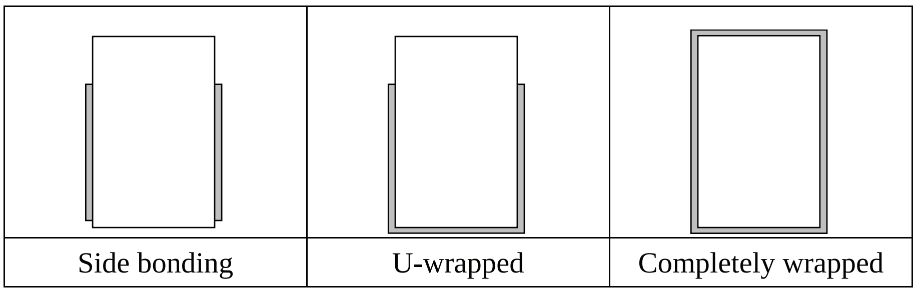


Figure 4-8 – Cross section of FRP strengthened members.

(4) For U-wrap strengthening of rectangular or T-section, 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 longitudinal axis. In such a case, the behavior of U-wrap strengthening can be considered equivalent to that of a completely wrapped member, provided that the effectiveness offered by these devices is proven.

(5) Shear strengthening may also be realized through the installation of FRP bars in dedicated slots made on the outer surface of the member to be strengthened as near-surface mounted reinforcement. Such type of strengthening is not dealt with in this document. If used, its effectiveness shall be supported by experimental evidence.

4.3.3 Shear capacity of FRP strengthened members

4.3.3.1 Shear capacity

(1) Shear capacity of FRP strengthened members can be evaluated as follows:

$$V_{\mathrm{Rd}} = \min\left\{ V_{\mathrm{Rd,ct}} + V_{\mathrm{Rd,s}} + V_{\mathrm{Rd,f}}, V_{\mathrm{Rd,max}} \right\} \tag{4.24}$$

where $V_{\mathrm{Rd,ct}}$ and $V_{\mathrm{Rd,s}}$ represent concrete and steel contribution to the shear capacity according to the current building code, and $V_{\mathrm{Rd,f}}$ is the FRP contribution to the shear capacity to be evaluated as indicated in the following. Shear strength shall not be taken greater than $V_{\text{Rd,max}}$. This last value denotes the ultimate strength of the concretet strut, to be evaluated according to the current building code.

(2) In the case of a RC member with a rectangular cross-section and FRP side bonding configuration, the FRP contribution to the shear capacity, $V_{\text{Rd,f}}$, shall be calculated as follows:

$$V_{\text{Rd,f}} = \frac{1}{\gamma_{\text{Rd}}} \cdot \min\left\{0.9 \cdot d, h_{\text{w}}\right\} \cdot f_{\text{fed}} \cdot 2 \cdot t_{\text{f}} \cdot \frac{\sin\beta}{\sin\theta} \cdot \frac{w_{\text{f}}}{p_{\text{f}}} \tag{4.25}$$

where the partial factor γ_{Rd} shall be assumed equal to 1.20 (Table 3-3, Section 3.4.2), d is the member effective depth, h_{w} is the stem depth, f_{fed} is the effective FRP design strength to be evaluated as indicated in Section 4.3.3.2, t_{f} is the thickness of the adopted FRP system, β is the fibers angle with respect to the member longitudinal axis, θ represents the angle of shear cracks (to be assumed equal to 45° unless a more detailed calculation is made), and w_{f} and p_{f} are FRP width and spacing, respectively, measured orthogonally to the fiber direction (Figure 4-9). For FRP strips installed one next to each other the ratio $w_{\text{f}} / p_{\text{f}}$ shall be set equal to 1.0.

(3) In the same case of a RC member with a rectangular cross-section and U-wrapped or completely wrapped configurations, the FRP contribution to the shear capacity shall be calculated according to the Moersch truss mechanism as follows:

$$V_{\text{Rd,f}} = \frac{1}{\gamma_{\text{Rd}}} \cdot 0.9 \cdot d \cdot f_{\text{fed}} \cdot 2 \cdot t_{\text{f}} \cdot (\cot\theta + \cot\beta) \cdot \frac{w_{\text{f}}}{p_{\text{f}}} \tag{4.26}$$

where all symbols have the meaning highlighted in item (2).

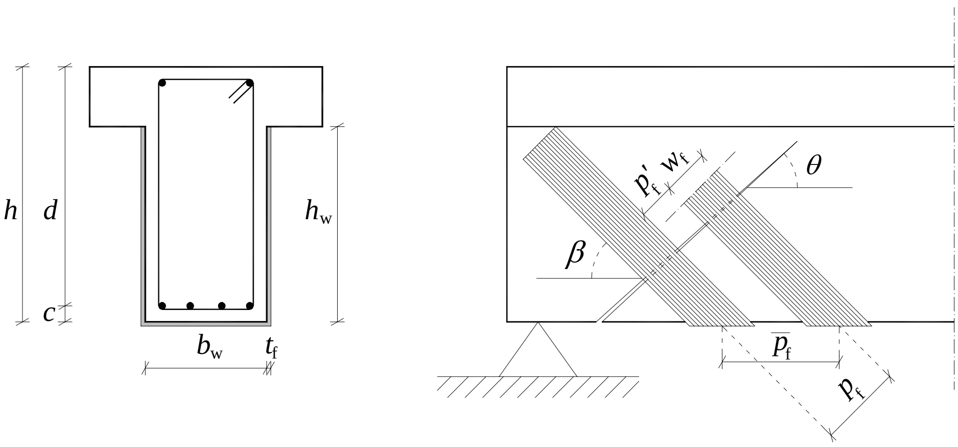


Figure 4-9 – Notation for shear strengthening using FRP strips.

(4) For completely wrapped members having circular cross-sections of diameter D when fibers are placed orthogonal to the axis of the member ($\beta = 90°$), the FRP contribution to the shear capacity, $V_{\text{Rd,f}}$, shall be calculated as follows:

$$V_{\text{Rd,f}} = \frac{1}{\gamma_{\text{Rd}}} \cdot D \cdot f_{\text{fed}} \cdot \frac{\pi}{2} \cdot t_{\text{f}} \cdot \cot\theta \tag{4.27}$$

(5) In all Equations (4.25) to (4.27), it is allowed to replace the term p_f with the term $\overline{p}_f$ measured along the member longitudinal axis, where $p_f = \overline{p}_f \sin\beta$.

4.3.3.2 Effective FRP design strength

(1) Debonding of FRP may be caused by stress concentrations at the concrete-FRP interface close to shear cracks. A simplified procedure to take into account such phenomenon requires the introduction of the "effective FRP design strength" defined as the FRP tensile strength when debonding starts.

(2) For FRP side bonding on a rectangular cross section, the effective FRP design strength can be calculated as follows:

$$f_{\text{fed}} = f_{\text{fdd}} \cdot \frac{z_{\text{rid,eq}}}{\min\{0.9 \cdot d, h_{\text{w}}\}} \cdot \left(1 - 0.6 \cdot \sqrt{\frac{l_{\text{eq}}}{z_{\text{rid,eq}}}}\right)^2 \tag{4.28}$$

where f_{fdd} represents the ultimate design strength, to be evaluated according to Equation (4.4) and item (5), d is the effective depth, and h_{w} is the beam stem depth. Moreover:

$$z_{\text{rid,eq}} = z_{\text{rid}} + l_{\text{eq}}, \quad z_{\text{rid}} = \min\{0.9 \cdot d, h_{\text{w}}\} - l_{\text{e}} \cdot \sin\beta, \quad l_{\text{eq}} = \frac{s_{\text{f}}}{f_{\text{fdd}} / E_{\text{f}}} \cdot \sin\beta \tag{4.29}$$

where l_{e} is the effective bond length according to Equation (4.1), β is the fibers angle with respect to the member longitudinal axis, s_{f} is the ultimate debonding slip assumed to be equal to 0.2 mm (see Appendix B), and E_{f} is the FRP Young modulus of elasticity.

(3) For U-wrap configurations on a rectangular cross section, the effective FRP design strength can be calculated as follows:

$$f_{\text{fed}} = f_{\text{fdd}} \cdot \left[1 - \frac{1}{3} \cdot \frac{l_{\text{e}} \cdot \sin\beta}{\min\{0.9 \cdot d, h_{\text{w}}\}}\right] \tag{4.30}$$

(4) For completely wrapped members having rectangular cross sections, the effective FRP design strength can be calculated as follows:

$$f_{\text{fed}} = f_{\text{fdd}} \cdot \left[1 - \frac{1}{6} \cdot \frac{l_{\text{e}} \cdot \sin\beta}{\min\{0.9 \cdot d, h_{\text{w}}\}}\right] + \frac{1}{2}(\phi_{\text{R}} \cdot f_{\text{fd}} - f_{\text{fdd}}) \cdot \left[1 - \frac{l_{\text{e}} \cdot \sin\beta}{\min\{0.9 \cdot d, h_{\text{w}}\}}\right] \tag{4.31}$$

where f_{fd} is the FRP design strength to be evaluated as in Section 3. Moreover:

$$\phi_{\text{R}} = 0.2 + 1.6 \cdot \frac{r_{\text{c}}}{b_{\text{w}}}, \qquad 0 \le \frac{r_{\text{c}}}{b_{\text{w}}} \le 0.5 \tag{4.32}$$

where r_{c} is the corner radius of the section to be wrapped, and b_{w} is the width of the member. The second term of Equation (4.31) shall be considered only when it is greater than zero.

(5) When calculating f_{fdd} from Equation (4.4), the k_{b} coefficient according to Equation (4.3) shall be considered as follows: for discrete FRP strips application, $b_{\mathrm{f}} = w_{\mathrm{f}}$ and $b = p_{\mathrm{f}}$; for FRP systems installed continuously along the span length of the member it shall be permitted to consider $b_{\mathrm{f}} = b = \min\{0.9 \cdot d, h_{\mathrm{w}}\} \cdot \sin(\theta + \beta)/\sin\theta$ where h_{w} is the beam stem depth.

(6) When special devices used to anchor the end portions of U-wrapped FRP systems are proven to be as effective as the completely wrapped strengthening configuration, the effective FRP design strength can be computed from Equation (4.31). If this is not the case, the effective FRP design strength shall be calculated according to Equation (4.30).

(7) For completely wrapped members having circular cross-section of diameter *D*, when fibers are placed orthogonal to the axis of the member ($\beta = 90°$), the effective FRP design strength shall be calculated as follows:

$$f_{\mathrm{fed}} = E_{\mathrm{f}} \cdot \varepsilon_{\mathrm{f,max}} \tag{4.33}$$

where E_{f} is the FRP Young modulus of elasticity, and $\varepsilon_{\mathrm{f,max}}$ represents FRP maximum allowable strain to be set equal to 0.005, unless a more detailed calculation is made.

4.3.3.3 Limitations and construction details

(1) For U-wrapped and completely wrapped configurations, a minimum 20 mm radius shall be provided when FRP sheets are installed around outside corners.

(2) For external FRP reinforcement in the form of discrete strips, strips width, w_{f} (mm), and center-to-center spacing between strips, p_{f} (mm), shall not exceed the following limitations, respectively: $50\ \mathrm{mm} \le w_{\mathrm{f}} \le 250\,\mathrm{mm}$, and $w_{\mathrm{f}} \le p_{\mathrm{f}} \le \min\{0.5 \cdot d, 3 \cdot w_{\mathrm{f}}, w_{\mathrm{f}} + 200\,\mathrm{mm}\}$.

4.4 TORSIONAL STRENGTHENING

4.4.1 Introduction

(1)P Strengthening for torsion is deemed necessary when the applied factored torsional moment is greater than the corresponding torsional capacity. The latter shall be determined considering the contributions of both concrete and steel transverse reinforcing bars when available.

(2)P Torsional strengthening shall be verified at ULS only.

(3)P In the following of this document some specific configurations of FRP torsional strengthening are considered. Other solutions are also possible, provided that their effectiveness is proven and their contribution to the shear capacity is quantified.

4.4.2 Strengthening configurations

(1) Strengthening for torsion is realized by applying one or more layers of FRP material externally bonded to the surface of the member to be strengthened (Figure 4-7). External FRP reinforcement can be applied in a discontinuous fashion with gaps between following strips, or continuously with strips next to each other.

(2) Design of FRP reinforcement depends on FRP thickness, width, and spacing. Fibers shall be arranged with an angle $\beta = 90°$ with respect to the longitudinal axis of the member.

(3) FRP shall be placed around the cross section as a completely wrapped system only (Figure 4-8).

(4) Strengthening for torsion may also be realized through the installation of FRP bars in dedicated slots made on the outer surface of the member to be strengthened as near-surface mounted reinforcement. Such type of strengthening is not dealt with in this document. If used, its effectiveness shall be supported by experimental evidence.

4.4.3 Torsional capacity of FRP strengthened members

(1)P The following applies to prismatic members where an ideal ring-shaped resisting area can be identified.

4.4.3.1 Torsional capacity

(1) Torsional capacity of FRP strengthened members can be evaluated as follows:

$$T_{\mathrm{Rd}} = \min\left\{ T_{\mathrm{Rd,s}} + T_{\mathrm{Rd,f}}, T_{\mathrm{Rd,max}} \right\} \tag{4.34}$$

where $T_{\mathrm{Rd,s}}$ is the existing steel contribution to the torsional capacity according to the current building code, and $T_{\mathrm{Rd,f}}$ is the FRP contribution to the torsional capacity, to be evaluated as indicated in the following. Torsional strength shall not be taken greater than $T_{\mathrm{Rd,max}}$. This last value denotes the ultimate strength of the concretet strut, to be evaluated according to the current building code.

(2) The existing steel contribution to the torsional capacity, $T_{\mathrm{Rd,s}}$, can be calculated as follows:

$$T_{\mathrm{Rd,s}} = \min\left\{ \frac{A_{\mathrm{sw}}}{p} \cdot 2 \cdot B_{\mathrm{e}} \cdot f_{\mathrm{ywd}} \cdot \cot\theta \,,\; \frac{A_{\mathrm{l}}}{u_{\mathrm{e}}} \cdot 2 \cdot B_{\mathrm{e}} \cdot f_{\mathrm{yd}} \cdot \tan\theta \right\} \tag{4.35}$$

where A_{sw} is a one-leg steel stirrup area, p is the stirrups spacing, B_{e} is the area bounded by the polygon whose edges are the centers of gravity of the longitudinal steel bars, f_{ywd} is the design yield strength of transverse steel reinforcement, A_{l} is the total area of existing steel longitudinal reinforcement, u_{e} is the perimeter of the above mentioned polygon, f_{yd} is the design yield strength of the existing steel longitudinal reinforcement, and θ is the angle of the compressed struts with respect to the member longitudinal axis (the value $\theta = 45°$ can be assumed if a more accurate determination is not available).

(3) $T_{\mathrm{Rd,max}}$ in Equation (4.34) shall be calculated as follows:

$$T_{\mathrm{Rd,max}} = 0.50 \cdot f_{\mathrm{cd}} \cdot B_{\mathrm{e}} \cdot h_{\mathrm{s}} \tag{4.36}$$

where f_{cd} is the design concrete compressive strength, and h_{s} is set equal to $1/6$ of the diameter of the maximum circle enclosed in the polygon whose edges are the centers of gravity of the existing steel longitudinal reinforcement.

(4) If Equation (4.35) shows that the minimum torsional capacity is due to the existing steel transverse reinforcement, the FRP contribution expressed in Equation (4.34) shall be calculated as follows:

$$T_{\mathrm{Rd,f}} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot 2 \cdot f_{\mathrm{fed}} \cdot t_{\mathrm{f}} \cdot b \cdot h \cdot \frac{w_{\mathrm{f}}}{p_{\mathrm{f}}} \cdot \cot\theta \qquad (4.37)$$

where the partial factor γ_{Rd} shall be set equal to 1.20 (Table 3-3, Section 3.4.2); f_{fed} is the FRP design effective strength to be evaluated as in Section 4.3.3.2; t_{f} is the thickness of the FRP strip or sheet; b and h are section width and depth, respectively; θ is the angle of the compressed struts with respect to the member longitudinal axis (the value $\theta = 45°$ can be assumed if a more accurate determination is not available); and w_{f} and p_{f} are width and center-to-center spacing of FRP strips measured orthogonally to the fiber direction, respectively. For FRP strips applied one next to each other the ratio $w_{\mathrm{f}} / p_{\mathrm{f}}$ shall be set equal to 1.0.

(5) If Equation (4.35) shows that the minimum torsional capacity is due to the existing steel longitudinal reinforcement, FRP strengthening shall not be performed.

(6) In case of combined torsion, T_{sd}, and shear, V_{sd}, the following limitation shall be met:

$$\frac{T_{\mathrm{Sd}}}{T_{\mathrm{Rd,max}}} + \frac{V_{\mathrm{Sd}}}{V_{\mathrm{Rd,max}}} \le 1 \qquad (4.38)$$

where $T_{\mathrm{Rd,max}}$ and $V_{\mathrm{Rd,max}}$ are calculated according to Equations (4.36) and the requirements of the current building code, respectively. Because strengthening for shear and torsion are calculated separately, the overall strengthening area is given by the sum of the area deemed necessary for shear and torsional FRP strengthening.

4.4.3.2 Limitations and construction details

(1) For U-wrapped and completely wrapped configurations, a minimum 20 mm radius shall be provided when FRP sheets are installed around outside corners.

(2) For external FRP reinforcement in the form of discrete strips, strips width, w_{f} (mm), and center-to-center spacing between strips, p_{f} (mm), shall not exceed the following limitations, respectively: $50\ \mathrm{mm} \le w_{\mathrm{f}} \le 250\ \mathrm{mm}$, and $w_{\mathrm{f}} \le p_{\mathrm{f}} \le \min\{0.5 \cdot d, 3 \cdot w_{\mathrm{f}}, w_{\mathrm{f}} + 200\ \mathrm{mm}\}$.

4.5 CONFINEMENT

4.5.1 Introduction

(1)P Appropriate confinement of reinforced concrete members may improve their structural performance. In particular, it allows the increase of the following:

- Ultimate capacity and strain for members under concentric or slightly eccentric axial loads.
- Ductility and capacity under combined bending and axial load, when FRP reinforcements are present with fibers lying along the longitudinal axis of the member (Section 4.2.2.4 and

Appendix C).

(2)P Confinement of RC members can be realized with FRP sheets disposed along the member perimeter as both continuous or discontinuous external wrapping.

(3)P The increase of axial capacity and ultimate strain of FRP-confined concrete depends on the applied confinement pressure. The latter is a function of the member cross section and FRP stiffness.

(4)P The redistribution of vertical loads cannot rely upon the ductility of members under concentric or slightly eccentric axial load.

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

(6)P A typical stress-strain (σ/ε) diagram for compression tests carried out on FRP-confined specimens is reported in Figure 4-10.

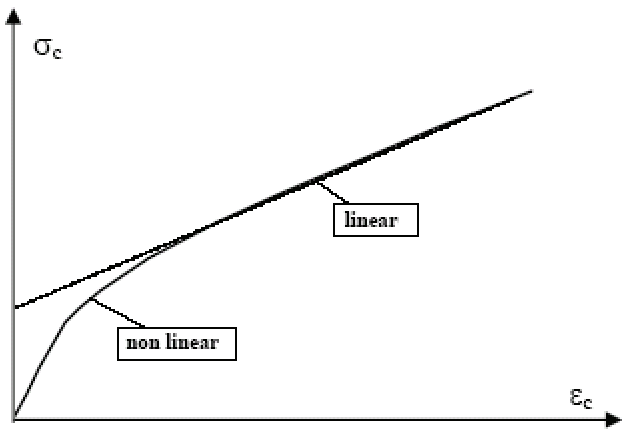


Figure 4-10 – Stress-strain relationship for FRP-confined concrete.

(7)P For axial strain values, ε_c, up to 0.2 %, the stress in the confined concrete is only slightly greater than that exhibited by unconfined concrete.

(8)P For axial strain values larger than 0.2 %, the stress-strain diagram is non linear and the slope of the corresponding σ/ε curve gradually lowers up to a nearly constant value. In the linear branch of the diagram, the confined concrete gradually loses its integrity due to widespread cracks.

(9)P Failure of RC confined member is attained by fiber rupture. However, beyond a critical value of the axial strain, the FRP-confined member may be linked to a recipient with very flexible walls filled with incoherent material. Beyond that threshold it loses its functionality since it can only carry small or negligible transverse forces. As a result, failure of the FRP-confined RC member is said to be reached when FRP strain equal to 0.4 % is attained.

4.5.2 Axial capacity of FRP-confined members under concentric or slightly eccentric force

(1)P Confinement of RC member with FRP is necessary for structural members subjected to concentric or slightly eccentric axial loads larger than the corresponding axial capacity.

(2)P Good confinement can only be achieved by installing FRP fibers orthogonally to the member axis.

(3)P When FRP reinforcement is spirally arranged around the member perimeter, the confinement effectiveness shall be properly evaluated.

(4)P If the adopted FRP system is not initially prestressed, it exerts a passive confinement on the compressed member. The confinement action becomes significant only after cracking of the concrete and yielding of the internal steel reinforcement due to the increased lateral expansion exhibited by the strengthened member. Prior to concrete cracking, FRP is practically unloaded.

(5)P Design at ULS of FRP confined members requires that both factored design axial load, N_{Sd}, and factored axial capacity, $N_{\mathrm{Rcc,d}}$, satisfy the following inequation:

$$N_{\mathrm{Sd}} \leq N_{\mathrm{Rcc,d}} \tag{4.39}$$

(6) For non-slender FRP confined members, the factored axial capacity can be calculated as follows:

$$N_{\mathrm{Rcc,d}} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot A_{\mathrm{c}} \cdot f_{\mathrm{ccd}} + A_{\mathrm{s}} \cdot f_{\mathrm{yd}} \tag{4.40}$$

where the partial factor γ_{Rd} shall be taken equal to 1.10 (Table 3-3, Section 3.4.2); A_{c} and f_{ccd} represent member cross-sectional area and design strength of confined concrete as indicated in item (7), respectively; and A_{s} and f_{yd} represent area and yield design strength of existing steel reinforcement, respectively.

(7) The design strength, f_{ccd}, of confined concrete shall be evaluated as follows:

$$\frac{f_{\mathrm{ccd}}}{f_{\mathrm{cd}}} = 1 + 2.6 \cdot \left(\frac{f_{\mathrm{l,eff}}}{f_{\mathrm{cd}}} \right)^{2/3} \tag{4.41}$$

where f_{cd} is the design strength of unconfined concrete as per the current building code, and $f_{1,\mathrm{eff}}$ is the effective confinement lateral pressure as defined in the following section. The same relationship shall be used also to attain the second objective mentioned in Section 4.5.1 (1)P.

(8) The confinement is effective if $f_{1,\mathrm{eff}} / f_{\mathrm{cd}} > 0.05$.

4.5.2.1 Confinement lateral pressure

(1)P The effectiveness of FRP-confined members only depends on a fraction of the confinement lateral pressure, f_1, exerted by the system, namely effective confinement lateral pressure $f_{1,\mathrm{eff}}$.

(2) The effective confinement lateral pressure, $f_{1,\mathrm{eff}}$, is a function of member cross section and FRP configuration as indicated in the following equation:

$$f_{\mathrm{l,eff}} = k_{\mathrm{eff}} \cdot f_{\mathrm{l}} \tag{4.42}$$

where k_{eff} is a coefficient of efficiency (≤1), defined as the ratio between the volume, $V_{\mathrm{c,eff}}$, of the effectively confined concrete and the volume, V_{c}, of the concrete member neglecting the area of existing internal steel reinforcement.

(3) The confinement lateral pressure shall be evaluated as follows:

$$f_{\mathrm{l}} = \frac{1}{2} \cdot \rho_{\mathrm{f}} \cdot E_{\mathrm{f}} \cdot \varepsilon_{\mathrm{fd,rid}} \tag{4.43}$$

where ρ_{f} is the geometric strengthening ratio as a function of section shape (circular or rectangular) and FRP configuration (continuous or discontinuous wrapping), E_{f} is Young modulus of elasticity of the FRP in the direction of fibers, and $\varepsilon_{\mathrm{fd,rid}}$ is a reduced FRP design strain, defined in the following item (9).

(4) The coefficient of efficiency, k_{eff}, shall be expressed as:

$$k_{\mathrm{eff}} = k_{\mathrm{H}} \cdot k_{\mathrm{V}} \cdot k_{\alpha} \tag{4.44}$$

(5) The coefficient of horizontal efficiency, k_{H}, depends on cross-section shape (see Sections 4.5.2.1.1 and 4.5.2.1.2).

(6) The coefficient of vertical efficiency, k_{V}, depends on FRP configurations. For RC confined members with continuous FRP wrapping, it is assumed $k_{\mathrm{V}} = 1$. For RC confined members with discontinuous FRP wrapping (Figure 4-11), such as FRP strips installed with a center-to-center spacing of p_{f} and clear spacing of p'_{f}, reduction in the confinement effectiveness due to the diffusion of stresses (approximately at 45°) between two subsequent wrappings shall be considered.

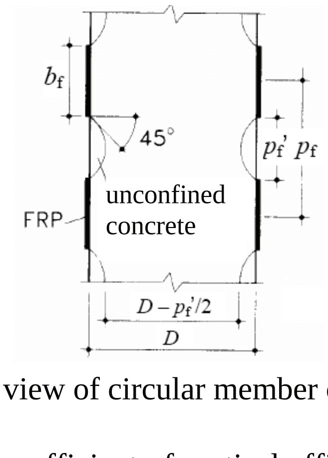


Figure 4-11 – Elevation view of circular member confined with FRP strips.

Regardless of the section shape, the coefficient of vertical efficiency, k_{V}, shall be assumed as follows:

$$k_{\mathrm{V}} = \left(1 - \frac{p'_{\mathrm{f}}}{2 \cdot d_{\mathrm{min}}}\right)^2 \tag{4.45}$$

where $d_{\min}$ is the minimum cross-section of the member.

(7) In case of discontinuous wrapping the net distance between strips shall satisfy the limitation $p_{\mathrm{f}}^{'} \leq d_{\min}/2$.

(8) Regardless of the section shape, the efficiency coefficient, k_{α}, to be used when fibers are spirally installed with an angle α_{f} with respect to the member cross-section, shall be expressed as follows:

$$k_{\alpha} = \frac{1}{1+(\tan\alpha_{\mathrm{f}})^2} \tag{4.46}$$

(9) The reduced FRP design strain, $\varepsilon_{\mathrm{fd,rid}}$, shall be computed as follows (see also Section 4.5.1, item (9)P):

$$\varepsilon_{\mathrm{fd,rid}} = \min\{\eta_{\mathrm{a}} \cdot \varepsilon_{\mathrm{fk}} / \gamma_{\mathrm{f}};\ 0.004\} \tag{4.47}$$

where η_{a} and γ_{f} represent environmental conversion factor and partial factor as suggested in Table 3-4 and Table 3-2, respectively.

4.5.2.1.1 Circular sections

(1)P FRP-confinement is particularly effective for a circular cross section subjected to both concentric or slightly eccentric axial loads.

(2)P Fibers installed transversely to the longitudinal axis of the strengthened member induce a uniform pressure that opposes the radial expansion of the loaded member.

(3) The geometric strengthening ratio, ρ_{f}, to be used for the evaluation of the effective confinement pressure shall be expressed as follows:

$$\rho_{\mathrm{f}} = \frac{4 \cdot t_{\mathrm{f}} \cdot b_{\mathrm{f}}}{D \cdot p_{\mathrm{f}}} \tag{4.48}$$

where (Figure 4-11) t_{f}, b_{f}, and p_{f} represent FRP thickness, width, and spacing, respectively, and D is the diameter of the circular cross section. In case of continuous wrapping, ρ_{f} becomes $4 \cdot t_{\mathrm{f}}/D$.

(4) For circular cross sections, the coefficient of horizontal efficiency, k_{H}, is equal to 1.0.

(5) For circular sections, the dimension $d_{\min}$ introduced in Equation (4.45) for the computation of the coefficient of vertical efficiency, is to be intended as the section diameter.

4.5.2.1.2 Square and rectangular sections

(1)P FRP-confinement of members with square or rectangular cross sections produce marginal increases of the member compressive strength. Therefore such applications shall be carefully validated and analyzed.

(2)P Prior to FRP application, the cross section edges shall be rounded to avoid stress concentrations that could lead to a premature failure of the system.

(3) The corner radius shall satisfy the following limitation:

$$r_c \geq 20 \text{ mm} \tag{4.49}$$

(4) The strengthening geometric ratio, ρ_f, to be used for the evaluation of the effective confinement pressure shall be expressed as follows:

$$\rho_f = \frac{2 \cdot t_f \cdot (b+d) \cdot b_f}{b \cdot d \cdot p_f} \tag{4.50}$$

where t_f, b_f, and p_f represent FRP thickness, width, and spacing, respectively, while b and d are cross section dimensions of the rectangular member. In case of continuous wrapping, ρ_f becomes $2 \cdot t_f \cdot (b+d)/(b-d)$.

(5)P For rectangular cross sections, the effectively confined concrete area may be considered to be only a fraction of the overall concrete cross section (Figure 4-12). The reason for such a behavior lies in the "arch effect" that forms within the concrete cross section. Such an effect depends on the values of the corner radius r_c.

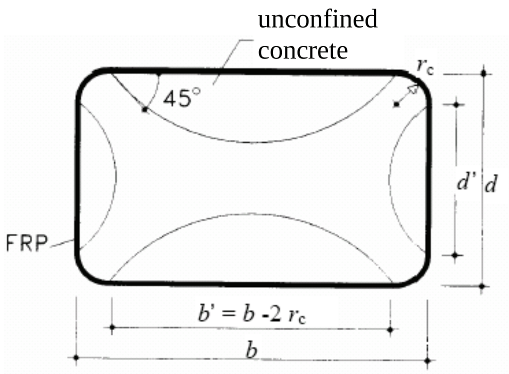


Figure 4-12 – Confinement of rectangular sections.

(6) For rectangular cross sections, the coefficient of horizontal efficiency, k_H, that takes into account the arch effect shall be expressed as follows:

$$k_H = 1 - \frac{b'^2 + d'^2}{3 \cdot A_g} \tag{4.51}$$

where b' and d' are the dimensions indicated in Figure 4-12, and A_g is the cross section area.

(7) The effect of FRP confinement shall not be considered for rectangular cross sections having $b/d > 2$, or $\max\{b, d\} > 900$ mm unless otherwise proven by suitable experimental tests.

4.5.3 Ductility of FRP-confined members under combined bending and axial load

(1)P FRP-confinement may also be realized on concrete members under combined bending and axial load; confinement will result in a ductility enhancement while the member axial capacity can only be slightly increased.

(2) Unless a more detailed analysis is performed, the evaluation of the ultimate curvature of a FRP confined concrete member under combined bending and axial load may be accomplished by assuming a parabolic-rectangular approach for the concrete stress-strain relationship, characterized by a maximum strength equal to f_{cd} and ultimate strain, ε_{ccu}, computed as follows:

$$\varepsilon_{ccu} = 0.0035 + 0.015 \cdot \sqrt{\frac{f_{l,eff}}{f_{cd}}} \tag{4.52}$$

where $f_{1,eff}$ is the effective confinement pressure, and f_{cd} is the design strength of unconfined concrete.

(3) In Equation (4.52), the effective pressure is computed assuming a reduced FRP design strain as follows:

$$\varepsilon_{fd,rid} = \eta_a \cdot \frac{\varepsilon_{fk}}{\gamma_f} \le 0.6 \cdot \varepsilon_{fk} \tag{4.53}$$

(4) More accurate evaluation of ultimate curvature and flexural capacity of FRP strengthened members may be obtained with suitable concrete-confined models (Appendix D) capable of capturing the behavior described in 4.5.1 and Figure 4-10.

4.6 FLEXURAL STRENGTHENING OF PRESTRESSED CONCRETE MEMBERS

4.6.1 Use of FRP for prestressed concrete members

(1) P The methods and criteria hereafter apply to non-prestressed FRP systems used for strengthening prestressed concrete (PC) structures.

4.6.1.1 Design at ultimate limit state

(1)P The evaluation of the flexural capacity of PC members subjected to a bending moment shall be carried out in the following procedures similar to those described in 4.2.2 for RC members, with the only changes specified in the following.

- The strain of prestressed reinforcement is equal to the algebraic sum of the strain of the concrete surrounding the tendon and the strain at the decompression limit, $\bar{\varepsilon}_p$. The latter represents the strain exhibited by existing tendons for an appropriate combination of internal forces such to produce zero stress in the concrete surrounding the tendons (Figure 4-13).
- The ultimate strain of prestressing tendons is equal to $0.01 + \bar{\varepsilon}_p$.
- If the concrete age is such to considered to have exhausted all long-term phenomena, the initial concrete strain, ε_0, is equal to the strain at the time of FRP installation.
- If long-term phenomena in the concrete can not be considered exhausted, the value of ε_0 is the algebraic sum of the previously computed value and the long-term strain developed in the concrete substrate after FRP strengthening takes place. In the evaluation of such a long-

term strain, as well as for any loss of prestress, the presence of the strengthening system can be neglected.

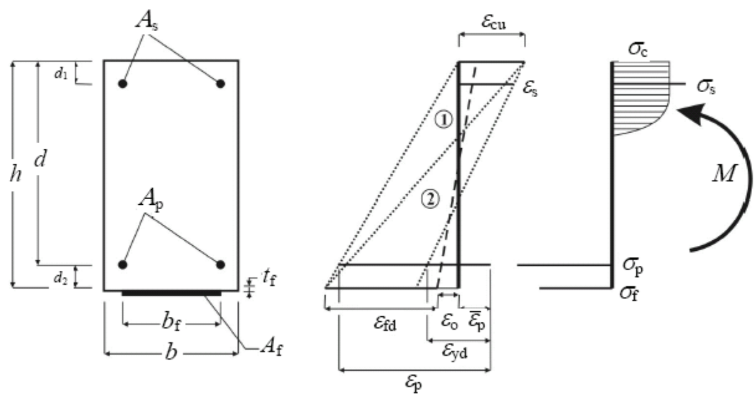


Figure 4-13 – Failure mode of PC member strengthened with FRP.

(2)P The attainment of the ultimate limit state shall be preceded by yielding of existing steel prestressing tendons.

(3) For debonding-failure mechanisms, refer to Sections 4.1 and 4.2.

4.6.1.2 Design at serviceability limit state

(1)P Service stress limitations for concrete and steel shall satisfy the requirements of the current building code. Service stress limitations for FRP material shall comply with 4.2.3.2.

(2)P FRP strengthening shall be neglected if temporarily compressed (*e.g.*, due to creep of concrete).

4.7 DESIGN FOR SEISMIC APPLICATIONS

4.7.1 Introduction

4.7.1.1 Design objectives

(1)P It shall be permitted to strengthen RC and PC members with FRP composites when the structure does not meet the seismic requirements specified in the current building code.

(2)P This part of the document recognizes the provisions of the current building code as well as the indications provided in the most updated literature related to seismic constructions; particular importance is given to the following:

- Evaluation of seismic safety.
- Safety requirements (verification of limit states).
- Levels of seismic protection (magnitude of the associated seismic action).
- Methods of analysis.
- Verification criteria (distinction between ductile and brittle members).
- Materials characteristics to be used for design.

4.7.1.2 Selection criteria for FRP strengthening

(1) Type and size of selected FRP systems as well as the urgency of FRP installation shall take into account the following:

- Common errors shall be eliminated.
- Major building irregularities can not be eliminated using FRP as a strengthening technique.
- A better resistance regularity can be obtained by strengthening a limited number of members.
- Enhancement of local ductility.
- Localized strengthening shall not reduce the overall ductility of the structure.

(2)P FRP strengthening may be classified as follows:

- Strengthening or total or partial reconstruction of members.

(3)P Design of FRP strengthening shall include the following:

- Motivated choice of the intervention type.
- Selection of appropriate technique and/or materials.
- Preliminary design of the selected FRP strengthening system.
- Structural analysis taking into account the characteristics of the structure after FRP strengthening.

(4)P FRP strengthening of RC members is mainly aimed at the following:

- Increase the member flexural capacity by using FRP material with fibers running in the axis direction of the strengthened member.
- Increase the member shear capacity by using FRP material with fibers running orthogonal to the direction of the axis of the strengthened member.
- Increase the ductility of end sections of beams and/or columns by using FRP material wrapped to the member cross section.
- Improve the efficiency of lap splices by using FRP material wrapped to the member cross section.
- Prevent buckling of steel longitudinal bars by using FRP material wrapped to the member cross section.
- Increase tensile capacity of beam-column joints by using FRP material installed with fibers located along the principal tensile stresses.

(5)P Design of FRP-strengthened members shall be in compliance with the following sections.

4.7.2 Strategies in FRP strengthening

(1)P When using FRP material for strengthening RC members, the following principles shall be taken into account:

- Removal of all brittle collapse mechanisms (Section 4.7.2.1).
- Removal of all storey collapse mechanisms (“soft storey”) (Section 4.7.2.2).
- Enhancement of the overall deformation capacity of the structure through one of the following mechanisms (Section 4.7.2.3):
  - Increasing the rotational capacity of the potential plastic hinges without changing their position (Section 4.7.2.3.1).
  - Relocating the potential plastic hinges following the capacity design criterion (hierarchy of resistance) (Section 4.7.2.3.2).

4.7.2.1 Removal of all brittle collapse mechanisms

(1)P Brittle collapse mechanisms to be removed as well as FRP strengthening methodologies are as follows:

- Shear failures shall be avoided; all members that present shear deficiency shall be strengthened.
- Failure due to loss of bond in steel overlapping areas; the areas where overlapping length of the longitudinal bars is not enough shall be confined with FRP wrapping.
- Failure due to buckling of steel longitudinal bars: areas where the formation of plastic hinges is expected shall be confined with FRP wrapping when existing steel transverse reinforcement can not prevent the post-elastic buckling of compressed longitudinal bars.
- Failure due to tensile stresses on beam-column joint; FRP strengthening shall be applied.

4.7.2.2 Removal of all storey collapse mechanisms

(1)P Storey collapse mechanisms usually start following the formation of plastic hinges at column top and bottom locations of structures with no vertical walls. In such a case, FRP strengthening can be performed to enhance the column flexural capacity with the intent of precluding the formation of plastic hinges. In no case is the removal of the storey collapse mechanisms allowed with the only intent of increasing storey displacements.

4.7.2.3 Enhancement of the overall deformation capacity of a structure

(1)P The ultimate deformation capacity of a structure is a measure of its capability to sustain seismic forces.

(2) The ultimate deformation capacity of a structure can be computed with non linear static analysis methods (push-over analysis).

(3)P The ultimate deformation capacity of a structure depends on the plastic deformation capacity of each single resisting member (beams, columns, and walls).

4.7.2.3.1 Increasing of the local rotational capacity of RC members

(1) The deformation capacity of beams and columns may be measured through the rotation θ of the end section with respect to the line joining the latter with the section of zero moment (chord rotation) at a distance equal to the shear span: $L_V = M/V$. Such a rotation is also equal to the ratio of the relative displacement between the two above mentioned sections to the shear span.

(2)P The deformation capacity of RC members in the plastic range is limited by the failure of compressed concrete. FRP confinement increases the ultimate deformation of compressed concrete and enhances the ductility of the strengthened member.

4.7.2.3.2 Capacity design criterion

(1)P The application of the capacity design criterion (hierarchy of resistance) implies the adoption of mechanisms such to prevent the formation of all potential plastic hinges in the columns. In the “weak column-strong beam” situations, typical for structures designed for vertical loads only, columns are under-designed due to lack of longitudinal reinforcement. In such a case, it is deemed necessary to increase the column capacity under combined bending and axial load toward a “strong column-weak beam” situation.

(2)P When FRP reinforcement is used to increase the flexural capacity of a member, it is important to verify that the member will be capable of resisting the shear forces associated with the increased flexural strength. If deemed necessary, shear strengthening shall be taken into account to avoid premature brittle collapse mechanisms.

4.7.3 Safety requirements

4.7.3.1 Ductile members and mechanisms

4.7.3.1.1 Combined bending and axial load

(1) Flexural capacity of ductile members may be increased with FRP material.

(2) FRP strengthening design of members subjected to bending or combined bending and axial load shall be performed according to Sections 4.2 and 4.5.

(3) When the member flexural capacity is increased, particular care shall be taken to properly anchor the adopted FRP reinforcement.

(4) Longitudinal fibers used for strengthening RC members subjected to combined bending and axial load shall be properly confined to avoid fiber delamination and concrete spalling under cyclic loads.

4.7.3.1.2 Chord rotation

(1) The chord rotation of mono-dimensional members (mainly beams and columns) may be increased with FRP-confinement.

(2) For the evaluation of the ultimate chord rotation, θ_u, of members strengthened with FRP confinement, the following relationship shall be used:

$$\theta_u = \frac{1}{\gamma_{el}} \cdot \left[\theta_y + (\phi_u - \phi_y) \cdot L_{pl} \cdot \left(1 - 0.5 \cdot \frac{L_{pl}}{L_v} \right) \right] \tag{4.54}$$

where γ_{el} is typically assumed equal to 1.5 or 1.0. Such values are only valid for "secondary members" (*e.g.*, members whose stiffness and resistance may be neglected in the response analysis, even though they should be able to carry the deformations of the structure under the design seismic loads preserving their supporting capacity against vertical loads). The remaining terms of Equation (4.54) have the following meaning:

- θ_y represents the chord rotation exhibited by the end section when the steel is yielded:

$$\theta_y = \phi_y \cdot \frac{L_V}{3} + 0.0013 \cdot \left(1 + 1.5 \cdot \frac{h}{L_V} \right) + 0.13 \cdot \phi_y \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \tag{4.55}$$

where h is the section depth, d_b is the (average) diameter of the longitudinal bars, and f_c and f_y are concrete compressive strength and steel yield longitudinal strength (in MPa), respectively, obtained from in-situ tests of the existing materials and divided by a confidence factor to be set larger than 1 when there is no adequate level of knowledge of the existing structure.

- ϕ_u represents the ultimate curvature of the end section, evaluated by assigning at the concrete ul-

timate strain, ε_{ccu}, the value defined in Section 4.5.3.
- ϕ_y represents the curvature exhibited by the end section when the steel is yielded.
- L_{p1} represents the magnitude of the plastic hinge evaluated as follows:

$$L_{pl} = 0.1 \cdot L_V + 0.17 \cdot h + 0.24 \cdot \frac{d_b \cdot f_y}{\sqrt{f_c}} \tag{4.56}$$

- L_V represents the shear span of the member (distance between the maximum moment point and the point of zero moment).

4.7.3.2 Brittle members and mechanisms

4.7.3.2.1 Shear

(1) For shear design of RC members strengthened with FRP the criteria of Section 4.3 shall apply; further provisions as follows shall be taken into account:

- Only U-wrapped or completely wrapped configurations are allowed.
- FRP strengthening can only be installed with fibers orthogonal to the member longitudinal axis ($\beta = 90°$).

4.7.3.2.2 Lap splices

(1) Slip of existing steel reinforcement in RC columns at the locations of lap splice may be avoided by confining the member cross section with FRP.

(2) For circular cross sections of diameter *D*, the thickness of the FRP confining the member cross section shall be evaluated as follows:

$$t_f = \frac{D \cdot (f_l - \sigma_{sw})}{2 \cdot 0.001 \cdot E_f} \tag{4.57}$$

where:
- σ_{sw} represents the stirrup's tensile stress corresponding to a 1 % strain or the mortar injection pressure between the FRP reinforcement and the RC column, if present.
- f_1 represents the confinement pressure at the lap splice location extending for a length, L_s, equal to:

$$f_l = \frac{A_s \cdot f_y}{\left[\frac{u_e}{2 \cdot n} + 2 \cdot (d_b + c)\right] \cdot L_s} \tag{4.58}$$

where u_e is the perimeter of the cross section within the polygon circumscribing the longitudinal bars having average diameter d_b, *n* is the number of bars spliced along u_e, and *c* is the concrete cover.

(3) For rectangular sections *b*x*d*, Equation (4.57) shall be used by replacing *D* with $\max\{b, d\}$; the effectiveness of the FRP confinement shall be reduced by the factor k_H defined in Section 4.5.2.1.2.

4.7.3.2.3 Buckling of longitudinal bars

(1) Buckling of existing steel vertical reinforcement of RC columns may be avoided by confining the member cross section with FRP.

(2) The thickness of such FRP confinement shall be evaluated as follows:

$$t_f = \frac{0.45 \cdot n \cdot f_y^2 \cdot d}{4 \cdot E_{ds} \cdot E_f} \approx \frac{10 \cdot n \cdot d}{E_f} \tag{4.59}$$

where:
- n represents the total number of existing steel longitudinal bars subjected to buckling.
- f_y has been introduced in Section 4.7.3.1.2.
- d represents the size of the cross section parallel to the bending plane.
- E_f represents the Young modulus of elasticity of FRP reinforcement in the direction of existing steel vertical bars.
- E_{ds} represents a suitable "reduced modulus" defined as follows:

$$E_{ds} = \frac{4 \cdot E_s \cdot E_i}{\left(\sqrt{E_s} + \sqrt{E_i}\right)^2} \tag{4.60}$$

where E_s and E_i are the initial Young modulus and the tangent modulus of elasticity of existing steel vertical bars after yielding, respectively.

4.7.3.2.4 Joints

(1) Beam-column joints of RC members can be effectively strengthened with FRP only when FRP reinforcement is applied with the fibers running in the direction of principal tensile stresses and provided that FRP reinforcement is properly anchored. In any case, the maximum tensile strain for FRP reinforcement shall not be larger than 4 %. When FRP reinforcement is not properly anchored, FRP strengthening shall not be considered effective.

4.8 INSTALLATION, MONITORING, AND QUALITY CONTROL

(1)P Several aspects influence the effectiveness of FRP material used as externally bonded systems for strengthening RC members. In addition to those discussed in previous chapters, surface preparation and FRP installation will be dealt with in this section. The relative importance of each of these aspects depends on whether reference is made to applications defined as "bond-critical" (flexure or shear) or "contact-critical" (confinement). For example, some tests on the quality of the substrate can be omitted for contact-critical applications (*e.g.*, confinement by wrapping) or when the FRP system is properly anchored provided that its effectiveness is tested by an independent laboratory.

(2)P Once FRP strengthening has been carried out, over time monitoring by non destructive or semi-destructive tests as listed in the following sections shall be performed to ensure the effectiveness of the proposed strengthening solution.

(3)P This document describes the tests that can be carried out for quality control as well as over time monitoring of the FRP system. The type and number of tests to be performed shall be linked to the importance of the application by taking into account the following:

- For strategic buildings or infrastructures, when their functionality during seismic events is of fundamental importance for civil protection or when their roles becomes relevant due to the consequences of a possible collapse.
- When FRP application concerns primary structural members (*e.g.,* beams and columns) or secondary elements (*e.g.,* floors).
- The extent of FRP application compared to the size of the structure.

(4)P The numerical values indicated in the following are to be understood as suggested values.

4.8.1 Quality control and substrate preparation

(1)P Quality control of the support implies determination of concrete conditions, removal of any deteriorated or loose concrete, cleaning as well as protection from corrosion of existing steel reinforcement, and finally substrate preparation for receiving the selected FRP reinforcement.

(2)P When special devices are used to properly anchor the selected FRP system, testing of such devices shall be conducted in compliance with available standardization documents. Anchoring devices shall be installed according to the manufacturer/supplier specifications regarding both material used as well as surface preparation, environmental conditions, and sequence of each phase. The investigation shall also evaluate the effect of such parameters on the final result.

4.8.1.1 Evaluation of substrate deterioration

(1) Prior to FRP application, soundness of the concrete substrate shall be checked. In any case, concrete compressive strength shall not be less than 15 $\mathrm{N/mm^2}$. FRP strengthening shall not be considered effective for concrete compressive strength less than 15 $\mathrm{N/mm^2}$.

(2) It is suggested to carry quality control tests on the entire area to be strengthened.

4.8.1.2 Removal of defective concrete, restoring of concrete substrate and protection of existing steel reinforcement

(1) Concrete substrate may have undergone physical-chemical, physical-mechanical, or impact-caused deterioration. Deteriorated concrete shall be removed from all damaged areas.

(2) Removal of unsound concrete allows for assessment of existing steel reinforcing bars. Corroded steel bars shall be protected against further corrosion so as to eliminate a possible source of deterioration of the restored concrete.

(3) Once all deteriorated concrete has been removed, and suitable measures have been taken to prevent further corrosion of existing steel reinforcement as well as all other phenomena causing concrete degradation (*e.g.,* water leakage), concrete restoration using shrinkage-free cement grouts shall be performed. Roughness of the concrete surface larger than 10 mm shall be leveled with compatible epoxy paste; specific filling material shall be used for unevenness larger than 20 mm. Also, cracks within solid concrete in the substrate wider than 0.5 mm shall be stabilized using epoxy injection methods before FRP strengthening can take place.

4.8.1.3 Substrate preparation

(1) Once the quality control of the substrate has been performed, the deteriorated concrete has been removed, the concrete cross section restored, and the existing steel reinforcement has been properly treated, sandblasting of the concrete surface to be strengthened shall be performed. Sandblasting shall provide a roughness degree of at least 0.3 mm; such level of roughness can be meas-

ured by suitable instruments (*e.g.,* a laser profilometer or an optical profile-measuring device).

(2) Poor concrete surfaces that do not need remedial work before FRP application, should be treated with a consolidating agent before *primer* application takes place.

(3) Cleaning of the concrete surface shall remove any dust, laitance, oil, surface lubricants, foreign particles, or any other bond-inhibiting material.

(4) All inside and outside corners and sharp edges shall be rounded or chamfered to a minimum radius of 20 mm.

4.8.2 Recommendations for the installation

(1) FRP strengthening of RC members is highly dependant upon environmental temperature and humidity as well as the characteristics of the concrete substrate. In addition to the above mentioned measure, and regardless of the strengthening type, further specific precautions to ensure quality of FRP installation are highlighted in the next sections.

4.8.2.1 Humidity and temperature conditions in the environment and substrate

(1) It is suggested not to install FRP material when the environment is very moist; a high degree of humidity may delay the curing of resin and affect the overall performance of the strengthening system especially for wet lay-up applications.

(2) FRP strengthening systems shall not be applied to substrates having a surface humidity level greater than 10 %; such conditions could delay the penetration of the *primer* in the concrete pores and generate air bubbles that could compromise bond between concrete and FRP system. Substrate humidity may be evaluated with a hygrometer for mortar or simply by employing absorbent paper.

(3) FRP material shall not be applied if both environment and surface temperature is too low, because curing of the resins as well as fiber's impregnation could be compromised. It is also recommended not to install FRP material when concrete surface is heavily exposed to sunlight. It is suggested not to apply any FRP material if temperatures fall outside the range of 10° to 35°C. In low temperature environments where the construction site schedule does not allow FRP installation to be delayed, it is suggested to artificially heat the locations where FRP reinforcement is to be applied.

(4) If curing of FRP reinforcement takes place under rainy conditions, heavy insulation, large thermal gradients, or in the presence of dust, protective measures can be employed to ensure proper curing.

4.8.2.2 Construction details

(1) Anchorage length of at least 200 mm shall be provided for the end portion of FRP systems used for strengthening RC members. Alternatively, mechanical connectors may be used.

(2) Proper fibers alignment shall be provided for in-situ wet lay-up application; waving of FRP reinforcement shall also be avoided during installation.

(3) When carbon fibers reinforcement is to be used for strengthening RC members where there is potential for direct contact between carbon and existing steel reinforcement, layers of insulating material shall be installed to prevent the occurrence of galvanic corrosion.

(4) When semi-destructive tests are planned, it is suggested to provide additional strengthening areas ("witness areas") in properly selected parts of the structure having dimensions of at least 500x200 mm^2, with a minimum extension of 0.1 m^2 nor less than 0.5 % of the overall area to be strengthened. Witness areas shall be realized at the same time of the main FRP installation, using the same materials and procedures in areas where removal of FRP strengthening system does not imply alteration of the failure mechanisms. In addition, witness areas shall be exposed to the same environmental conditions of the main FRP system and shall be uniformly distributed on the strengthened structure.

4.8.2.3 Protection of the FRP system

(1) For outdoor FRP applications it is recommended to protect the FRP system from direct sunlight, which may produce chemical-physical alterations in the epoxy matrix. This can be achieved by using protective acrylic paint provided that cleaning of the composite surface with a sponge soaked in soap is performed.

(2) Alternatively, a better protection can be achieved by applying plastering or mortar layer (preferably concrete-based) to the installed strengthening system. The plaster, whose thickness is recommended by the FRP manufacturers/suppliers, is to be laid on the strengthening system after treating the surface by means of epoxy resin applications with subsequent quartz dusting green-on-green. The final layer is particularly suitable to receive any kind of plastering.

(3) For fire protection, two different solutions may be adopted: the use of intumescent panels or the application of protective plasters. In both cases, manufacturers/suppliers shall indicate the degree of fire protection as a function of the panel/plaster thickness. The panels -generally based on calcium silicates -are applied directly on the FRP system provided that fibers will not be cut during their installation. Protective plasters represent the most widely adopted solution for fire protection; they shall be applied to the FRP system as indicated before. Protective coatings of adequate thickness and consistency capable of keeping the composite temperature below 80°C for 90 minutes are available.

4.8.3 Quality control during installation

(1) Quality control during FRP installation should include at least one cycle of semi-destructive tests for the mechanical characterization of the installation itself, and at least one non destructive mapping to ensure its uniformity.

4.8.3.1 Semi-destructive tests

(1) Both pull-off tests and shear tearing tests may be carried out. Semi-destructive tests shall be carried out on witnesses and, where possible, in non-critical strengthened areas at the rate of one test for every 5 m^2 of application, and in any case, not less than 2 per each type of test.

(2) <u>Pull-off test</u>. The test is used for assessment of the properties of restored concrete substrate; it is carried out by using a 20 mm thick circular steel plates with a diameter of at least 3 times the characteristic size of the concrete aggregate nor less than 40 mm, adhered to the surface of the FRP with an epoxy adhesive. After the steel plate is firmly attached to the FRP, it is isolated from the surrounding FRP with a core drill rotating at a speed of at least 2500 rpm; particular care shall be taken to avoid heating of the FRP system while a 1-2 mm incision of the concrete substrate is achieved. FRP application may be considered acceptable if at least 80 % of the tests (both tests in case of only two tests) return a pull-off stress not less than 0.9-1.2 MPa provided that failure occurs in the concrete substrate.

(3) Shear tearing test. The test is particularly significant to assess the quality of bond between FRP and concrete substrate. It may be carried out only when it is possible to pull a portion of the FRP system in its plane located close to an edge detached from the concrete substrate. FRP application may be considered acceptable if at least 80 % of the tests (both in the case of two tests) return a peak tearing force not less than 24 kN.

4.8.3.2 Non destructive tests

(1) Non destructive tests may be used to characterize the uniformity of FRP application starting from adequate two-dimensional survey of the strengthened surface with a different spatial resolution as a function of the strengthening area (see Table 4-1).

(2) Stimulated Acoustic testing. Similar to impact-echo testing, such tests rely on the different oscillatory behavior of the composite layer depending on the bond between FRP layers and concrete substrate. In its most basic version, such a test may be carried out by a technician hammering the composite surface and listening to the sound from the impact. More objective results may be obtained with automated systems.

(3) High-frequency ultrasonic testing. They should be carried out using reflection methods with frequencies no less than 1.5 MHz and probes with a diameter no greater than 25 mm, adopting the technique based on the first peak amplitude variation to localize defects.

Table 4-1 – Minimum resolution for defects thickness to be identified with non destructive tests.

Shear stress transfer at interface	Example	Non destructive test	Surface mapping grid	Minimum resolution for defects thickness
absent	wrappings, with the exception of the overlapping area in single-layer application	Optional	250 mm	3.0 mm
weak	central area of very extensive plane reinforcement	Optional	250 mm	3.0 mm
moderate	central area of longitudinal flexural strengthening	Suggested	100 mm	0.5 mm
critical	anchorage areas, overlapping areas between layers, stirrups for shear strengthening, interface areas with connectors, areas with large roughness or cracks in the substrate	Required	50 mm	0.1 mm

(4) Thermographic tests. They are effective only for FRP systems with low thermal conductivity and can not be applied to carbon or metallic FRP strengthening systems unless special precautions are taken. The heat developed during the test shall be smaller than the glass transition temperature of the FRP system.

(5) Acoustic emission tests. The technique is based on the acoustic emission (AE) method and allows the assessment of a damage inside a structural member subjected to loading by listening to and recording the sound generated by either formation of cracks or delamination phenomena that propagates as elastic waves. Such a test is particularly suitable for detecting defects in the application of FRP composites on RC structures as well as the occurrence of delamination from the concrete substrate.

4.8.4 Personnel qualification

(1) Personnel in charge of the tests shall have one of the three qualification levels specified in Table 4-2, according to UNI EN 473 and UNI EN 45013.

Table 4-2 – Qualification levels to perform semi and non-destructive tests.

Level 1	Proper knowledge of tests equipment; performing tests; recording and classifying test results according to written criteria; writing a report on test results.
Level 2	Choosing the way of performing the test; defining the application limits of the test for which the level 2 technician is certified; understanding test specifications and translating them into practical test instructions suitable to the in-situ working conditions; adjusting and calibrating test equipments; performing and controlling the test; interpreting and evaluating test results according to the specifications to comply with; preparing written test instructions for level 1 personnel; performing and supervising all level 1 functions; training personnel of level 1; organizing test results and writing the final report.
Level 3	Be in charge of a laboratory facility; establishing and validating test techniques and procedures; interpreting specifications and procedures; having the skill to evaluate and understand test results according to existing specifications; having a sufficient practical knowledge of materials, production methods and installation technology of the system to be tested to be able to choose appropriate methods, establish techniques and collaborate in the definition of acceptance criteria when they are not pre-established; be knowledgeable in different application fields; being able to lead personnel of level 1 and 2.

4.8.5 Monitoring of the strengthening system

(1) Due to the poor availability of data regarding long term behavior of FRP systems used for strengthening RC structures, it is recommended to perform appropriate monitoring of the installed FRP system by means of semi and non-destructive tests periodically conducted on the strengthened structure. The aim of such a monitoring process is to keep the following parameters under control:

- Temperature of the installed FRP system.
- Environmental humidity.
- Measure of displacements and deformations of the strengthened structure.
- Potential damage of fibers.
- Extensions of defects and delaminations in the installed FRP system.

4.9 NUMERICAL EXAMPLES

Some numerical examples concerning FRP strengthening of RC structures are reported in Appendix E.

5 STRENGTHENING OF MASONRY STRUCTURES

5.1 INTRODUCTION

5.1.1 Scope

(1)P This chapter specifies design recommendations for masonry structural members strengthened with FRP.

(2)P The primary objective of FRP strengthening is to increase the capacity of each member as well as the overall capacity of the masonry structure; whenever possible, enhancement of structural displacement at failure is also recommended.

5.1.2 Strengthening of historical and monumental buildings

(1)P Strengthening of historical and monumental buildings shall be justified only when inevitable; the adopted strengthening technique shall be in compliance with the theory of restoration (see Section 3.1 item (3)).

5.1.3 FRP strengthening design criteria

(1) Strengthening methodologies addressed in this document consist of the application of FRP materials in the form of laminates, sheets, grids, and bars, installed on the members by adhesion or by means of mechanical anchorage devices. FRP reinforcement may be applied to the external surfaces of the masonry structure as well as in slots or grooves cut in the masonry itself.

(2) FRP strengthening can be employed for the following reasons:

- Flexural and shear strengthening to carry tensile stresses within the structural member or between adjoining members.
- Connection between members (vault and wall ties, connections between orthogonal walls, etc.).
- Floor stiffening to act as a stiff diaphragm.
- Crack width limitation.
- Columns confinement to increase the material strength.

(3)P Design of FRP reinforcement shall ensure that the selected FRP system is always in tension. In fact, compression FRP is unable to increase the performance of the strengthened masonry member due to its small area compared to that of compressed masonry. Moreover, FRP in compression may be subjected to debonding due to local instability.

(4) For masonry structures strengthened with FRP and subjected to cyclic loads (*e.g.*,seismic, thermal variations), bond between masonry and FRP may degrade remarkably during the structure’s lifetime. In such a case, it could be necessary to properly anchor the FRP system to the masonry by either inserting FRP reinforcement in suitable grooves to prevent local instability or applying mechanical anchoring devices.

(5) For proper anchoring, FRP shall be extended up to the compressed masonry areas.

(6)P FRP strengthening shall be applied to structural members having suitable mechanical properties. If the masonry is damaged, not uniform, or cracked, it shall first be repaired with appropriate techniques to ensure a proper sharing of loads between support and FRP. The selection of the appropriate strengthening material (carbon, glass, or aramid FRP) shall also take into account physical and chemical properties of the masonry.

(7)P FRP reinforcement that completely encases the strengthened member may prevent migration of moisture. Such FRP systems shall not be applied continuously on extended areas of the wall surface to ensure migration of moisture.

5.1.4 Strengthening Rationale

(1) For the appropriateness of a FRP system for a particular application, the engineer should evaluate the existing structure to establish its existing load-carrying capacity, identify deficiencies and their causes, and determine the condition of the masonry substrate. The overall evaluation should include a thorough field inspection, review of existing design or as-built documents, as well as a structural analysis. The load-carrying capacity of the existing structure should be based on the information collected in the field investigation as well as the review of the existing documents and determined by analytical or any other suitable methods. FRP strengthening may be aimed at the following:

- Increasing the capacity of panels, arches, or vault.
- Wrapping of columns to enhance their compressive strength and ductility.
- Reducing thrust forces in thrusting structures.
- Transforming non structural members into structural members by increasing their stiffness and strength.
- Strengthening and stiffening horizontal non-thrusting structures.
- Wrapping buildings at floors and roof locations.

5.2 SAFETY EVALUATION

5.2.1 Structural modelling

(1)P Design of FRP reinforcement shall be based on a structural scheme representing the behavior of the building for the expected future use.

(2)P Internal forces acting on the masonry shall be determined using the methods of structural analysis. In particular, the structure can be modeled as either linear elastic or through proven non linear models capable of simulating the inelastic behavior and the negligible tensile strength of the masonry.

(3) Simplified schemes can also be used to describe the behavior of the structure. For example, provided that tensile stresses are directly taken by the FRP system, the stress level may be determined by adopting a simplified distribution of stresses that satisfies the equilibrium conditions but not necessarily the compatibility of strain. The use of simplified stress distributions should be very carefully chosen because a statically satisfactory stress level could have already caused the structure to collapse due to the brittle nature of the FRP-masonry system. In case of structures with regular or repetitive parts, partial structural schemes may be identified that allow for a rapid evaluation of the overall behavior of the strengthened structure. Likewise, simplified models may be adopted for verifications of local failure mechanisms, provided that their use is correctly motivated.

5.2.2 Verification criteria

(1)P Possible failure modes of masonry walls strengthened with FRP systems can be summarized as follows:

- Excessive cracking due to tensile stresses in the wall.

- Crushing of masonry.
- Shear-slip of masonry.
- FRP rupture.
- FRP debonding.

Failure mode of FRP strengthened masonry structures usually involves a combination of the above mentioned mechanisms.

5.2.3 Safety verifications

(1) Masonry can be considered an anisotropic material exhibiting a non-linear behavior. The stress-strain relationship may vary quite significantly depending whether the structure is built using artificial or natural blocks as well as the type of mortar employed.

(2) Masonry exhibits a brittle behavior when subjected to tensile loading; the corresponding tensile strength is negligible compared to its compressive strength. For design purposes, it is accepted to neglect tensile strength of masonry.

(3)P Laboratory tests show that the stress-strain diagram of masonry blocks subjected to compressive loads can be described as follows:

- Basically linear for low strain values.
- Non-linear as the load increases up to the ultimate value.
- Non-linear softening after the load at ultimate has been reached.

(4) The masonry behavior for compressive load also depends on the availability of transverse confinement. By increasing the transverse confinement, strength and ductility of the material is improved.

(5)P Masonry shear strength depends on the applied axial load because it usually relies upon cohesion and friction of the material.

(6)P The characteristic values for strength are as follows:

- f_{mk} for vertical compression.
- $f_{\mathrm{mk}}^{\mathrm{h}}$ for horizontal compression.
- f_{vk} for shear.

Such values shall be determined in compliance with the current building code. A reference value for $f_{\mathrm{mk}}^{\mathrm{h}}$ is 50 % of f_{mk}.

(7) Design values for mechanical properties of masonry are computed by dividing the characteristics' values by the partial factor of the material $\gamma_{\mathrm{m}} = \gamma_{\mathrm{M}}$ as well as by the partial factor for resistance model, γ_{Rd}, according to the current building code and the present document.

(8)P For most engineering applications, the behavior of masonry under uniaxial loads may be simplified as follows:

- tensile stress: neglected.

- compression: linear behavior with slope equal to the secant modulus of elasticity up to both design strength of f_{md} and design strain of $\bar{\varepsilon}_{\mathrm{m}}$; design strength equal to f_{mk} for strain between $\bar{\varepsilon}_{\mathrm{m}} \le \varepsilon \le \varepsilon_{\mathrm{mu}}$; and zero strength for strain larger than $\varepsilon_{\mathrm{mu}}$.

(9)P Unless experimental data is available, the masonry ultimate design strain, $\varepsilon_{\mathrm{mu}}$, may be assumed equal to 0.35 %.

(10) Alternatively, appropriate stress-strain diagrams embracing the behavior described in (3)P may be used, provided that their performance is validated on the basis of experimental investigations.

(11)P FRP materials are characterized by an anisotropic behavior, as described in detail in Section 6.2. When stressed in the fiber direction, FRP exhibits a linear elastic behavior up to failure, whose characteristic value is f_{fk}.

The maximum design strain allowed to the FRP system shall be expressed as follows:

$$\varepsilon_{\mathrm{fd}} = \min\left\{\eta_{\mathrm{a}} \cdot \frac{\varepsilon_{\mathrm{fk}}}{\gamma_{\mathrm{f}}}, \varepsilon_{\mathrm{fdd}}\right\} \tag{5.1}$$

where $\varepsilon_{\mathrm{fk}}$ represents the FRP characteristic strain at failure, and $\varepsilon_{\mathrm{fdd}}$ is the maximum FRP strain once FRP debonding takes place (see next section). Unless more accurate data is available, $\varepsilon_{\mathrm{fdd}}$ shall be taken from item (7) of Section 5.3.2. The values to be assigned to the conversion factor, η_{a}, and the partial factor, $\gamma_{\mathrm{m}} = \gamma_{\mathrm{f}}$, are indicated in Table 3-4 and Table 3-2, respectively.

(12)P Design recommendations are based on limit-states-design principles. For ultimate limit states analysis, two possible approaches may be recognized depending upon the type of structural analysis performed. If non linear models are used, the member's load carrying capacity shall be larger than the factored applied load. The latter is computed according to the current building code. Care shall be taken to ensure that the proposed solution is not affected by the particular discretization adopted for the computation. If linear elastic models or simplified schemes adopting a balanced distribution of stresses that satisfy equilibrium conditions but not necessarily compatibility of strain are used, the resulting stress on each structural member shall be verified. In particular, for bi-dimensional members (slabs, shells), the unit stress shall be considered (*e.g.*, those evaluated per unit length of the member). Assuming that a plane section before loading remains plane after loading, the design criteria is met when factored shear forces and bending moments due to the applied loads are smaller than the corresponding design factored shear and flexural capacities. The latter shall be evaluated as a function of the applied axial force, considering the non linear behavior of the material represented by the simplified stress-strain diagram introduced in item (8)P.

(13)P Other limit states shall be verified according to the current building code requirements.

5.3 EVALUATION OF DEBONDING STRENGTH

(1)P Bond between masonry and FRP is of great relevance because debonding yields to undesirable, brittle failure modes. When designing according to the capacity design criterion, FRP debonding shall always follow the post-elastic behavior of compressed masonry. If special anchorage devices for the FRP system are used, failure by FRP debonding is accepted, provided that the variation of the resisting scheme is accounted for.

(2) Due to the wide variety of existing masonry structures (*e.g.*, artificial clay, concrete masonry blocks, squared or non-squared stones, etc.), debonding may occur at the interface between different materials. Moreover, in masonry structures with irregular faces, a layer of mortar may be used to create a suitable surface for FRP application. The same strengthening system may then be linked to different materials characterized by different interface properties.

(3) If the tensile strength of the adhesive used to install FRP reinforcement is larger than that of the substrate, debonding between FRP and masonry takes place at the masonry face-level.

5.3.1 General considerations and failure modes

(1)P Debonding between FRP reinforcement applied in isolated strips along straight lines and masonry may occur according to the following failure modes: plate end debonding or intermediate crack debonding. In masonry structures strengthened with FRP and loaded such to provide tensile stresses in FRP reinforcement both at laminate ends as well as close to the locations of existing cracks, the FRP-masonry interface undergoes high stresses localized within 50 to 200 mm from the discontinuity section.

(2) Combined stresses reduce the bond strength. In particular, when FRP strengthening is applied to curved surfaces or when high flexural stiffness FRP reinforcement is used, significant tensile stresses perpendicular to the masonry-FRP interface (peeling stresses) arise, thus reducing the force that may be transferred. In case of FRP laminates applied with fibers inclined with respect to the orthogonal direction of the crack, stresses around cracks generated by relative movements produce a stress concentration at the masonry-FRP interface.

(3) Shear debonding takes place close to FRP reinforcement ends (anchorage sections) and may be followed by removal of a significant portion of brick (rip-off failure), specifically when shear stresses at the FRP end are combined with normal tensile stresses. Such failure mode appears with formation of cracks due to spreading of anchorage stresses that may be accompanied by tensile stresses in the masonry responsible for its fracture (Figure 5-1).

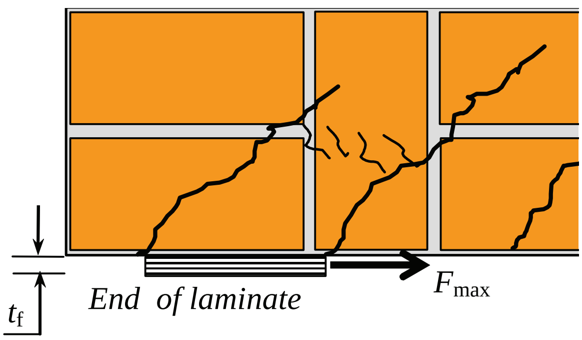


Figure 5-1 – Failure due to rip-off of the anchorage brick.

5.3.2 Bond strength at ultimate limit state

(1)P Experimental bond tests show that the ultimate value of the force transferred from FRP reinforcement to the support prior to FRP debonding depends on the length, l_b, of the bonded area. This value grows with l_b up to a maximum corresponding to a length l_e; further increase of the bonded area does not increase the force that it is possible to transfer. The length l_e is called optimal bond length and corresponds to the minimal bond length able to carry the maximum anchorage force.

(2) The optimal bond length, l_e, may be estimated as follows:

$$l_e = \sqrt{\frac{E_f \cdot t_f}{2 \cdot f_{mtm}}} \qquad \text{[lengths in mm]} \tag{5.2}$$

where:
- E_f is the Young modulus of elasticity of FRP reinforcement
- t_f represents FRP thickness
- f_{mtm} is the masonry average tensile strength; and unless specific data is available, it may be assumed equal to $0.10 f_{mk}$ (in particular, because bond between FRP and masonry is generally granted, the value of f_{mtm} to be considered in Equation (5.2) is the average tensile strength of the masonry).

(3) If debonding involves the first masonry layers, the characteristic value, Γ_{Fk}, of the specific fracture energy of the FRP strengthened masonry shall be given as follows:

$$\Gamma_{Fk} = c_1 \cdot \sqrt{f_{mk} \cdot f_{mtm}} \qquad \text{[forces in N and lengths in mm]} \tag{5.3}$$

where c_1 is an experimentally determined coefficient. Unless specific data is available, the value of c_1 may be assumed equal to 0.015.

(4) When debonding involves the first masonry layers and the bond length is longer or equal to the optimal bond length, the design bond strength, f_{fdd}, shall be expressed as follows:

$$f_{fdd} = \frac{1}{\gamma_{f,d} \cdot \sqrt{\gamma_M}} \cdot \sqrt{\frac{2 \cdot E_f \cdot \Gamma_{Fk}}{t_f}} \tag{5.4}$$

where $\gamma_{f,d}$ is the partial factor as per Table 3-2 (Section 3.4.1), and γ_M is the partial factor of the masonry.

(5) The maximum design strain, ε_{fdd}, before FRP debonding takes place is given by the ratio between the design bond strength, f_{fdd}, and the FRP Young modulus of elasticity, E_f.

(6) For bond lengths, l_b, smaller than l_e, the design bond strength shall be reduced as follows:

$$f_{fdd,rid} = f_{fdd} \cdot \frac{l_b}{l_e} \cdot \left(2 - \frac{l_b}{l_e}\right) \tag{5.5}$$

(7) If debonding mechanism between FRP and masonry takes place by rupture of a portion of the masonry unit, it shall be assumed that each masonry unit can not concur for more than 80 % of its length to the determination of the above mentioned lengths l_b and l_e.

(8) When special anchorage devices (FRP transverse bars, FRP end wrapping) are used, the

maximum bond force shall be evaluated with experimental investigations.

5.3.3 Bond strength with stresses perpendicular to the surface of bond

(1) Experimental data carried out on FRP strengthened masonry structures should be used for the determination of the bond strength in case of stresses perpendicular to the bonding surface.

(2) Unless a more detailed analysis is carried out, bond strength of low curvature FRP strengthened masonry structures should be evaluated by reducing the value of the force transferred from the FRP system to the support depending upon the magnitude of the stress perpendicular to the bonding surface.

(3) The design value of the bond strength with stresses perpendicular to the surface of bond can be expressed as follows:

$$f_{\mathrm{fpd}} = f_{\mathrm{fdd}} \cdot \left(1 - \frac{\sigma_{\mathrm{Sd}}}{f_{\mathrm{mtd}}}\right) \tag{5.6}$$

where f_{fdd} is the design value of the bond strength given in Equation (5.4), f_{mtd} is the design value of the tensile strength of the masonry, and σ_{Sd} represents the magnitude of the stress perpendicular to the bonding surface. If the bond length, l_{b}, is smaller than the optimal length, l_{e}, the value $f_{\mathrm{fdd,rid}}$ shall be used in Equation (5.6).

(4) The magnitude of the stress perpendicular to the bonding surface, σ_{Sd}, for FRP strengthening of curved profile having bending radius r and subjected to a constant tensile stress σ_{f}, can be calculated as follows:

$$\sigma_{\mathrm{Sd}} = \sigma_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot \frac{1}{r} \tag{5.7}$$

5.4 SAFETY REQUIREMENTS

(1) Principles as stated in Section 5.2 are hereafter referred to practical applications.

5.4.1 Strengthening of masonry panels

(1) Masonry panels may be strengthened with FRP to increase their load carrying capacities or their ductilities for in-plane or out-of-plane loads. In the following, simple requirements to control the degree of safety of the strengthened masonry panel are suggested. Such requirements are not exhaustive and should be integrated with further analysis suitable to the complexity of the case studied.

5.4.1.1 Strengthening for out-of-plane loads

(1) Out-of-plane collapse of masonry panels is one of the most frequent mechanism of local crisis for masonry structural members. Such mechanism is primarily due to seismic actions and secondary to horizontal forces originated by the presence of arches and vaults. Out-of-plane collapse can appear as any of the following:

- Simple overturning.
- Vertical flexure failure.
- Horizontal flexure failure.

5.4.1.1.1 Simple overturning

(1) The kinematic motion is represented by an overturning about a hinge at the bottom of the masonry panel. Due to the small tensile strength of the masonry, the hinge is usually located on the outer surface of the panel. Collapse by overturning may happen in the presence of walls not connected to the orthogonal walls nor restrained at their top. Collapse by overturning may depend on several factors, such as boundary conditions, slenderness of the wall, and geometry of masonry member. A possible retrofitting technique may consist in the use of FRP applied to the top portion of the masonry panel and properly anchored into the orthogonal walls. The optimal solution from a performance point of view, would be embracing the entire perimeter of the building with the selected FRP system. Particular care shall be taken in the rounding of masonry corners to avoid stress concentration in the FRP.

As an example, a masonry panel subjected to the following loads (design values) is considered:

- P_d panel self weight.
- N_d axial force acting on top of the panel.
- Q_d horizontal load due to seismic effects.
- F_d force exerted on the masonry panel by the FRP system.

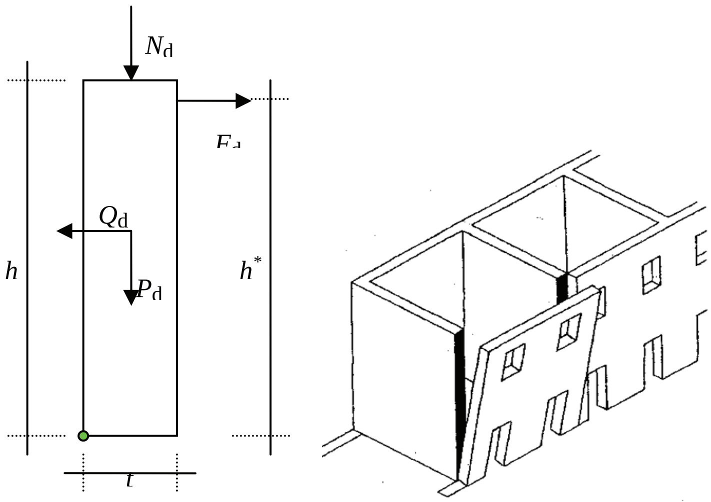


Figure 5-2 – Collapse mechanism by simple overturning.

Assuming that floors and walls perpendicular to the panel being studied provide negligible restraint to the panel itself (Figure 5-2), tensile force in the FRP reinforcement can be calculated via moment equilibrium as follows:

$$F_d = \frac{1}{2 \cdot h^*} \cdot \left(Q_d \cdot h - N_d \cdot t - P_d \cdot t \right) \tag{5.8}$$

where h^* is the distance between FRP and the bottom portion of the masonry panel. To prevent simple overturning of the masonry panel, the two following conditions shall be met:

- <u>FRP tensile strength</u>
  It shall be verified that

$$F_d \le 2 \cdot F_{Rd} \tag{5.9}$$

where $F_{Rd} = A_f \cdot f_{fd}$, A_f represents the FRP reinforcement area, and f_{fd} is the design tensile strength of the FRP.

- Rip-off of FRP from orthogonal walls
  It shall be verified that

$$F_{\mathrm{d}} \leq 2 \cdot F_{\mathrm{pd}} \tag{5.10}$$

  where $F_{\mathrm{pd}} = A_{\mathrm{f}} \cdot f_{\mathrm{pd}}$ represents the maximum anchorage force of the FRP applied to one of the two orthogonal walls.

Usually, rip-off is more demanding than tensile strength on FRP. In the case of a fully wrapped structure with an appropriate overlapping length, the second condition becomes unnecessary. Additional requirements include determination of stresses due to combined axial load and bending moment as well as determination of shear force on panel horizontal sections according to Section 5.4.1.2.

5.4.1.1.2 Vertical flexural failure

(1) For masonry panels restrained at both top and bottom regions and subjected to horizontal loading, failure may occur due to flexure with formation of three hinges: one at the bottom of the panel, one at the panel top and the last at a certain height of the panel itself. Failure occurs when the force corresponding to the applied axial load and bending moment falls outside to the masonry cross section. Flexural collapse may occur in very high masonry panels and/or panels restrained far apart from orthogonal walls. In case of seismic loads, masonry panels loaded from opposite side by floors located at different heights are particularly sensitive to flexural collapse mechanisms. Such masonry panels may be strengthened with FRP having fibers running in the vertical direction. As an example, a unit width strip of masonry panel strengthened with FRP and subject to the following external loading (design values) is considered:

- $P_{\mathrm{d}}^{(\mathrm{s})}$ weight of the upper side of the panel.
- $P_{\mathrm{d}}^{(\mathrm{i})}$ weight of the lower side of the panel.
- $Q_{\mathrm{d}}^{(\mathrm{s})}$ seismic force related to the upper side of the panel.
- $Q_{\mathrm{d}}^{(\mathrm{i})}$ seismic force related to the lower side of the panel.
- N_{d} axial force acting on the panel.
- Q_{d} load due to a further horizontal loading condition.

By force equilibrium, the horizontal reaction in C may be calculated as follows (Figure 5-3):

$$H_{\mathrm{C,d}} = \frac{h_{\mathrm{i}} \cdot (2 \cdot Q_{\mathrm{d}} + Q_{\mathrm{d}}^{(\mathrm{i})}) + Q_{\mathrm{d}}^{(\mathrm{s})} \cdot (2 \cdot h - h_{\mathrm{s}}) - t \cdot (N_{\mathrm{d}} + P_{\mathrm{d}}^{(\mathrm{s})} + P_{\mathrm{d}}^{(\mathrm{i})})}{2 \cdot h} \tag{5.11}$$

The masonry panel at cross-section B where the FRP is applied to prevent formation of the hinge is subjected to axial force and bending moment equal to the following:

$$\begin{aligned} N_{\mathrm{Sd}} &= N_{\mathrm{d}} + P_{\mathrm{d}}^{(\mathrm{s})}, \\ M_{\mathrm{Sd}} &= H_{\mathrm{C,d}} \cdot h_{\mathrm{s}} - Q_{\mathrm{d}}^{(\mathrm{s})} \cdot \frac{h_{\mathrm{s}}}{2}. \end{aligned} \tag{5.12}$$

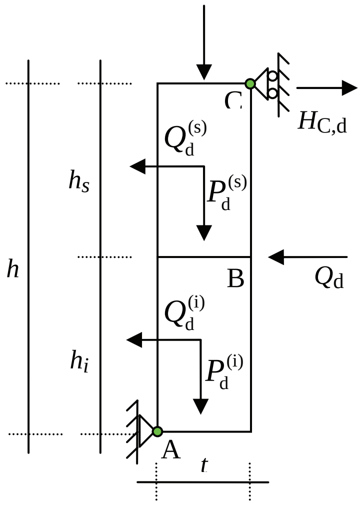


Figure 5-3 – Collapse mechanism by vertical flexure.

The masonry panel flexural capacity is verified when the following relationship is met:

$$M_{\mathrm{Sd}} \leq M_{\mathrm{Rd}} \tag{5.13}$$

The flexural capacity of the strengthened masonry panel, M_{Rd}, may be determined as a function of the mechanical characteristics of masonry and FRP, the thickness t of the masonry panel, and the value of the applied axial force (the partial factor for resistance models, γ_{Rd}, can be taken from Table 3-3 of Section 3.4.2; for this particular case it shall be assumed equal to 1.00). The compressive stress-strain relationship for masonry can be assumed to be rectangular with a uniform compressive stress of $0.85 \cdot f_{\mathrm{md}}$ distributed over an equivalent compression zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis, x, at a distance of 0.6÷0.8 x. The maximum masonry and FRP strain shall be taken as indicated in Section 5.2.3.

FRP vertical reinforcements shall be placed at a center-to-center distance, p_{f}, such that:

$$p_{\mathrm{f}} \leq 3 \cdot t + b_{\mathrm{f}} \tag{5.14}$$

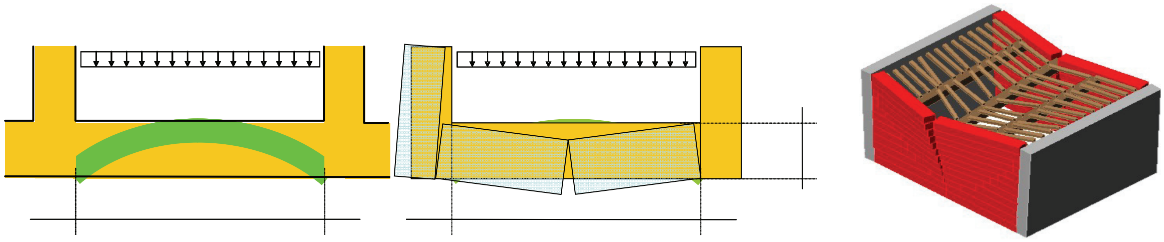
where b_{f} is the FRP width. Larger center-to-center spacing can be used only if adequately justified.

5.4.1.1.3 Horizontal flexural failure

(1) For masonry panels restrained at the bottom as well as firmly connected with transversal walls, the resistance to horizontal forces is ensured by arching effect of the top strip as illustrated in Figure 5-4(a). The value of the maximum uniformly distributed horizontal load, q, that can be carried out can be expressed as follows:

$$q = \frac{2 \cdot t^2}{L^2} \cdot f_{\mathrm{md}}^{\mathrm{h}} \tag{5.15}$$

where L is the panel width, and $f_{\mathrm{md}}^{\mathrm{h}}$ represents the design compressive strength of the masonry in the horizontal direction. If the applied load is larger than the value of q given by Equation (5.15), the panel would collapse due to crushing of the masonry. In such a case, the application of a FRP strengthening system could be beneficial. For masonry panels restrained at the bottom, but not firmly connected with transversal walls, the masonry panel may collapse as illustrated in Figure 5-4(b). With reference to the unit strip located on top of the panel, failure occurs when the force corresponding to the applied axial load and bending moment falls outside to the masonry cross section.



(a) (b)

Figure 5-4 – Collapse by horizontal flexure.

The use of FRP prevents such mechanisms from failure providing flexural capacity to the mentioned unit strip can be considered a masonry beam strengthened with FRP. The applied bending moment, M_{sd}, is due to earthquake loads, wind pressure, and other possible horizontal loads due to the presence of other structural members. Flexural safety of the masonry panel is satisfied when Equation (5.13) is met. The moment M_{rd} may be determined as a function of the mechanical characteristics of masonry and FRP and the thickness t of the masonry panel. Unless a more detailed analysis is available, the horizontal force due to the presence of transversal walls may be considered equal to zero when evaluating M_{rd}.

An additional shear check shall be carried out on the connection joints between masonry panel and transversal walls by taking into account the two-dimensional effect of the entire panel. Verification of the magnitude of tensile loads on transversal walls close to the main masonry panel shall also be performed.

5.4.1.2 Strengthening for in-plane loads

(1) The following checks shall be carried out for masonry panels subjected to in-plane loads:

- In-plane combined bending and axial load.
- Shear force.

5.4.1.2.1 In-plane combined bending and axial load

(1) FRP materials can effectively enhance the in-plane capacity with regard to combined bending and axial load of masonry panels when installed on both sides of the panel where tension loads are expected. Such FRP reinforcements shall be adequately anchored to the end portions of the masonry panel.

(2) Assuming that sections perpendicular to the axis of bending which are plane before bending remain plane after bending, the compressive stress-strain relationship for masonry can be assumed to be rectangular with a uniform compressive stress of $0.85 \cdot f_{md}$ distributed over an equivalent compression zone bounded by the edges of the cross section and a straight line located parallel to the neutral axis, x, at a distance of 0.6÷0.8 x. The maximum masonry and FRP strain shall be taken as indicated in Section 5.2.3.

5.4.1.2.2 Shear force

(1P) The shear capacity of masonry panels strengthened with FRP applied on both sides of the panel can be seen as the combination of two resisting mechanisms: (1) shear forces due to friction in presence of compression loads, and (2) for elements capable of resisting tensile stress a truss mechanism becomes active, and shear forces are carried out by equilibrium.

(2) Usually, shear strengthening with FRP of masonry panels is obtained by applying external reinforcement to carry tensile loads due to flexure and shear force such to allow the formation of the mentioned truss mechanism. When FRP reinforcement is not applied to carry flexural loads, shear strengthening with FRP can be carried out by installing external reinforcement along the panel diagonals.

(3) When formation of truss mechanism is ensured, the design shear capacity, V_{Rd}, of the FRP strengthened masonry panel is computed as the sum of the masonry contribution, $V_{\text{Rd,m}}$, and the FRP contribution, $V_{\text{Rd,f}}$, up to the maximum value $V_{\text{Rd,max}}$ inducing failure of the compressed strut of the truss:

$$V_{\text{Rd}} = \min\left\{ V_{\text{Rd,m}} + V_{\text{Rd,f}}, V_{\text{Rd,max}} \right\} \tag{5.16}$$

If shear strengthening is placed parallel to the mortar joints, the above defined contributions may be evaluated as follows:

$$V_{\text{Rd,m}} = \frac{1}{\gamma_{\text{Rd}}} \cdot d \cdot t \cdot f_{\text{vd}} \tag{5.17}$$

$$V_{\text{Rd,f}} = \frac{1}{\gamma_{\text{Rd}}} \cdot \frac{0.6 \cdot d \cdot A_{\text{fw}} \cdot f_{\text{fd}}}{p_{\text{f}}} \tag{5.18}$$

where:
- γ_{Rd} is the partial factor to be assumed equal to 1.20 (Table 3-3, Section 3.4.2).
- d is the distance between the compression side of the masonry and the centroid of FRP flexural strengthening.
- t is the masonry panel thickness.
- f_{vd} is the design shear strength of the masonry, equal to $f_{\text{vk}}/\gamma_{\text{M}}$.
- A_{fw} is the area of FRP shear strengthening in the direction parallel to the shear force.
- p_{f} is the center-to-center spacing of FRP reinforcement measured orthogonally to the direction of the shear force.
- f_{fd} is the design strength of FRP reinforcement, defined as the lesser between FRP tensile failure strength and debonding strength.

The value of the partial factor for masonry, γ_{M}, shall be set according to the current building code; the partial factor for resistance model, γ_{Rd}, shall be obtained from Table 3-3 of 3.4.2; for shear it is set equal to 1.20. If the angle of friction, ϕ, of mortar joints is smaller than 45°, the value of $V_{\text{Rd,f}}$ provided by Equation (5.18) shall be reduced by a multiplicative factor of $\cot(90° - \phi)$.

(4) The design shear capacity, $V_{\mathrm{Rd,max}}$, of the masonry panel corresponding to failure of the compressed strut of the truss can be calculated as follows:

$$V_{\mathrm{Rd,max}} = 0.3 \cdot f_{\mathrm{md}}^{\mathrm{h}} \cdot t \cdot d \tag{5.19}$$

where $f_{\mathrm{md}}^{\mathrm{h}}$ is the design compressive strength of the masonry parallel to the mortar joints.

(5) In case of FRP flexural strengthening of masonry panels, shear capacity enhancement due to the increased compressive strength of the masonry can be calculated by computing the value of the parameter f_{vk} related to the average compressive strength (including the one due to flexure) acting on the masonry panel.

(6) If shear strengthening is not placed parallel to the mortar joints, Equations (5.17) and (5.18) valid for FRP strengthening placed parallel to the mortar joints, should be carefully evaluated with suitable models.

5.4.2 Lintel and tie areas

(1)P The areas connecting different wall bays within a masonry panel are named tie areas. Their function is twofold: (1) restrain the adjoining wall to assume deformed shapes compatible with the applied horizontal load, and (2) support the masonry wall located above openings. The former mechanism generates shear and flexural stresses within the tie area itself and plays a significant role in case of seismic loads; the latter is played by lintels located above any opening that are primary subjected to vertical loads.

(2)P Due to the presence of vertical loads, two effects in the areas above openings are displayed: (1) the portion of masonry wall above the opening can not withstand its own weight and shall be supported by lintel functioning as a beam; and (2) when the wall bays surrounding the openings are slender so as not to withstand the horizontal load due to the presence of the opening itself, the lintel shall be such to provide adequate strength to carry the tensile stresses to ensure the overall equilibrium of the wall.

(3) P In the next two sections, three methods for designing both lintels and tie areas subjected to seismic loads are presented (see Figure 5-5).

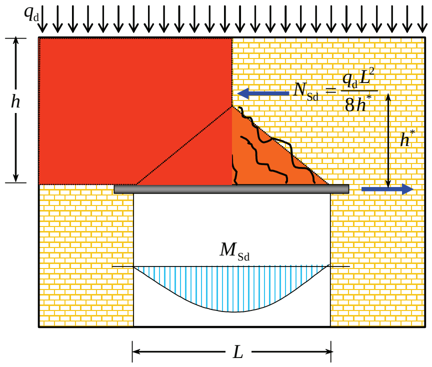


Figure 5-5 – Lintels design subjected to combined bending and axial loads.

5.4.2.1 Design of lintels

(1) Lintels may be realized using structural elements having both axial and flexural capacity; alternatively, lintels having only axial capacity can be employed. In the former case, lintels are capa-

ble of functioning as a beam and carry tensile stresses to ensure the overall equilibrium of the wall; in the latter, support to the wall above the opening shall be ensured by formation of a reinforced masonry member located just above the opening where tensile stresses are carried out by the applied FRP strengthening system. In this case, equilibrium condition is probably reached with remarkable displacements showed by the masonry wall above the strengthened member. Such FRP strengthening system shall be applied on the bottom portion of the lintel; unless appropriate explanation is given, the selected FRP system shall not be installed on the sides of the masonry wall.

(2) To ensure proper functioning of lintels, FRP reinforcement shall be properly anchored into the adjacent masonry walls.

(3) The portion of the FRP strengthened lintel shall have a flexural capacity, M_{Rd}, larger than the applied moment, M_{Sd}. The latter can be calculated as follows:

$$M_{\text{Sd}} = \gamma_{\text{G}} \cdot \frac{1}{24} \cdot g \cdot t \cdot L^3 \tag{5.20}$$

where g is the masonry weight per cubic meter, t is the thickness of the masonry, L is the net span of the opening, and γ_{G} is the partial factor for self weight at ultimate limit state.

FRP shall also be capable to withstand the following force:

$$N_{\text{Sd}} = \frac{q_d \cdot L^2}{8 \cdot h^*} \tag{5.21}$$

where q_{d} is the design vertical load at ultimate limit state acting on the lintel (sum of factored dead and live loads), and h^* is the internal lever arm, to be assumed not larger than the span L of the opening nor the height h of the tie area.

5.4.2.2 Design of tie areas

(1) FRP strengthened tie areas shall be verified for bending moment, shear, and axial loads acting at the connection with vertical masonry walls. Flexural and shear capacity shall be calculated according to the rules given for the masonry wall panels by tacking into account the compressive strength of the masonry f_{md}^{h} parallel to the mortar joints.

(2) FRP strengthening of tie areas may be carried out by installing reinforcement with fibers running in the horizontal direction located at the level of the floor; alternatively, external FRP reinforcement may be located at the top or bottom region of tie areas itself. FRP reinforcement may be continuous or discontinuous and applied to either the internal or external face of the masonry wall. When FRP reinforcement is applied to the external face of the masonry wall, it also may function as external wrapping of the building.

(3) To ensure a satisfying behavior with respect to the applied shear force, FRP reinforcement with fibers running in the diagonal direction could also be applied to tie areas. FRP reinforcement should be installed symmetrically on both internal and external sides of the strengthened masonry wall.

5.5 STRENGTHENING OF STRUCTURAL MEMBERS WITH SINGLE OR DOUBLE CURVATURE

(1)P Structural members with single or double curvature generally lose their functionality due to the formation of hinges that promote the mechanisms of collapse. Hinges form in such masonry structures due to the negligible tensile strength of the masonry.

(2)P Such hinges are located in regions of limited contact area, externally to the mid plane of the structure. As first approximation, they can be located either at the intrados or extrados of the masonry panel. A masonry hinge can carry axial and shear forces. As a result, the hinged section can only carry an axial force having an eccentricity equal to half of the structure thickness.

(3)P FRP reinforcement delays both opening of cracks and formation of hinges within the masonry panel located on the opposite side with respect to the one where the FRP system is installed. Therefore, a properly anchored FRP reinforcement applied to the extrados (intrados) prevent the formation of hinges on the opposite side of the intrados (extrados). FRP reinforcement is not recommended when collapse is controlled by either crushing of the masonry or shear failure.

(4)P FRP reinforcement used as external strengthening of masonry structures is such to prevent the formation of certain hinges.

5.5.1 Arches

(1)P Two structural schemes can be taken into consideration:

- Arch scheme, for arches resting on fixed and/or hinged supports.
- Arch-pier scheme, also known as frame scheme, for arches resting on piers.

(2)P Both schemes generally tend to collapse due to the formation of at least four hinges. In particular, a possible mechanism may be due to the formation of three (real) hinges and a double pendulum (pseudo-hinge) leading to a shear failure of a portion of the arch with respect to the other.

5.5.1.1 Arch scheme

(1)P To prevent the mechanism characterized by the formation of four hinges, FRP reinforcement may be bonded either to the extrados or the intrados of the masonry arch. Experimental evidence shows that application of FRP reinforcement on the side surface of the arch does not provide significant improvement of the structural behavior. In such a case, a premature debonding of the FRP reinforcement from the masonry face takes place. Such debonding is localized in the arch compressed region and it is due to local FRP instability, followed by a fast degradation of the bond between masonry and FRP.

(2) FRP strengthening is preferably carried out by applying FRP reinforcement on the extrados of the masonry arch to prevent the formation of hinges on the intrados. Alternatively, FRP reinforcement may be applied on the arch intrados to prevent the formation of hinges on the extrados. As a final remark, FRP reinforcement may also be applied to both extrados and intrados of the masonry arch to prevent the formation of first and second type-hinge. However, this application is not common.

(3) Unless the formation of hinges is prevented in the region of the masonry arch close to supports, when computing internal forces of the strengthened arch the formation of hinges at supports shall always be considered.

(4) Partial FRP strengthening carried out on a portion of the extrados or intrados does not pre-

vent the possibility of formation of hinges responsible for the activation of a kinematic mechanism of the structure. However, when FRP strengthening is properly designed and realized, it may enhance the structure's ultimate capacity. It shall be preferable to do the following:

- Carry out complete FRP strengthening on the extrados or intrados of the arch.
- Choose FRP fabric over laminate, because they better fit the geometry of the masonry arch.
- Apply FRP strengthening on the arch extrados; in this case the arch curvature is such to display compressive stress orthogonal to the FRP reinforcement. On the other hand, when FRP is installed on the intrados, the curvature is such to display tensile stress orthogonal to the FRP reinforcement that enhance debonding between FRP and masonry.

(5) When computing internal forces of the FRP strengthened masonry arch, the formation of hinges located on the opposite side with respect to the side of FRP installation shall be taken into account. A more realistic approach should consider that the hinge will form at a certain distance to the side located at the opposite face of FRP reinforcement. Such a distance depends on the masonry compressive strength; it increases as the masonry compressive strength decreases.

(6)P When masonry tensile stresses can be neglected, the following checks shall be performed for FRP strengthened arches:

- Overall stability of the structure.
- Combined bending and axial force, when failure occurs by either crushing of the masonry and/or FRP rupture.
- Shear.
- FRP debonding.

(7) Combined bending and axial force as well as shear check shall be in compliance with the procedure indicated for masonry panels. Check for FRP debonding shall be performed at a distance l_b from the end of FRP reinforcement; the moment corresponding to such section shall be evaluated using the FRP design strength according toSection 5.3.3 item (3).

5.5.1.2 Arch-pier scheme

(1) For arch-pier structures, application of FRP reinforcement to the arch intrados or extrados may not be sufficient to prevent relative displacements of the pier-arch connections. In such a case, it is preferable to either act on the piers or set a tie rod between the pier-arch connections.

(2)P Checks to be carried out are the same as those considered for the arch scheme.

5.5.2 Single curvature vaults: barrel vaults

(1)P In most situations, the study of barrel vaults is similar to that of a unit depth arch. Consequently, barrel vaults may be strengthened with FRP applied both on the extrados and intrados. To satisfy safety requirements, FRP strengthening shall be applied along the entire longitudinal length of the vault. For this reason, FRP reinforcement shall be placed at a center-to-center distance, p_f, calculated as follows:

$$p_f \leq 3 \cdot t + b_f \tag{5.22}$$

where t is the vault thickness and b_f is the FRP width. A greater distance is allowed only if properly substantiated.

(2) Longitudinal FRP strengthening has only the secondary importance of bridging the ideal arches forming the barrel vault. Such mechanism is particularly important in cases of horizontal loading.

(3) Typically, it is suggested to install in the longitudinal direction at least 10 % of the FRP reinforcement applied in the transversal direction. It shall be increased to 25 % for FRP strengthening in seismic area.

(4) If vaults are used in cellular buildings with small-size rooms, FRP strengthening should be performed on the building walls rather than the vault.

5.5.3 Double curvature vaults: domes

(1)P Domes exhibit membrane-type and flexural-type stresses.

5.5.3.1 Membrane-type stresses

(1) In a dome subjected to vertical loads, normal tensile stresses directed along the dome parallels are displayed. The typical cracking pattern with cracks located along the meridians is primarily due to the negligible tensile strength of the masonry. The mentioned crack pattern modifies the equilibrium condition of the dome enhancing the horizontal forces where the dome connects with the supporting structure. The use of FRP reinforcement applied in a circle around the lower portion of the dome's perimeter may help in preventing the opening of cracks as well as reducing the magnitude of the horizontal force acting on the supporting structure.

(2)P The degree of safety of a masonry dome shall be performed by checking the following:

- Tensile stress in FRP reinforcement.
- FRP debonding according to Section 5.3.3.

5.5.3.2 Flexural-type stresses

(1) Flexural-type stress is typically localized where the dome meets the supporting structure or at the edge of skylight, when available. In particular, flexural-type stress may cause collapse of portions of the dome delimitated by meridian cracks. If the load carrying capacity of such portions is controlled by failure of the region connecting the dome to the supporting structure, the dome may be strengthened by applying FRP reinforcement in a circle around the lower portion of the dome perimeter. If the dome supporting structure does not show any displacement, the above mentioned FRP circular strengthening is inactive. In such a case, FRP reinforcement shall be applied along the dome meridians.

(2) P The degree of safety of a masonry dome shall be performed by checking the following:

- Combined bending and axial force.
- Shear.
- FRP debonding.

For combined bending and axial loads as well as shear check, internal forces shall be evaluated on a unit dome element according to Sections 5.4.1.2.1 and 5.4.1.2.2. Possible strength reductions for the loading carrying capacity of the strengthened dome shall be considered due to the complexity of the internal forces associated to the analysis of dome structures. Precautions shall be taken in case of combined bending and axial load when the tensile zone in one direction corresponds to a compression zone in the opposite direction. In such a case, unless a more rigorous analysis is performed, the ratio of the absolute value of the design applied moment to the nominal moment calculated under the applied axial load shall not be larger than 1. On the contrary, unless a more rigorous analysis is performed, the specific flexural capacity in each plane can be assumed equal to the one resulting from a monoaxial loading condition.

Planar shear design can be performed according to the first of the two cases previously mentioned.
It is to be noted that flexural and shear capacity shall be calculated with reference to the design compressive strength of the masonry by taking into account differences due to loading perpendicular or parallel to the masonry texture (Section 5.2.3 item (6)P). Othogonal shear design can not take into account the presence of FRP reinforcement and shall be performed as in the case of unreinforced masonry considering the complexity of the existing internal forces. Checks for FRP debonding shall consider tensile stresses acting perpendicular to the FRP reinforcement according to Section 5.3.3.

(3) To ensure proper behavior of the FRP system applied in a circle around the lower portion of the dome perimeter, FRP reinforcement shall be accurately anchored to the dome supporting structure eventually by means of mechanical anchorage.

5.5.4 Double curvature vaults on a square plane

(1) FRP strengthening of double curvature vaults resting on a square plane shall primarily be performed on the masonry walls of the room that support the vault itself. For vertically loaded structures, integrity and stiffness of the supporting masonry walls ensure that the vault is primarily subjected to compression stresses. If this is not the case, FRP strengthening may be performed within the corner region of the vaults where tensile stress is expected to exist in a direction orthogonal to the room diagonals.

5.6 CONFINEMENT OF MASONRY COLUMNS

(1)P FRP confinement of masonry columns subjected to axial loads increases both ultimate capacity and strain. It also may enhance the column performance under service limit state.

(2)P FRP reinforcement is typically installed by wrapping of the member; such wrapping exerts a beneficial effect on the lateral strain of the column by providing a tri-axial confinement. FRP strengthening may be employed either in case of rehabilitation of deteriorated structures or for their seismic upgrade.

(3) Confinement with composites may be performed by using FRP sheets or bars. FRP sheets are applied as external reinforcement along the perimeter of the member to be strengthened in form of continuous or discontinuous wrap. Instead, FRP bars are inserted in spread holes drilled through the member that requires upgrade.

(4) FRP bars are inserted in holes drilled along two directions orthogonal to the member transversal cross section. Each set of two bars inserted in one of the two directions represents a “layer of bars” (Figure 5-6). Such reinforcement can effectively contrast the transversal strain of the masonry. To ensure continuity between FRP bars and surrounding masonry, each hole is filled with epoxy paste or, alternatively, FRP bar ends are mechanically fastened to the masonry.

(5)P When FRP sheets and bars are used in the same application for strengthening masonry columns, it is recommended that such FRP materials exhibit similar mechanical characteristics.

(6) Confinement with FRP of masonry structures shall be performed using design mechanical parameters in compliance with the current building code.

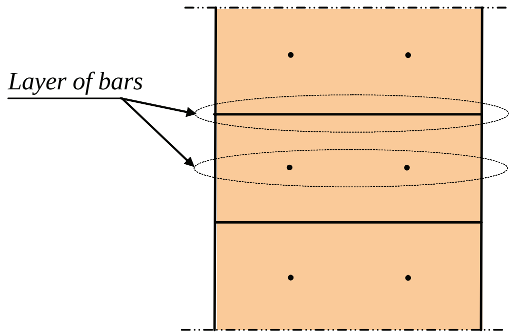


Figure 5-6 – Lateral view of a column with FRP bars arranged along two orthogonal directions.

5.6.1 Design of axially loaded confined members

(1)P Design of FRP confined masonry columns is based upon the appropriateness of the FRP system as a function of the member geometry.

(2)P It is recommended to install FRP reinforcement with fibers running in the orthogonal direction with respect to the vertical axis of the strengthened member. Effectiveness of spiral FRP reinforcement shall be adequately proven.

(3)P The axial capacity of FRP strengthened member, $N_{\mathrm{Rmc,d}}$, shall exceed the design axial force due to applied loads calculated according to the current building code, N_{Sd}, as follows:

$$N_{\mathrm{Rmc,d}} \geq N_{\mathrm{Sd}} \tag{5.23}$$

(4)P $N_{\mathrm{Rmc,d}}$ is given as follows:

$$N_{\mathrm{Rmc,d}} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot A_{\mathrm{m}} \cdot f_{\mathrm{mcd}} \geq A_{\mathrm{m}} \cdot f_{\mathrm{md}} \tag{5.24}$$

where the partial factor, γ_{Rd}, shall be set equal to 1.10 (Table 3-3, Section 3.4.2), A_{m} represents the cross sectional area of the FRP confined member, f_{md} represents the design compressive strength of unconfined masonry, and f_{mcd} is the design compressive strength of FRP confined member.

(5) The design compressive strength, f_{mcd}, for members confined with FRP subjected to a lateral confining pressure, f_1, can be written as follows:

$$f_{\mathrm{mcd}} = f_{\mathrm{md}} + k' \cdot f_{\mathrm{l,eff}} \tag{5.25}$$

where k' is a non-dimensional coefficient, and $f_{\mathrm{l,eff}}$ represents the effective confining pressure.

(6) Unless a more detailed analysis is performed, k' may be calculated as follows:

$$k' = \frac{g_{\mathrm{m}}}{1000} \tag{5.26}$$

where g_{m} is the masonry mass-density expressed as kg/m^3.

(7) The effective confining pressure, $f_{1,\mathrm{eff}}$, is a function of cross section shape and the FRP system. Defining V_{m} as the volume of the masonry member to be strengthened, and $V_{\mathrm{c,eff}}$ as the portion of the effectively confined volume, the following coefficient of efficiency can be written:

$$k_{\mathrm{eff}} = \frac{V_{\mathrm{c,eff}}}{V_{\mathrm{m}}} \tag{5.27}$$

The effective confining pressure may be defined as a function of the coefficient of efficiency. In turn, this may be expressed as the product of a horizontal and vertical coefficient of efficiency, k_{H} and k_{V}, respectively:

$$f_{\mathrm{l,eff}} = k_{\mathrm{eff}} \cdot f_{\mathrm{l}} = k_{\mathrm{H}} \cdot k_{\mathrm{V}} \cdot f_{\mathrm{l}} \tag{5.28}$$

(8) When spiral FRP sheets are used, the effectiveness of FRP confinement is penalized by fibers' inclinations. Indicating with α_{f} the FRP fiber inclination with respect to the horizontal plane of the member cross section, the following coefficient can be defined:

$$k_{\alpha} = \frac{1}{1 + \mathrm{tg}^2 \alpha_{\mathrm{f}}} \tag{5.29}$$

Such coefficient shall multiply the lateral confining pressure, f_1, reported in Equation (5.28). FRP strengthening performed with FRP bars inserted in holes shall not be affected by such coefficient.

(9) To restrict axial deformation and prevent damage at serviceability limit state, the increased axial capacity due to FRP confinement shall not be larger than 50 % of the design compressive strength, f_{md}, of the unconfined member.

5.6.2 Confinement of circular columns

(1) The geometric ratio of FRP confined members when both FRP sheets and bars are employed can be defined as follows:

$$\rho_{\mathrm{f}} = \frac{4 \cdot t_{\mathrm{f}} \cdot b_{\mathrm{f}}}{D \cdot p_{\mathrm{f}}}, \qquad \rho_{\mathrm{b}} = \frac{n_{\mathrm{b}} \cdot A_{\mathrm{b}}}{D \cdot p_{\mathrm{b}}} \tag{5.30}$$

where:

- t_{f} is the FRP thickness.
- b_{f} is the FRP strip width.
- D is the masonry cross-section diameter.
- p_{f} is the center-to-center spacing of FRP strips.

- n_b is the number of bars installed in the generic layer of bars (for circular cross section it is assumed that all layers are realized with the same number of bars).
- A_b is the cross sectional area of each FRP bar.
- p_b is the center-to-center distance between two subsequent layers of bars along the same direction.

In case of continuous FRP wrapping, the ratio ρ_f becomes equal to $4 \cdot t_f / D$.

(2) Via equilibrium, the confining pressure, f_1, can be calculated as follows:

$$f_l = \frac{1}{2} \cdot (\rho_f \cdot E_f + 2 \cdot \rho_b \cdot E_b) \cdot \varepsilon_{fd,rid} \tag{5.31}$$

where E_f and E_b are the Young modulus of elasticity of FRP sheets and bars, respectively, and $\varepsilon_{fd,rid}$ represents the reduced design value of the FRP strain measured at column collapse.

(3)P In case of combined use of FRP sheets and bars, the reduced design strain for FRP reinforcement can be written as follows:

$$\varepsilon_{fd,rid} = \min\left\{\eta_a \cdot \varepsilon_{fk}^{(r)} / \gamma_f^{(r)}; \eta_a \cdot \varepsilon_{fk}^{(b)} / \gamma_f^{(b)}\right\} \tag{5.32}$$

where η_a is the environmental conversion factor (Table 3-4); $\varepsilon_{fk}^{(r)}$ and $\varepsilon_{fk}^{(b)}$ represent ultimate strain of FRP sheets and bars, respectively; and $\gamma_f^{(r)}$ and $\gamma_f^{(b)}$ are the partial factors of FRP sheets and bars, respectively (Table 3-2; for FRP bars, not included in the table, a value of $\gamma_f^{(b)} = 1.50$ is recommended).

(4) For circular cross section strengthened with FRP sheets, the horizontal coefficient of efficiency, k_H, can be assumed equal to 1. For continuous vertical FRP wrapping, the coefficient of vertical efficiency, k_V, is assumed equal to 1. For spiral FRP wrapping, the effectiveness of FRP strengthening shall appropriately be reduced by means of the k_α coefficient (Eq. (5.29)).

(5) When discontinuous FRP reinforcement having a strip width b_f and center-to-center spacing p_f is used, the coefficient of vertical efficiency, k_V, can be assumed as follows (Figure 5-7):

$$k_V = \left(1 - \frac{p_f'}{2 \cdot D}\right)^2 \tag{5.33}$$

where p_f' is the net distance between FRP strips.

(6) Confinement with FRP bars inserted in holes drilled through the member are less effective than external wrapping with FRP sheets. Such applications shall therefore be carefully analyzed. When FRP bars are used for strengthening masonry columns, staggering of bars along the vertical direction is necessary; moreover, the center-to-center distance between different layers of bars shall

not exceed $D/5$.

(7) Similar to discontinuous wrapping with FRP sheets, the reduction of confinement effectiveness of masonry columns strengthened with FRP bars is to be attributed to the stress diffusion phenomenon, which may be described by a parabolic relationship with a starting slope of 45° located at the termination point of the FRP reinforcement. Unless a more detailed analysis is available, the coefficient of efficiency defined by Equation (5.27) can be evaluated from Equation (5.33) with p_f' replaced by the center-to-center spacing between FRP bars, p_b.

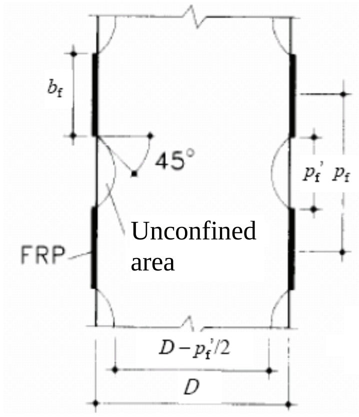


Figure 5-7 – Front view of circular masonry member confined with discontinuous FRP strips.

5.6.3 Confinement of prismatic columns

(1)P FRP confinement of non-circular cross sections shows only a slight increase in the load carrying capacity. Such applications shall therefore be carefully analyzed.

(2)P Prior to FRP wrapping of the masonry member, a minimum 20 mm radius (rounding radius) when the sheet is wrapped around outside corners shall be provided.

(3) The confining pressure, f_1, of a rectangular member having dimension *b*x*d* can be calculated as follows:

$$f_1 = \frac{1}{2} \cdot \min\left\{\rho_{f,x} \cdot E_f + 2 \cdot \rho_{b,x} \cdot E_b;\ \rho_{f,y} \cdot E_f + 2 \cdot \rho_{b,y} \cdot E_b\right\} \cdot \varepsilon_{fd,rid} \tag{5.34}$$

where the non-dimensional parameters $\rho_{f,x}$, $\rho_{f,y}$, $\rho_{b,x}$, and $\rho_{b,y}$ are defined as follows (Figure 5-8):

$$\rho_{f,x} = \frac{4 \cdot t_f \cdot b_f}{d \cdot p_f}, \quad \rho_{f,y} = \frac{4 \cdot t_f \cdot b_f}{b \cdot p_f}, \quad \rho_{b,x} = \frac{n_{b,x} \cdot A_b}{p_b \cdot d}, \quad \rho_{b,y} = \frac{n_{b,y} \cdot A_b}{p_b \cdot b} \tag{5.35}$$

and $n_{b,x}$ and $n_{b,y}$ represent the number of bars in the x and y direction, respectively.

Only for the case of continuous FRP wrapping, the confining pressure can be calculated according to Equation (5.31), assuming $\rho_f = 4 \cdot t_f / \max\{b, d\}$ and $\rho_b = 0$.

(4) Figure 5-8 shows a rectangular cross section confined with a continuous FRP reinforcement.

Due to the arch-effect displayed in the figure, the confined section is only a portion of the total area of the masonry column. The extension of the confined area depends on the adopted rounding radius.

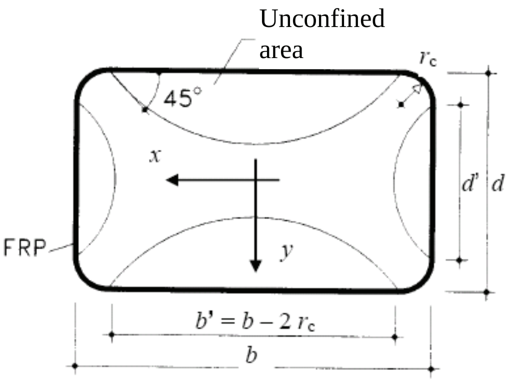


Figure 5-8 – Confinement of rectangular sections externally wrapped with FRP.

(5) The horizontal coefficient of efficiency is given by the ratio between the confined area and the total area of the masonry column, A_m, as follows:

$$k_H = 1 - \frac{b'^2 + d'^2}{3 \cdot A_m} \tag{5.36}$$

where b' and d' are dimensions indicated in Figure 5-8.

(6) Unless experimental testing is available, FRP confinement of rectangular member with aspect ratio (b/d) larger than 2 or when $\max\{b,d\} > 900\text{mm}$ shall be neglected.

(7)P The combined use of external FRP wrapping and internal FRP bars inserted in holes drilled through the member cross section may increase the area effectively confined for square, rectangular, or more complex cross sections (Figure 5-9).

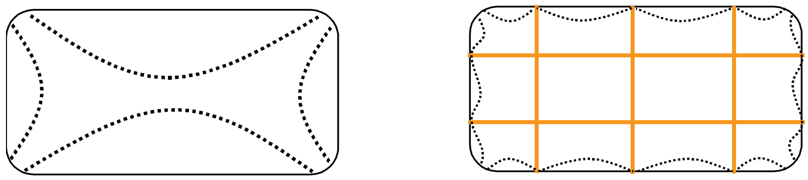


Figure 5-9 – Confinement of masonry members with FRP sheets and bars.

(8) When discontinuous FRP reinforcement has a strip width b_f and center-to-center spacing p_f is used, the coefficient of vertical efficiency, k_V, can assumed as follows (Figure 5-7):

$$k_V = \left(1 - \frac{p'_f}{2 \cdot \min\{b,d\}}\right)^2 \tag{5.37}$$

For spiral FRP wrapping, the effectiveness of FRP strengthening shall appropriately be reduced by means of the k_α coefficient (Eq. (5.29)).

(9) When FRP bars are used for strengthening masonry columns, staggering of bars along the

vertical direction is necessary. With reference to Figure 5-10, it can be assumed that the area of the effectively confined masonry is reduced with respect to the total member area due to the “arch effect” between different layers of FRP reinforcement.

(10) The reduction of the effectively confined area between layers of different FRP reinforcement is due to the stress diffusion phenomenon described by a parabolic relationship with a starting slope of 45° (Figure 5-10).

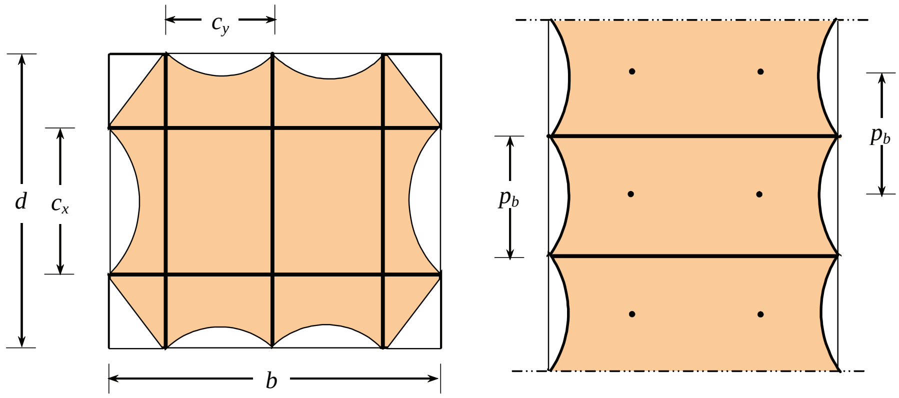


Figure 5-10 – Plan and lateral view of confinement with FRP bars.

(11) Unless a more appropriate determination of the portion of the effective confined volume is made, the coefficient of efficiency, k_{eff}, according to Equation (5.27) can be assumed as follows:

$$k_{\text{eff}} = k_{\text{H}} \cdot k_{\text{V}} = \left[1 - \frac{1}{6}\cdot\left(2\cdot\frac{n_{\text{bx}}-1}{n_{\text{bx}}^2}\cdot\frac{d}{b} + 2\cdot\frac{n_{\text{by}}-1}{n_{\text{by}}^2}\cdot\frac{b}{d} + \frac{3}{n_{\text{bx}}\cdot n_{\text{by}}}\right)\right]\cdot\left(1 - \frac{p_{\text{b}}}{2\cdot\min\{b,d\}}\right)^2 \tag{5.38}$$

In case of square cross sections, the coefficient of efficiency reduces to the following:

$$k_{\text{eff}} = k_{\text{H}} \cdot k_{\text{V}} = \left(1 - \frac{1}{6}\cdot\frac{4\cdot n_{\text{b}}-1}{n_{\text{b}}^2}\right)\cdot\left(1 - \frac{p_{\text{b}}}{2\cdot b}\right)^2 \tag{5.39}$$

where b is the member width and $n_{\text{bx}} = n_{\text{by}} = n_{\text{b}}$.

(12) FRP bars inserted in holes drilled through the strengthened masonry member shall be anchored for a length at least equal to 10 times the FRP bar diameter. When such length is greater than 1/5 of the FRP bar length, the anchorage force should be adequately spread at the two bar ends.

(13) Horizontal and vertical FRP bar spacing shall not be greater than half of the width of the strengthened member; similarly, the distance between the member edge and the FRP bar closer to the edge itself shall not be larger than 1/4 of the member width.

5.7 DESIGN FOR SEISMIC APPLICATIONS

5.7.1 Design objectives

(1)P FRP strengthening of masonry structures subjected to seismic loads can be performed when the unstrengthened member does not satisfy one or more limit states according to the current building code.

(2)P This part of the document recognizes the provisions of the current building code as well as the indications provided in the most updated literature related to seismic constructions; particular importance is given to the following:

- Evaluation of seismic safety.
- Safety requirements (verification of limit states).
- Levels of seismic protection (magnitude of the associated seismic action).
- Methods of analysis.
- Verification criteria (distinction between ductile and brittle members).
- Materials characteristics to be used for design.

5.7.2 Selection criteria for FRP strengthening

(1)P Type and size of selected FRP systems shall take into account the following:

- Masonry structure unable to withstand vertical and horizontal loads shall be strengthened or replaced.
- Walls ending on masonry T-junctions or masonry edges shall be appropriately connected.
- Unsatisfactory connections between floors/roof and vertical walls shall be made effective.
- Horizontal forces generated from roofs, arches, and vaults shall be taken by appropriate structural members.
- Floors effectively connected to vertical walls shall be properly stiffened in their plane to be able to transfer horizontal forces to the vertical walls located in the earthquake direction. They also shall provide restraint to the movement of vertical walls located in the earthquake orthogonal direction.
- Weak members for which strengthening is not appropriate shall be eliminated.
- In case of strongly irregular buildings (in term of resistance and/or stiffness), FRP strengthening is usually unable to provide relief to the structure. It may be used for few structural members to grant a minimum regularity to the structure.
- FRP strengthened members where local ductility is enhanced are always recommended.
- Local FRP strengthening shall never reduce the overall ductility of the structure.

(2)P FRP retrofitting is typically aimed at the following:

- Total or partial strengthening, replacing, or rebuilding of structural members.
- Modifying the overall structural behavior by means of connection of different structural members.

(3)P Design of FRP reinforcement shall include the following:

- Rational choice of the retrofitting technique.
- Choice of the appropriate technique and/or material.
- Preliminary dimensioning of FRP reinforcement.
- Structural analysis, taking into account the FRP strengthened structure.
- Safety checks of the strengthened structure performed on strengthened and newly added members (for existing, repaired, or strengthened members safety checks shall be carried out according to this Guideline; for newly added members, safety checks shall be in compliance with the current building code).

(4)P FRP strengthening in a seismic area shall be aimed at the following:

- Increase flexural and shear capacity of masonry panels ensuring transfer of tensile stresses within single members or between adjacent members.

- Eliminate horizontal forces applied orthogonally to the masonry walls.
- Connect members resisting horizontal loads to achieve a box-like behavior of the structure.
- In-plane floors stiffening to achieve a stiff diaphragm behavior.
- Limiting crack width to improve energy dissipation.
- Confining columns to increase material strength and ductility.

(5)P The FRP-based retrofitting strategy shall follow the principle of increasing the capacity of under-designed members, with the aim of achieving at the same time a greater structural regularity and elimination of possible local collapse of masonry walls or structural components.

(6) Seismic efficiency of the designed retrofitting technique may be measured by the increase of horizontal displacement of the strengthened member close to collapse.

(7)P To avoid seismic vulnerability of masonry structures, care shall be taken to ensure that the FRP system will not decrease the overall ductility of the structure. Particular care shall be taken to remedial work aiming at joining vertical columns to avoid formation of hinges in arches and vaults. It is preferable to increase hinges ductility of both masonry columns and vaults. Similarly, for bracing walls of ordinary buildings, FRP reinforcement shall be placed such to increase the overall structural ductility and to make sure that the collapse of tie areas will anticipate the rupture of columns.

5.8 INSTALLATION, MONITORING, AND QUALITY CONTROL

(1)P Several aspects influence the effectiveness of FRP material used as externally bonded systems for strengthening of masonry members. In addition to those discussed in previous chapters, surface preparation and FRP installation will be dealt with in this section. The relative importance of each of these aspects depends on whether reference is made to applications defined as “bond-critical” (flexure or shear) or “contact-critical” (confinement). For example, some tests on the quality of the substrate can be omitted for contact-critical applications (*e.g.*, confinement by wrapping) or when the FRP system is properly anchored provided that its effectiveness is tested by an independent laboratory.

(2)P Once FRP strengthening has been carried out, over time monitoring by non destructive or semi-destructive tests as listed in the following sections shall be performed to ensure the effectiveness of the proposed strengthening solution.

(3)P This document describes the tests that can be carried out for quality control as well as over time monitoring of the selected FRP system. The type and number of tests to be performed shall be linked to the importance of the application by taking into account the following:

- For strategic buildings when their functionality during seismic events is of fundamental importance for civil protection or when their roles become relevant due to the consequences of a possible collapse.
- Historical and cultural value.
- Whether FRP application concerns primary structural members (*e.g.*, vaults, domes, columns, arches, walls) or secondary elements.
- The extent of FRP application compared to the size of the structure.

(4)P The numerical values indicated in the following are to be understood as suggested values.

5.8.1 Quality control and substrate preparation

(1)P Quality control of the support implies determination of the masonry conditions, removal, and reconstruction of any deteriorated or loose masonry block, cleaning, and removal of a portion of masonry subjected to moisture, vegetation plants, or similar.

(2)P When special devices are used to properly anchor the selected FRP system, testing of such devices shall be conducted in compliance with available standardization documents. Anchoring devices shall be installed according to the manufacturer/supplier specifications regarding both material used as well as surface preparation, environmental conditions, and sequence of each phase. The investigation shall also evaluate the effect of such parameters on the final result.

5.8.1.1 Evaluation of substrate deterioration

(1) Prior to FRP application, tests on the homogeneity of the portion of the structure to be strengthened shall be performed to ensure proper quality of the masonry support.

(2) Mechanical characterization tests on masonry shall be at least 1 every 100 m^2 of area to be strengthened, with a minimum of 2 tests for each homogeneous area. Tests shall be performed according to at least of one of the following categories:

- Compression test on a masonry specimen.
- Shear test on a masonry specimen.
- Flat jack test.
- Shear test by jack.
- Dilatometer test.
- Ultrasonic test.

(3) When homogeneity tests are performed on the entire area to be strengthened with the exception of critical areas, they shall be distributed according to a square mesh spaced 1 m apart for areas smaller than 5 m^2, and proportionally increased for larger areas. Tests shall be performed as follows:

- Hand hammering of the interested area.
- X-ray analysis.
- Ultrasound speed in near-surface mode.
- Recording speed of sonic pulse (with instrumented hammer and accelerometers).
- Penetrometry.
- Thermography.
- Tomography.

5.8.1.2 Removal and reconstruction of defective masonry support

(1) Masonry substrate may have undergone physical-chemical, physical-mechanical or impact-caused deterioration. Deteriorated masonry shall be removed from all damaged areas

(2) Removal of defective masonry allows examining characteristics of both natural or artificial masonry as well as mortar. When exfoliation, pulverization, cracking, or chemical attack processes are in place, it is necessary to remove all defective areas and protect them with appropriate inhibitors.

(3) Once all deteriorated masonry has been removed, and suitable measures have been taken to prevent further deterioration of the existing substrate as well as all other phenomena causing masonry degradation (*e.g.*, water leakage, vegetation), masonry restoration using masonry-compatible materials shall be performed. Roughness of masonry between 10 and 20 mm shall be leveled with compatible epoxy paste; specific filling material shall be used for unevenness larger than 20 mm.

Crack widths wider than 0.5 mm shall be closed using epoxy injection methods before FRP strengthening can take place.

(4) To improve the bond between masonry support and FRP, sandblasting of the portion of masonry surface to be strengthened shall be performed. Sandblasting shall provide a roughness degree of at least 0.3 mm; such level of roughness can be measured by suitable instruments, such as a laser profilometer or an optical profile-measuring device.

(5) Poor quality masonry surfaces that do not require remedial work before FRP application should be treated with a reinforcing agent prior to primer installation.

(6) Cleaning of the surface to be strengthened shall remove any dust, laitance, oil, surface lubricants, foreign particles, or any other bond-inhibiting material.

(7) All inside and outside corners and sharp edges that require FRP strengthening shall be rounded or chamfered to a minimum radius of 20 mm.

5.8.2 Recommendations for the installation

(1) FRP strengthening of masonry structures is highly dependant upon environmental temperature and humidity as well as the characteristics of the substrate. In addition to the above mentioned measure, and regardless of the strengthening type, further specific measures to ensure quality of FRP installation are highlighted in the next sections.

5.8.2.1 Humidity and temperature conditions in the environment and substrate

(1) It is suggested not to install FRP material when the environment is very moist; a high degree of humidity may delay curing of the resin and affect the overall performance of the FRP system especially for wet lay-up applications.

(2) FRP systems shall not be applied to substrates having a surface humidity level greater than 10 %; such conditions could delay penetration of the *primer* in the concrete pores and generate air bubbles that could compromise bond between concrete and FRP system. Substrate humidity may be evaluated with a hygrometer for mortar or simply by employing absorbent paper.

(3) FRP material shall not be applied if both environment and surface temperature is too low, because curing of the resins as well as fibers impregnation could be compromised. It is also recommended not to install FRP material when the masonry surface is heavily exposed to sunlight. It is suggested not to apply any FRP material if temperatures fall outside the range of 10° to 35°C. In low temperature environments where the construction site schedule does not allow FRP installation to be delayed, it is suggested to artificially heat the locations where FRP reinforcement is to be applied.

(4) If curing of FRP reinforcement takes place under rainy conditions, heavy insulation, large thermal gradients, or in presence of dust, protective measures can be employed to ensure proper curing.

5.8.2.2 Construction details

(1) Anchorage lengths of at least 300 mm shall be provided for the end portion of FRP systems used for strengthening masonry members. Alternatively, mechanical connectors may be used.

(2) Proper fibers alignment shall be provided for in-situ wet lay-up application; waving of FRP

reinforcement shall also be avoided during installation.

(3) When semi-destructive tests are planned, it is suggested to provide additional strengthening areas ("witness areas") in properly selected portions of the structure having dimensions of at least 500x300 mm^2, with a minimum extension of 0.15 m^2 nor less than 0.5 % of the overall area to be strengthened. Witness areas shall be realized at the same time of the main FRP installation, using the same materials and procedures in areas where removal of FRP strengthening system does not imply alteration of the failure mechanisms. In addition, witness areas shall be exposed to the same environmental conditions of the main selected FRP system and shall be uniformly distributed on the strengthened structure.

5.8.2.3 Protection of FRP systems

(1) For outdoor FRP applications it is recommended to protect the FRP system from direct sunlight, which may produce chemical-physical alterations in the epoxy matrix. This can be achieved by using protective acrylic paint provided that cleaning of the composite surface with a sponge soaked in soap is performed.

(2) Alternatively, a better protection can be achieved by applying plastering or mortar layer (preferably concrete-based) to the installed strengthening system. The plaster, whose thickness is recommended by the FRP manufacturers/suppliers, is to be laid on the strengthening system after treating the surface by means of epoxy resin applications with subsequent quartz dusting green-on-green. The final layer is particularly suitable to receive any kind of plastering.

(3) For fire protection, two different solutions may be adopted: use of intumescent panels, or application of protective plasters. In both cases, manufacturers/suppliers shall indicate the degree of fire protection as a function of the panel/plaster thickness. The panels -generally based on calcium silicates- are applied directly on the FRP strengthening system, provided that fibers will not be cut during their installation. Protective plasters represent the most widely adopted solution for fire protection; they shall be applied to the FRP system as indicated before. Protective coatings of adequate thickness and consistency capable of keeping the composite temperature below 80°C for 90 minutes are available.

5.8.3 Quality control during installation

(1) Quality control during FRP installation should include at least one cycle of semi-destructive tests for the mechanical characterization of the installation itself, and at least one non destructive mapping to ensure its uniformity.

5.8.3.1 Semi-destructive tests

(1) Both pull-off tests and shear tearing tests may be carried out. Semi-destructive tests shall be carried out on witnesses and, where possible, in non-critical strengthened areas at the rate of one test for every 5 m^2 of application, and, in any case, not less than 2 per each type of test.

(2) <u>Pull-off test</u>. The test is used for assessment of the properties of the restored substrate; it is carried out by using a 20 mm thick circular steel plates with a diameter of at least 40 mm. After the steel plate is firmly attached to the FRP, such steel plate is isolated from the surrounding FRP with a core drill rotating at a speed of at least 2500 rpm; particular care shall be taken to avoid heating of the FRP system while a 1-2 mm incision of the masonry substrate is achieved.

FRP application may be considered acceptable if at least 80 % of the tests (both in case of only two tests) return a pull-off stress not less than 10 % of the compressive strength of the support provided that failure occurs in the substrate itself.

(3) Shear tearing test. The test is particularly significant to assess the quality of bond between FRP and masonry substrate. It may be carried out only when it is possible to pull a portion of the FRP system in its plane located close to an edge detached from the masonry substrate. FRP application may be considered acceptable if at least 80 % of the tests (both in the case of two tests) return a peak tearing force not less than 5 % of the compressive strength of the support.

5.8.3.2 Non destructive tests

(1) Non destructive tests may be used to characterize the uniformity of FRP application starting from adequate two-dimensional survey of the strengthened surface with a different spatial resolution as a function of the strengthening area (see Table 5-1).

Table 5-1 – Minimum resolution for defects thickness to be identified with non destructive tests.

Shear stress transfer at interface	Example	Non destructive test	Surface mapping grid	Minimum resolution for defects thickness
absent	wrappings, with the exception of the overlapping area in single-layer application	optional	250 mm	3 mm
weak	central area of very extensive reinforcement	optional	250 mm	3 mm
moderate	central area of longitudinal flexural strengthening	suggested	100 mm	0.5 mm
critical	anchorage areas, overlapping areas between layers, stirrups for shear strengthening, interface areas with connectors, areas with large roughness or cracks in the substrate	required	50 mm	0.1 mm

(2) Stimulated Acoustic testing. Similar to impact-echo testing, such tests rely on the different oscillatory behavior of the composite layer depending on the bond between FRP layers and substrate. In its most basic version, such a test may be carried out by a technician hammering the composite surface and listening to the sound from the impact. More objective results may be obtained with automated systems.

(3) High-frequency ultrasonic testing. They should be carried out using reflection methods with frequencies no less than 1.5 MHz and probes with a diameter no greater than 25 mm, adopting the technique based on the first peak amplitude variation to localize defects.

(4) Thermographic tests. They are effective only for FRP systems with low thermal conductivity and can not be applied to carbon or metallic FRP strengthening systems unless special precautions are taken. The heat developed during the test shall be smaller than the glass transition temperature of the FRP system.

(5) Acoustic emission tests. The technique is based on the acoustic emission (AE) method and allows the assessment of a damage inside a structural member subjected to loading by listening to and recording the sound generated by either formation of cracks or delamination phenomena that propagates as elastic waves. Such a test is particularly suitable for detecting defects in the application of FRP composites masonry structures as well as the occurrence of delamination from the substrate.

5.8.4 Personnel qualification

(1) Personnel in charge of the tests shall have one of the three qualification levels specified in

Table 5-2, according to UNI EN 473 and UNI EN 45013.

Table 5-2 – Qualification levels to perform semi and non-destructive tests.

Level 1	Proper knowledge of tests equipment; performing tests; recording and classifying test results according to written criteria; writing a report on test results.
Level 2	Choosing the way of performing the test; defining the application limits of the test for which the level 2 technician is certified; understanding test specifications and translating them into practical test instructions suitable to the in-situ working conditions; adjusting and calibrating test equipments; performing and controlling the test; interpreting and evaluating test results according to the specifications to comply with; preparing written test instructions for level 1 personnel; performing and supervising all level 1 functions; training personnel of level 1; organizing test results and writing the final report.
Level 3	Be in charge of a laboratory facility; establishing and validating test techniques and procedures; interpreting specifications and procedures; having the skill to evaluate and understand test results according to existing specifications; having a sufficient practical knowledge of materials, production methods and installation technology of the system to be tested to be able to choose appropriate methods, establish techniques and collaborate in the definition of acceptance criteria when they are not pre-established; be knowledgeable in different application fields; being able to lead personnel of level 1 and 2.

5.8.5 Monitoring of the strengthening system

(1) Due to the poor availability of data regarding long term behavior of FRP systems used for strengthening masonry structures, it is recommended to perform appropriate monitoring of the installed FRP system by means of semi and non-destructive tests periodically conducted on the strengthened structure. The aim of such a monitoring process is to keep the following parameters under control:

- Temperature of the installed FRP system.
- Environmental humidity.
- Measure of displacements and deformations of the strengthened structure.
- Potential damage of fibers.
- Extensions of defects and delaminations in the installed FRP system.

6 APPENDIX A (MANUFACTURING TECHNIQUES AND STRESS-STRAIN RELATIONSHIP OF ORTHOTROPIC LINEAR ELASTIC MATERIALS)

6.1 MANUFACTURING TECHNIQUES

6.1.1 Pultrusion

Pultrusion is a technology mainly used for production of fiber-reinforced laminates. Such products are widely utilised in civil engineering field. This technology is based on a continuous manufacturing process, consisting of three main phases:

- Forming.
- Impregnation.
- Hardening.

In the most common version designed for thermosetting resin, the components (resin and fibers) are separately fed into a machine that catches and pulls the fibers through the different production stages. A widespread version of the process includes impregnation with resin bath, as shown in Figure 6-1.

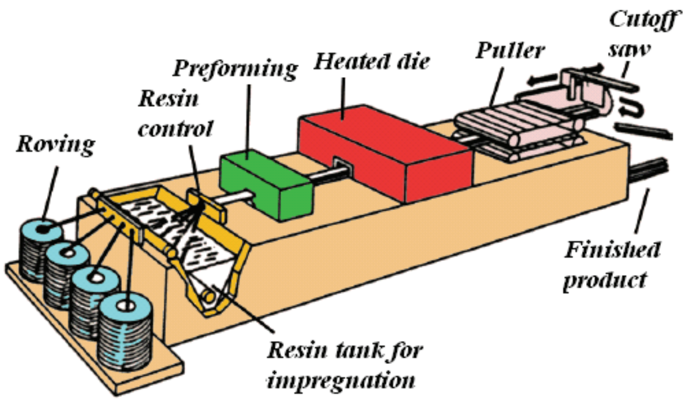


Figure 6-1 – Pultrusion process with resin bath impregnation.

The fibers are taken directly from the rovings and conveyed to a resin bath where impregnation takes place. Bundles of impregnated fibers enter the heating die where the material is formed and crosslinked under high pressure. During this phase, gaps between fibers are eliminated to ensure proper continuity in the transverse direction.

Heating is generally supplied by electrical resistances and the temperature is controlled by means of thermocouples. The duration of the heating stage is regulated by production speed. Upon exiting from the die, the matrix is cured and the composite it is pulled at a constant speed. At the end of the process the material is cut to length. Fabric layers may be added to ensure strength of FRP in directions other than the longitudinal. Weaving, winding, and twisting may be carried out directly on the production line with special equipment.

FRP pultruded material is light, corrosion-resistant, with constant cross section and thicknesses up to few centimeters. Typically pultruded products include laminates, bars, structural shapes (C, double T, etc.), panels and plates. They are used as internal and external reinforcement for existing and new structures, structural components for transportation, supports for lighting and billposters, risers for oil industry, etc.

6.1.2 Lamination

Lamination is used nearly exclusively to produce innovative and high performance composites. It is a discontinuous process that allows producing laminates of maximum thickness up to few centimeters by totally controlling fiber orientation and the complexity of the structure.

Compared to pultrusion, it allows a nearly complete freedom as to fiber orientation and curvature of produced material is concern. The main limitation regards the speed of production, which is roughly 0.5 kg/h for simple components.

The following fundamental phases can be identified in the lamination process:

a) Material preparation.
b) Lamination (cut of material, stacking of plies and compaction).
c) Vacuum bag preparation.
d) Material curing (at room temperature, oven, or autoclave).
e) Inspection (visual, by ultrasound and X-rays).
f) Finishing (cutting of edges with cutters or high pressure water jet).

Lamination may be carried out with dry fibers impregnated during field installation, or pre-impregnated fibers running in either one or multiple directions.

The next phase in the lamination process requires preparation of vacuum bag (phase c) as it is shown in Figure 6-2. The vacuum allows for a fast removal of solvents and entrapped air in the laminates prior to complete curing of the resin.

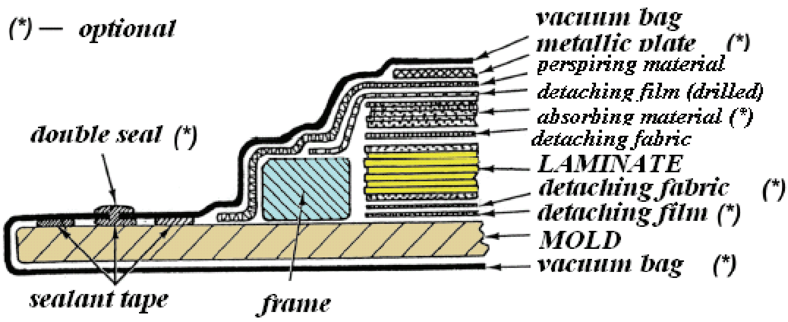


Figure 6-2 – Lamination system.

The main advantage of this technology is the extreme versatility that allows producing of complex components using inexpensive molds. Main applications refer to the aeronautical and aerospace fields, car racing, sailing, and transportation. FRP strengthening of columns or RC beams by means of dry or pre-impregnated fibers represents one field of application where lamination can effectively be used in the construction field.

6.2 MECHANICAL BEHAVIOR OF COMPOSITES

Fiber-reinforced composites are heterogeneous (*e.g.,* made of different materials) and anisotropic (*e.g.,* exhibiting different properties when tested in different directions) materials. Because the application related to fiber-reinforced composites for civil engineering is far greater than the material micro-structure (see Table 2-2), the heterogeneity may be neglected, and the actual material may be considered to behave homogeneously. If the stress and strain at a generic location of fiber-reinforced composite may be represented by the components of the tensor of stress $\underline{\sigma}$ (Figure 6-3) and strain $\underline{\varepsilon}$, the mechanical behavior of a homogeneous, elastic, and anisotropic solid may be defined by 21 independent elastic constants as follows:

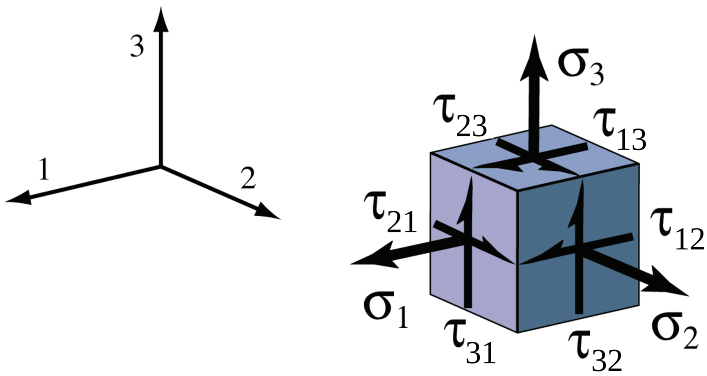


Figure 6-3 – Representation of stresses for an infinitesimal element.

$$\underline{\sigma} = [C]\underline{\varepsilon} \quad \Leftrightarrow \quad \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \sigma_3 \\ \tau_{23} \\ \tau_{31} \\ \tau_{12} \end{bmatrix} = \begin{bmatrix} C_{11} & C_{12} & C_{13} & C_{14} & C_{15} & C_{16} \\ C_{12} & C_{22} & C_{23} & C_{24} & C_{25} & C_{26} \\ C_{13} & C_{23} & C_{33} & C_{34} & C_{35} & C_{36} \\ C_{14} & C_{24} & C_{34} & C_{44} & C_{45} & C_{46} \\ C_{15} & C_{25} & C_{35} & C_{45} & C_{55} & C_{56} \\ C_{16} & C_{26} & C_{36} & C_{46} & C_{56} & C_{66} \end{bmatrix} \begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \varepsilon_3 \\ \gamma_{23} \\ \gamma_{31} \\ \gamma_{12} \end{bmatrix} \tag{6.1}$$

where $[C]$ is the stiffness matrix.

The complete characterization of the stiffness matrix would require the evaluation of the 21 constants by means of combinations of tensile and shear tests. The number of tests to be performed can significantly be reduced if the material has some degree of symmetry, a circumstance that occurs in a majority of fiber-composite materials having engineering interest.

Many unidirectional composites may be considered transversely isotropic, as it is shown in Figure 6-4, where the 2-3 plane perpendicular to fibers is the isotropy plane. In this case, the independent elastic constants reduce from 21 to 5 and the stiffness matrix becomes:

$$[C] = \begin{bmatrix} C_{11} & C_{12} & C_{12} & 0 & 0 & 0 \\ C_{12} & C_{22} & C_{23} & 0 & 0 & 0 \\ C_{12} & C_{23} & C_{22} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{2}(C_{22} - C_{23}) & 0 & 0 \\ 0 & 0 & 0 & 0 & C_{66} & 0 \\ 0 & 0 & 0 & 0 & 0 & C_{66} \end{bmatrix} \tag{6.2}$$

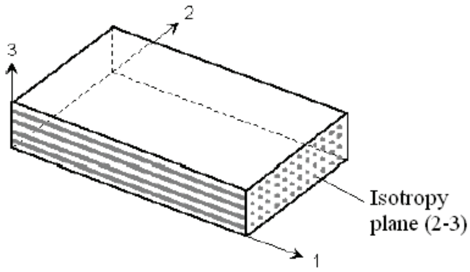


Figure 6-4 – Unidirectional composite with a transverse isotropy plane.

It is often convenient to refer to the so-called engineering constants: E (Young modulus of elasticity), ν (Poisson ratio), and G (shear modulus) for which well-established procedures for their experimental evaluation exist. Such constants have generally different values in different directions. The Young modulus of elasticity in the fiber direction, E_1, may be expected to be greater compared to the transverse direction, E_2, which in turn can be different from that in the third direction, E_3. The same consideration is applied to the modules G_{12}, G_{13}, G_{23} (directions 1, 2, and 3 are defined according to Figure 6-4).

The deformability matrix $[S]$, defined as the matrix inverse of the stiffness matrix $[C]$ (Eq. (6.2)), can be expressed as a function of the engineering constants as follows:

$$[S]=\begin{bmatrix} \frac{1}{E_1} & -\frac{\nu_{12}}{E_1} & -\frac{\nu_{12}}{E_1} & 0 & 0 & 0 \\ -\frac{\nu_{12}}{E_1} & \frac{1}{E_2} & -\frac{\nu_{23}}{E_2} & 0 & 0 & 0 \\ -\frac{\nu_{12}}{E_1} & -\frac{\nu_{23}}{E_2} & \frac{1}{E_2} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{2\cdot(1+\nu_{23})}{E_2} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{G_{12}} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{1}{G_{12}} \end{bmatrix} \tag{6.3}$$

The independent engineering constants are as follows:

$$E_1,\ E_2,\ \nu_{12},\ \nu_{23},\ G_{12}$$

For unidirectional thin laminate subjected to plane stresses, the deformability matrix becomes:

$$\begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \gamma_{12} \end{bmatrix} = \begin{bmatrix} \frac{1}{E_1} & -\frac{\nu_{12}}{E_1} & 0 \\ -\frac{\nu_{12}}{E_1} & \frac{1}{E_2} & 0 \\ 0 & 0 & \frac{1}{G_{12}} \end{bmatrix} \begin{bmatrix} \sigma_1 \\ \sigma_2 \\ \tau_{12} \end{bmatrix} \tag{6.4}$$

The mechanical behavior of unidirectional laminates can therefore be characterized by four independent elastic constants. For their determination, uniaxial tensile tests are typically carried out with fibers inclined with an angle θ with respect to the direction of the applied load. By setting $\theta = 0°$ (*e.g.*, fibers parallel to the load direction), E_1 and ν_{12} may be obtained, while with $\theta = 90°$ (fibers perpendicular to the direction of load), E_2 may be determined. G_{12} can be determined depending upon the choice of the angle θ as a function of the selected strengthening geometry. Approximate values of the mentioned elastic constants can also be determined with simple "micro-mechanical" models based on properties of each components (fibers and matrix) and their volumetric fraction. For unidirectional laminate, longitudinal properties may be evaluated by using a relationships known as the "rule of mixtures." It derives from the application of a simple micro-mechanical model where fibers and matrix work in parallel. The model provides good results for the value E_1 of the Young modulus of the elasticity in the fibers direction and the Poisson ratio ν_{12}:

$$\begin{aligned} E_1 &= V_{\text{fib}} \cdot E_{\text{fib}} + (1 - V_{\text{fib}}) \cdot E_{\text{m}}, \\ \nu_{12} &= V_{\text{fib}} \cdot \nu_{\text{fib}} + (1 - V_{\text{fib}}) \cdot \nu_{\text{m}}, \end{aligned} \tag{6.5}$$

where V_{fib} is the fiber volumetric fraction (ratio between the volume of fibers and the overall volume of composite); E_{fib} and E_{m} are the Young modulus of elasticity of fibers and matrix, respectively; and ν_{fib} and ν_{m} are the corresponding Poisson ratios.

Instead of the volumetric fraction, the weight fraction of fibers and matrix, P_{fib} and P_{m}, respectively, are more often known. If ρ_{fib} and ρ_{m} represent the density of fibers and matrix, respectively, the following relationships apply:

$$\begin{aligned} V_{\text{fib}} &= \frac{P_{\text{fib}}/\rho_{\text{fib}}}{P_{\text{fib}}/\rho_{\text{fib}} + P_{\text{m}}/\rho_{\text{m}}}, \\ P_{\text{fib}} + P_{\text{m}} &= 1. \end{aligned} \tag{6.6}$$

As an example, the computation of the volumetric fraction of fibers in a glass-fiber reinforced composite having weight fraction equal to 60 %, is presented. The characteristics of each of the components are reported in Table 6-1.

Table 6-1

	Weight fraction	Density [g/cm^3]
Fiber	0.60	2.5
Matrix	0.40	1.2

By applying Eq. (6.6), a volumetric fraction of glass fibers equal to 42 % is obtained. Considering the values of both fibers ($E_{fib} = 80\,\text{GPa}$, $\nu_{fib} = 0.3$) and matrix ($E_m = 3\,\text{GPa}$, $\nu_m = 0.34$) mechanical properties, the following values of the elastic constants can be obtained.

$$E_1 = 35.2\ \text{GPa}$$

$$\nu_{12} = 0.32$$

For more details on micro-mechanical modes the reader should refer to specialized literature.

6.2.1 Effect of loading acting on directions other than that of material symmetry

Once the elastic constants of the material are known, the behavior of fiber-reinforced material is completely determined for any loading direction irrespectively of the axes of symmetry of the material. For example, Figure 6-5 relates to a laminate with continuous unidirectional fibers; the equivalent elastic constants E_x, E_y, G_{xy} and ν_{xy}, referred to the reference axes *x* and *y* of the load system may be determined as a function of the angle θ and the elastic constants of the material $E_1, E_2, G_{12}, \nu_{12}$.

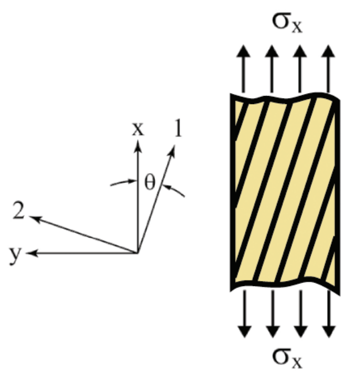


Figure 6-5 – Definition of the reference systems *x*, *y* and 1, 2.

In Figure 6-6 and Figure 6-7, the values of both Young modulus of elasticity, E_x, and shear modulus, G_{xy}, are plotted as a function of the angle, θ, between fibers and applied load, for different values of the modulus E_1.

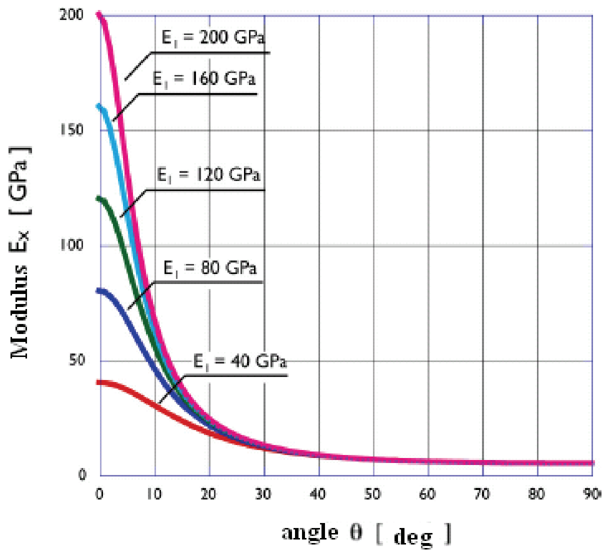


Figure 6-6 – Young modulus of elasticity E_x as a function of θ for fiber-reinforced composites for several values of the Young modulus of elasticity E_1 ($E_2 = 5$ GPa ; $G_{12} = 3$ GPa ; $\nu_{12} = 0.35$).

The significant variations of the modules E_x and G_{xy} with the angle θ are apparent.

In case of fabrics, fibers are distributed along two or more directions (multi-axial fabrics). If one were to neglect the crimping due to weaving of fibers and assuming the fabric made out of two separated unidirectional layer of fibers running at 0° and 90° direction, the modulus of elasticity, E_x, can be evaluated with simplify methods neglecting the slip between layers. For the particular case on fabric having the same percentage of fibers in the two considered directions (balanced fabric), Figure 6-8 plots the relationship between E_x and θ.

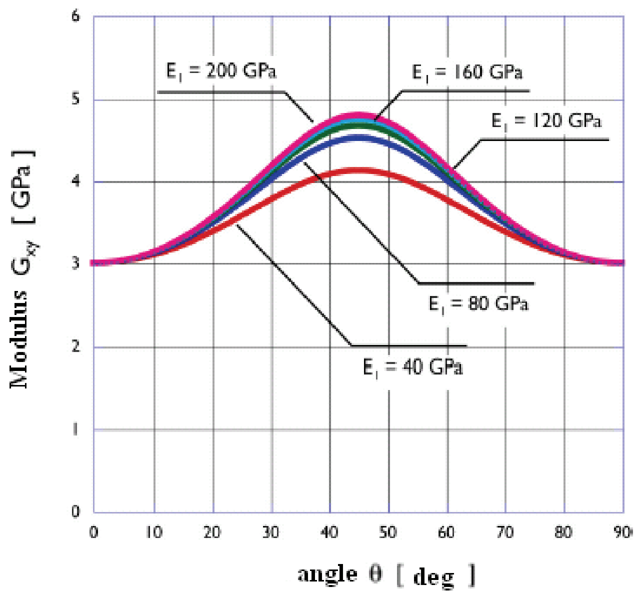


Figure 6-7 – Shear modulus G_{xy} as a function of θ for fiber-reinforced composites for several values of the Young modulus of elasticity E_1 ($E_2 = 5$ GPa ; $G_{12} = 3$ GPa ; $\nu_{12} = 0.35$).

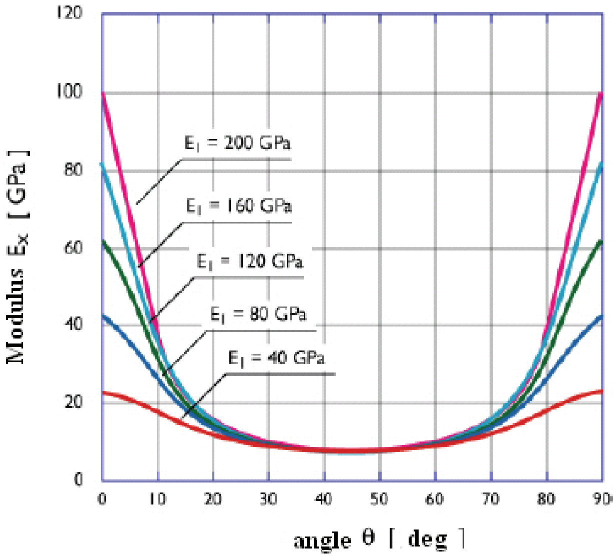


Figure 6-8 – Modulus of elasticity, E_x, as a function of θ for balanced fabric depending upon the modulus of elasticity, E_1 ($E_2 = E_1$; G_{12} = 3 GPa; ν_{12} = 0.35).

6.2.2 Failure criteria

The micro-mechanic collapse mechanism of fiber-reinforced materials is a complex phenomenon that depends on a multitude of parameters that include type of loading, and fiber and resin type. For such a reason, failure criteria for composites usually refer to the macro-mechanical level assuming that the composite itself can be regarded as a homogeneous material exhibiting a linear elastic behaviour up to collapse. In the case of laminates subjected to a planar stresses, one of the simplest failure criteria is that of the maximum stress. If $\sigma_{1u,t}(\sigma_{1u,c})$ and $\sigma_{2u,t}(\sigma_{2u,c})$ represent the tensile (compressive) failure stress in the symmetry directions, and τ_{12u} is the corresponding shear stress at failure, this criterion can be represented as follows:

$$\begin{aligned} &\sigma_1 \begin{cases} \le \sigma_{1u,t} \ \text{per } \sigma_1 > 0, \\ \ge \sigma_{1u,c} \ \text{per } \sigma_1 < 0, \end{cases} \\ &\sigma_2 \begin{cases} \le \sigma_{2u,t} \ \text{per } \sigma_2 > 0, \\ \ge \sigma_{2u,c} \ \text{per } \sigma_2 < 0, \end{cases} \\ &|\tau_{12}| \le \tau_{12u}. \end{aligned} \tag{6.7}$$

The criterion does not depend on the sign of the shear stress nor does it consider the interactions between different failure modes. Different failure modes can occur irrespectively one from another. The maximum stress that the laminate can withstand is given by the lowest among the following values (Figure 6-5):

$$\begin{aligned} \sigma_{xu} &< \frac{\sigma_{1u}}{\cos^2\theta}, \\ \sigma_{xu} &< \frac{\sigma_{2u}}{\sin^2\theta}, \\ \sigma_{xu} &< \frac{\tau_{12u}}{\sin\theta \cdot \cos\theta}. \end{aligned} \tag{6.8}$$

Figure 6-9 shows the variation of σ_{xu} as a function of θ.

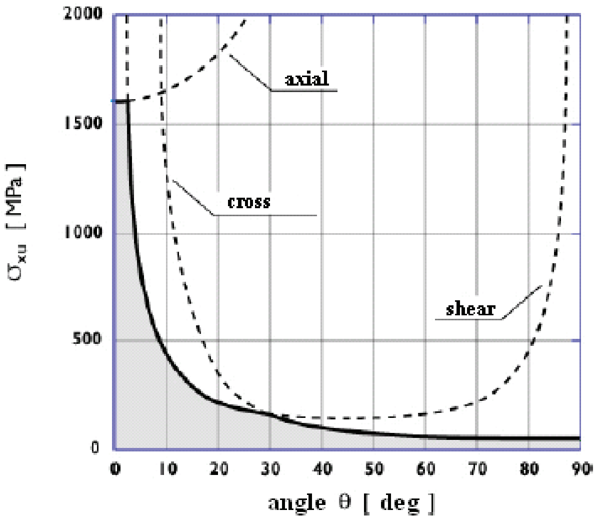


Figure 6-9 – Criterion of maximum stress: Tensile failure stress as a function of θ for unidirectional laminates ($\sigma_{1u} = 1600$ MPa , $\sigma_{2u} = 40$ MPa , $\tau_{12u} = 70$ MPa).

The criterion of the maximum stress is usually in good agreement with experimental data only for tensile test with θ smaller than 15° and larger than 45°. Otherwise, the measured values for compression are quite higher. Another widely used criterion to estimate the failure of a laminate is due to Tsai-Hill, which may be expressed as follows:

$$\left(\frac{\sigma_1}{\sigma_{1u}}\right)^2 + \left(\frac{\sigma_2}{\sigma_{2u}}\right)^2 - \frac{\sigma_1 \cdot \sigma_2}{\sigma_{1u}^2} + \left(\frac{\tau_{12}}{\tau_{12u}}\right)^2 \leq 1 \tag{6.9}$$

The stress at failure as function of θ can be written as follows (Figure 6-5):

$$\sigma_{xu} = \left[\frac{\cos^4\theta}{\sigma_{1u}^2} + \left(\frac{1}{\tau_{12u}^2} - \frac{1}{\sigma_{1u}^2}\right)\cos^2\theta \cdot \sin^2\theta + \frac{\sin^4\theta}{\sigma_{2u}^2}\right]^{-1/2} \tag{6.10}$$

and it is plotted in Figure 6-10.

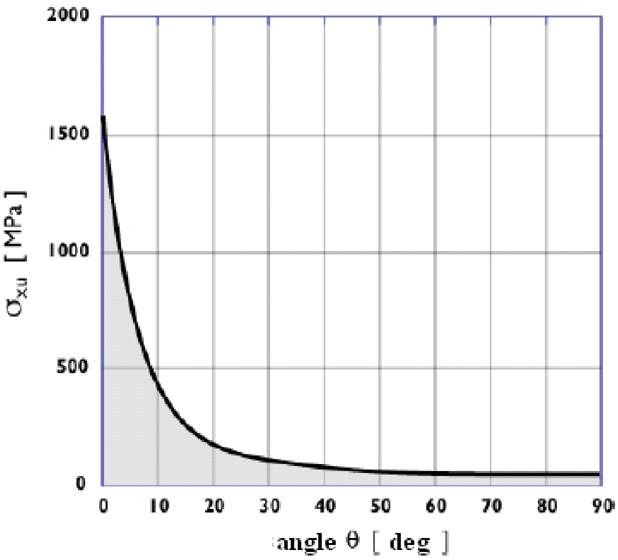


Figure 6-10 – Tsai-Hill criterion: tensile failure stress as a function of θ for unidirectional laminates ($\sigma_{1u} = 1600$ MPa , $\sigma_{2u} = 40$ MPa , $\tau_{12u} = 70$ MPa).

As it is previously shown, the great variability of elastic and strength properties of fiber-reinforced materials depends upon the direction of the fibers compared to the direction of applied load.

6.3 MECHANICAL CHARACTERIZATION TESTS FOR FIBER-REINFORCED MATERIALS

The mechanical characterization of composite materials is generally more complex than for any other materials due to their significant anisotropy. This has led to either modification of existing test methods related to isotropic material as well as the definition of new tests suitable for anisotropic material. The great variety of existing test methods for the characterization of physical and mechanical properties of fiber-reinforced composites makes their detailed description too long to be addressed in this document. For information the reader is referred to specific tests as well as international standards such as the *International Organization for Standardization* (ISO) and the *American Society for Testing and Materials* (ASTM) which have been referenced in the technical data sheet of chapter 2 of this document.

Specific test methods are also available in a number of guidelines, design recommendations and technical documents, including:
- ACI 440.3R 04 *“Guide Test Methods for Fiber-reinforced Polymers for Reinforcing or Strengthening Concrete Structures”*;
- JSCE (1995) *“Test methods for continuous fiber reinforcing materials”*;
- JSCE (2000) *“Test methods for continuous fiber sheets”*;
- ISO (TC71/SC6N) *“Non-conventional strengthening of concrete - Test methods-Part 1: Fiber strengthened polymer (FRP)bars and grids”*;
- ISO (TC71/SC6N) *“Non-conventional strengthening of concrete - Test methods-Part 2: Fiber strengthened polymer (FRP) sheets”*.

The phase of specimen preparation is of fundamental importance because the quality of the samples has a great impact on the mechanical properties. The specimen subjected to testing shall be identical as much as possible to that realized during the in-situ installation in compliance to Section 2.4. The technique adopted for specimen preparation changes depending upon the preparation site (field site or laboratory), the nature of materials (fiber and resin type), and the adopted strengthening system (unidirectional or multi-directional dry fabrics, pre-impregnated fabrics, pultruded laminates). Specimen laboratory preparation allows for a better control of sampling; conversely, on-site preparation is more significant for the specific application.

There are many important issues related to specimen preparation. Some of the most relevant irregularities may be caused by the following factors:

- Misalignments of fibers during lamination.
- Presence of undesired contaminating agents (entrapped during the lamination process).
- High residual fraction of gaps (poor debulking).
- Misalignments during the final cut of the laminate (imperfect parallelism between the cut line and the main direction of the specimen).

Figure 6-11 describes each phase starting from specimen preparation to the performance of the final experimental test.

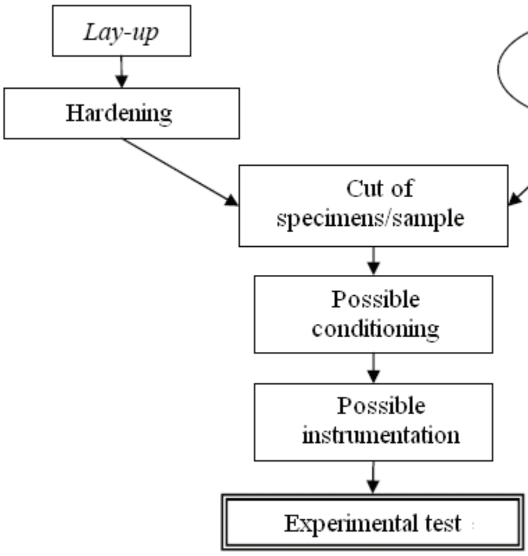


Figure 6-11 – Phases of specimen preparation.

In the following, recommendations are given for specimens' preparation. Such recommendations are of a general nature and do not take into account a number of specific factors related to type of material and type of application. Reference shall be made to specific guidelines such as ISO 1268(E) and ASTM D5687/D5687M-95.

- Surface parallelism and regularity: for specimen preparation, an aluminium-type plate function as support is usually used to lay out the composite material. After the lamination phase is concluded, a second aluminium plate to ensure uniformity of thickness is placed on top the specimen. Both aluminium plate surfaces in contact with the specimen shall be treated with a detaching layer, either in spray or film form (typically PTFE), to allow easy removal of the sample after the hardening phase.
- Plies orientation: layers are to be laid following pre-defined lamination sequence.
- Removal of gaps within the laminate: it is carried out using vacuum or simply through a spatula or a roller. This operation can be carried out both at high or room temperature.
- Sample contamination: during lamination, samples shall be protected from dust and any other possible contaminating agents.

- Laminate hardening: the pressure and temperature to be applied for hardening of samples shall be clearly specified by the system manufacturer. Hardening of samples may also be improved by vacuum.
- Surface protection: the laminate may be coated with a protective film to prevent surface contamination. Using protective films may be useful for multiple layer of reinforcement.
- Cutting of samples: it may be carried out with several cutting tools (diamond blades, water jet, etc.). The choice shall be made according to the material properties and the sample dimensions (thickness). When further surface finishing is needed, abrasive paper or suitable tools can be adopted.
- Conditioning: when required, conditioning shall be carried out prior to testing, according to the available literature guideline.

Depending upon the selected test method, tabs may be required for proper anchoring to avoid localized damage of the specimen. Tabs shall have a larger deformability compared to the specimen and a suitable length to ensure transfer of loading from the testing apparatus to the sample.

Depending upon the mechanical property to be determined, different readings shall be taken while performing the test. The most common measurements are referred to the applied load, displacements, strain (by means of Linear Variable Differential Transducers, LVDTs, or strain gages) as a function of time. When needed, temperature and relative humidity can also be recorded.

7 APPENDIX B (DEBONDING)

7.1 FAILURE DUE TO DEBONDING

The main failure modes of FRP-strengthened structural members due to debonding are summarized as follows:

- Mode 1 (plate end debonding) (Figure 7-1). The end portions of the FRP system are subjected to high interfacial shear stresses for a length of approximately 100-200 mm.

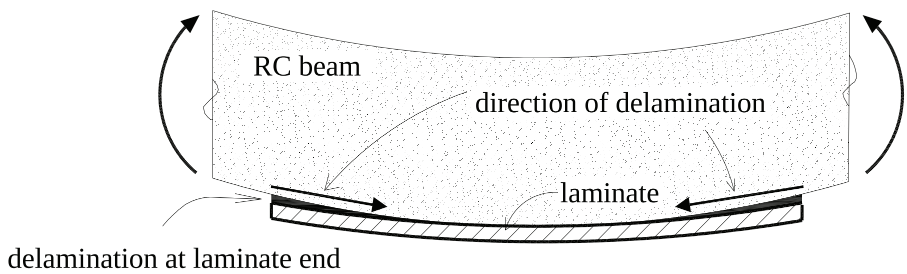
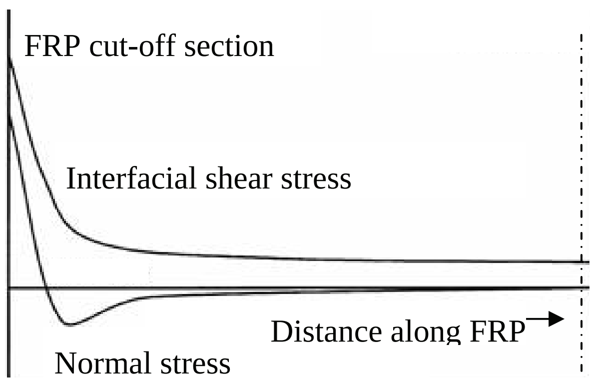
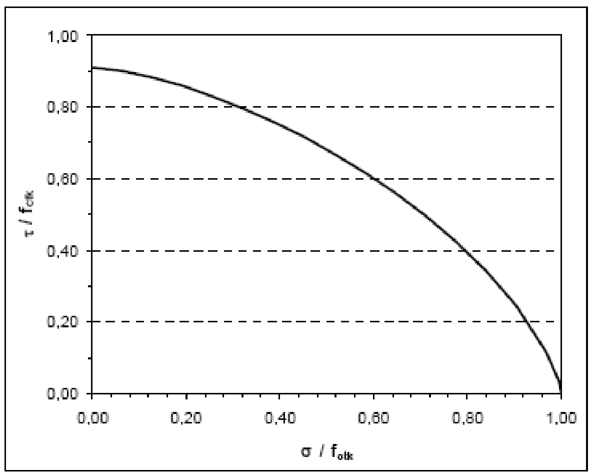


Figure 7-1 – Plate end debonding.

- When strengthening is carried out with FRP laminates, tensile stress perpendicular to the interface between FRP and support (normal stress) may arise due to the significant stiffness of FRP laminate (Figure 7-2(a)). Such a normal stress may reduce the value of interfacial shear stress as shown in Figure 7-2(b). Failure mode by end debonding is characterized by brittle behavior.





(a) (b)

Figure 7-2 – (a) Interfacial shear and normal stress along the length of a bonded FRP laminate (linear-elastic analysis); (b) Strength domain represented by interfacial shear and normal stresses.

- Mode 2 (Debonding by flexural cracks in the beam) (Figure 7-3). Flexural cracking generates discontinuity within the support that enhances interfacial shear stress responsible for FRP debonding. Cracking may be oriented perpendicular to the beam axis when flexural loads are predominant; inclined when there is a combination of flexure and shear.

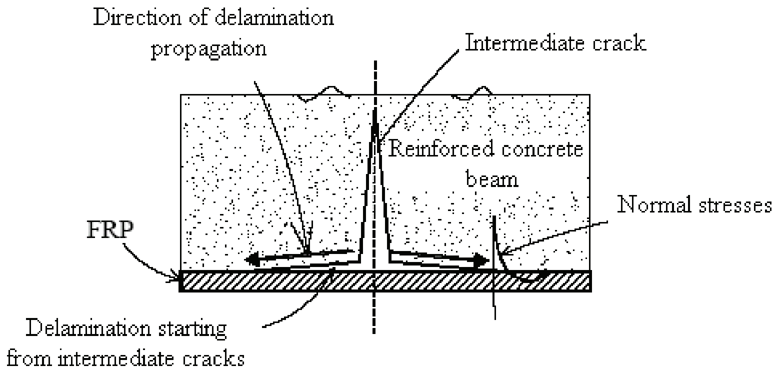


Figure 7-3 - Debonding starting from vertical cracks in concrete.

- Mode 3 (Debonding by diagonal shear cracks) (Figure 7-4). For members where shear stresses are predominant compared to flexural stresses, a relative displacement between the edges of the crack is displayed. Such displacement increases normal stress perpendicular to the FRP laminate responsible for FRP debonding. Such a debonding mechanism is active irrespectively of the presence of stirrups. Collapse due to debonding by diagonal shear cracks is peculiar of four-point-bending laboratory tests; it is not common for field application where the applied load is distributed over the beam's length. For heavily strengthened beams with low transverse reinforcement, debonding usually generates at the end plate section due to peeling.

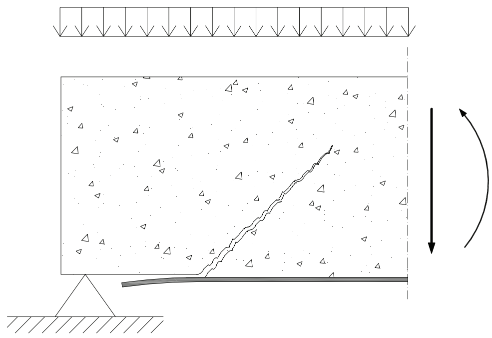


Figure 7-4 – Debonding by diagonal shear crack.

- Mode 4 (Debonding by irregularities and roughness of the concrete surface). Localized debonding due to surface irregularities of the concrete substrate may propagate and cause full debonding of the FRP system. This failure mode can be avoided if the concrete surface is treated in such a way to avoid excessive roughness.

7.2 BOND BETWEEN FRP AND CONCRETE

In the following, additional recommendations related to bond between FRP and concrete support are given. Reference is made to Figure 7-5; symbols are those introduced in Section 4.1.

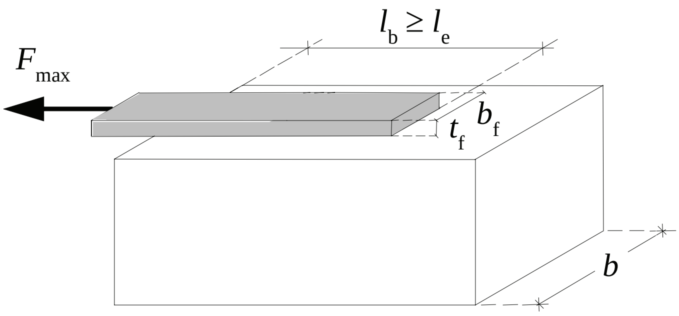


Figure 7-5 – Maximum force allowed to FRP reinforcement.

7.2.1 Specific fracture energy

When the stiffness of the concrete support is much greater compared to the stiffness of the FRP system, the following relationship applies:

$$F_{max} = b_f \cdot \sqrt{2 \cdot E_f \cdot t_f \cdot \Gamma_F} \tag{7.1}$$

between the maximum force, F_{max}, allowed to the FRP reinforcement considered of infinite length and the fracture energy, Γ_f, assuming:

$$F_{max} = b_f \int_0^\infty \tau_b(x)\, dx\,, \quad \Gamma_F = \int_0^\infty \tau_b(s)\, ds \tag{7.2}$$

where t_f, b_f, E_f represent FRP thickness, width, and Young modulus of elasticity in the direction of the applied force, respectively.

The fracture energy depends on the strength properties of both concrete and adhesive, as well as on the characteristics of the concrete surface. For properly installed FRP systems, failure by debonding takes place on the concrete support and the specific fracture energy may be written in a similar fashion to that used for failure mode 1 of the concrete:

$$\Gamma_F = k_G \cdot k_b \cdot \sqrt{f_{ck} \cdot f_{ctm}} \quad \text{[force in N, length in mm]} \tag{7.3}$$

where the coefficient k_G shall be experimentally adjusted. The value of such coefficient has been computed over a large population of experimental results available in the literature. The statistical analysis of the results has provided an average value equal to 0.064, a standard deviation equal to 0.023, and a 5th percentile of the statistical distribution equal to 0.026. When the latter value is used in Eq. (7.3), the characteristic value, Γ_{Fk}, of the specific fracture energy is obtained. On the basis of this consideration, this Guideline suggest adopting the value of 0.03 for k_G.

7.2.2 Bond-slip law

Bond between FRP and concrete is typically expressed with a relationship between interfacial shear stress and the corresponding slip ("$\tau_b - s$" relationship). Both FRP and concrete mechanical characteristics as well as geometry of the FRP system and concrete support shall be considered in the analysis.

The $\tau_b - s$ relationship is typically non linear with a descending branch; for design purposes, it may be treated as a bi-linear relationship as shown in Figure 7-6. The first ascending branch is defined by taking into account the deformability of adhesive layer and concrete support for an appropriate depth. Unless a more detailed analysis is performed, the average mechanical parameters defining the $\tau_b - s$ relationship, can be evaluated as follows:

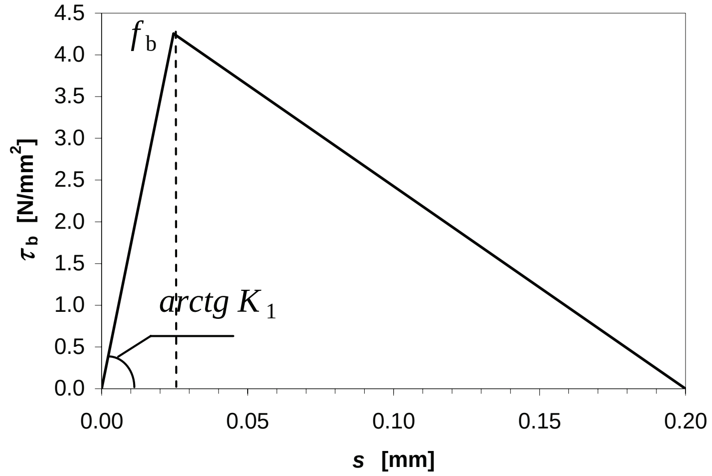


Figure 7-6 – Bi-linear "$\tau_b - s$" relationship ($f_{ck} = 20$ MPa, $k_b = 1$).

- <u>Maximum bond strength,</u> f_b:

The maximum experimental average bond strength can be expressed as follows:

$$f_b = 0.064 \cdot k_b \cdot \sqrt{f_{ck} \cdot f_{ctm}} \quad \text{[force in N, length in mm]} \tag{7.4}$$

where f_{ck} and f_{ctm} represent the concrete characteristic compressive strength and the average concrete tensile strength, respectively; and the geometrical factor k_b can be written as follows:

$$k_b = \sqrt{\frac{2 - \frac{b_f}{b}}{1 + \frac{b_f}{400}}} \geq 1 \qquad \text{[length in mm]} \tag{7.5}$$

where b and b_f represent beam and FRP width, respectively. Equation (7.5) is valid when $b_f / b \geq 0.33$ (when $b_f / b < 0.33$ the value corresponding to $b_f / b = 0.33$ shall be adopted for k_b).

- <u>Slope, K_1, of the ascending branch</u>:

$$K_1 = \frac{c_1}{t_a / G_a + t_c / G_c} \tag{7.6}$$

where G_a, G_c represent shear modules of adhesive and concrete, respectively; t_a is the nominal thickness of the adhesive; and t_c is the effective depth of concrete (suggested values for t_c and c_1 are 20-30 *mm* and 0.5-0.7, respectively).

- Interface slip corresponding to full debonding, s_{f} :

$$s_{\mathrm{f}} = 0.2 \text{ mm} \tag{7.7}$$

This value of s_{f} ensures the specific fracture energy, Γ_{f}, (represented by the area subtended the "$\tau_{\mathrm{b}} - s$" relationship) is equal to the value reported in Equation (7.3). At SLS, the "$\tau_{\mathrm{b}} - s$" relationship reduces to the ascending branch only and K_1 is given by Equation (7.6) for $c_1 = 1$.

7.3 SIMPLIFIED METHOD FOR DEBONDING DUE TO FLEXURAL CRACKS (MODE 2) AT ULTIMATE LIMIT STATE

As an alternative to the general method reported at item 1(P) of Section 4.1.4, this simplified method is based on the definition of the maximum design strain, $\varepsilon_{\mathrm{fdd}}$, for FRP reinforcement to be evaluated with Equation (4.7), hereafter reported for the sake of completeness:

$$\varepsilon_{\mathrm{fdd}} = k_{\mathrm{cr}} \cdot \frac{f_{\mathrm{fdd}}}{E_{\mathrm{f}}} = k_{\mathrm{cr}} \cdot \frac{1}{\gamma_{\mathrm{f,d}} \cdot \sqrt{\gamma_{\mathrm{c}}}} \cdot \sqrt{\frac{2 \cdot \Gamma_{\mathrm{Fk}}}{E_{\mathrm{f}} \cdot t_{\mathrm{f}}}} \tag{7.8}$$

This relationship is similar to that proposed for maximum stress or strain in FRP reinforcement when mode 1 FRP debonding controls. However, for a constant applied force, the maximum interfacial shear stresses are significantly smaller compared to that achieved close to the end of the FRP itself due to the reduced distance between cracks. This implies that the value of maximum FRP strain related to failure mode 2 is greater than that pertaining to failure mode 1. These considerations have suggested adopting a magnifying coefficient $k_{\mathrm{cr}} > 1$ for Equation (4.7).

The calibration of k_{cr} has been carried out on reinforced concrete beams strengthened with FRP laminates or fabrics failed by FRP intermediate debonding (mode 2). The statistical analysis of the results has provided an average value equal to 4.289, a standard deviation of 0.743, and a 5$^{\text{th}}$ percentile value equal to 3.070. Given the limited scattering exhibited by the application of Equation (4.6) and considering the lower brittleness of FRP intermediate debonding compared to FRP end debonding, it is suggested to use Equation (4.6) for the evaluation of the design strain, assuming k_{cr} equal to 3.0 that corresponds to the 5$^{\text{th}}$ percentile of the statistical distribution.

8 APPENDIX C (STRENGTHENING FOR COMBINED BENDING AND AXIAL LOAD OF REINFORCED CONCRETE MEMBERS)

8.1 FLEXURAL CAPACITY OF FRP STRENGTHENEND MEMBERS SUBJECTED TO COMBINED BENDING AND AXIAL LOAD

FRP strengthened members subjected to combined bending and axial load shall be designed as follows:

$$M_{\mathrm{Sd}} \leq M_{\mathrm{Rd}}(N_{\mathrm{Sd}}) \tag{8.1}$$

where M_{sd} is the design applied moment and M_{rd} represents the flexural capacity of the strengthened member considering the design axial force N_{sd}.

A possible design procedure is hereafter described. The mechanical ratio μ_{s} and μ_{f} related to tension steel reinforcement and FRP system, respectively, can be calculated as follows:

$$\mu_{\mathrm{s}} = \frac{A_{\mathrm{s1}} \cdot f_{\mathrm{yd}}}{\overline{f_{\mathrm{ccd}}} \cdot b \cdot d} \tag{8.2}$$

$$\mu_{\mathrm{f}} = \frac{b_{\mathrm{f}} \cdot t_{\mathrm{f}} \cdot f_{\mathrm{fd}}}{\overline{f_{\mathrm{ccd}}} \cdot b \cdot d} \tag{8.3}$$

where A_{s1} and f_{yd} represent area and design yield strength of existing steel reinforcement, respectively; $\overline{f_{\mathrm{ccd}}}$ is equal to the design strength of confined concrete, f_{ccd}, suitably reduced if it is necessary; b and d are width and effective depth of the FRP strengthened member, respectively; b_{f} and t_{f} are FRP width and thickness, respectively; and f_{fd} is FRP ultimate design strength calculated according to Section 4.2.2.4, item (2)P. Material design strengths for non-seismic applications shall be in compliance with Section 3.3.3, item (7); for seismic applications, such values shall be obtained from in-situ experimental tests. For seismic applications, unless a more detailed analysis regarding structural details and material properties is available, mechanical properties of existing materials shall be divided by an appropriate coefficient greater than 1.

The following non-dimensional equations reflecting applied loads are introduced:

$$n_{\mathrm{Sd}} = \frac{N_{\mathrm{Sd}}}{\overline{f_{\mathrm{ccd}}} \cdot b \cdot d} \tag{8.4}$$

$$m_{\mathrm{Sd}} = \frac{M_{\mathrm{Sd}}}{\overline{f_{\mathrm{ccd}}} \cdot b \cdot d^2} \tag{8.5}$$

When FRP width and mechanical properties are known, a trial and error procedure can be initiated to evaluate the thickness of FRP reinforcement as follows.

Step 1
η is computed as follows:

$$\eta = n_{\mathrm{Sd}} + \mu_{\mathrm{s}} \cdot (1-u) + \mu_{\mathrm{f}} \tag{8.6}$$

Step 2
The following parameters η_{i} (i = 0, 1, 2, 3) are defined:

$$\eta_0 = -\mu_{\mathrm{s}} \cdot u\,, \quad \eta_1 = \frac{2}{3} \cdot \frac{r}{r+1}\,, \quad \eta_2 = 0.8 \cdot \frac{1.75 \cdot r}{1.75 \cdot r + 1}\,, \quad \eta_3 = 0.51 + \mu_{\mathrm{f}} \cdot (1-r) \tag{8.7}$$

where:
- u is the ratio of steel existing compression, A_{s2}, and tension, A_{s1} area.
- $r = 2/1000\,\varepsilon_{\mathrm{fd}}$.

Step 3
From Table 8-1, failure mode (Figure 4-5, 4.2.2.3) and the corresponding value of the parameter $m_{(\mathrm{mr})}(\eta)$ can be determined as a function of η when compared with the limits presented in Step 2.

Table 8-1

Failure mode	η	$m_{(\mathrm{mr})}(\eta)$
1a	$\eta_0 \le \eta \le \eta_1$	$m_{(1\mathrm{a})}(\eta) = \frac{1}{2} \cdot \left\{ \eta_0 + \frac{\eta_1 \cdot (1-\eta_1) - \eta_0}{\eta_1 - \eta_0} \cdot (\eta - \eta_0) \right\}$
1b	$\eta_1 \le \eta \le \eta_2$	$m_{(1\mathrm{b})}(\eta) = \frac{1}{2} \cdot \left\{ \eta_1 \cdot \eta_2 + \left[1 - (\eta_1 + \eta_2)\right] \cdot \eta \right\}$
2	$\eta_2 \le \eta \le \eta_3$	$m_{(2)}(\eta) = \frac{1}{2} \cdot \left\{ \eta_2 \cdot (1-\eta_2) + \frac{(0.75 - \eta_3) - \eta_2 \cdot (1-\eta_2)}{\eta_3 - \eta_2} \cdot (\eta - \eta_2) \right\}$

Step 4
The non-dimensional flexural capacity, $m_{\mathrm{Rd}}(n_{\mathrm{Sd}})$, of the strengthened member is evaluated as follows:

$$m_{\mathrm{Rd}}(n_{\mathrm{Sd}}) = m_{(\mathrm{mr})}(\eta) + \frac{1}{2} \cdot \left[\mu_{\mathrm{s}} \cdot (1+u) + \mu_{\mathrm{f}} \right] \tag{8.8}$$

Step 5
The following relationship shall be met:

$$m_{\mathrm{Rd}}(n_{\mathrm{Sd}}) \ge m_{\mathrm{Sd}} \tag{8.9}$$

If this is not the case, the thickness, t_{f}, of the strengthening system is increased as well as its mechanical ratio, μ_f, and the iterative procedure is repeated from Step 1.

9 APPENDIX D (CONFINED CONCRETE)

9.1 CONSTITUTIVE LAW OF CONFINED CONCRETE

Modeling the mechanical behavior of FRP-confined concrete members calls for the preliminary definition of a suitable constitutive law $\sigma(\varepsilon)$ related to the mechanical behavior of members subjected to uni-axial compression (σ and ε are considered positive in compression).

In this context, as an alternative method to the parabolic-rectangular model proposed in 4.5.3, a non-linear relationship can be adopted similar to that shown in Figure 9-1, where a parabolic branch is followed by a linear ascending branch. At the intersection point between the two branches, the first derivative of the function $\sigma(\varepsilon)$ shall be assumed continuous.

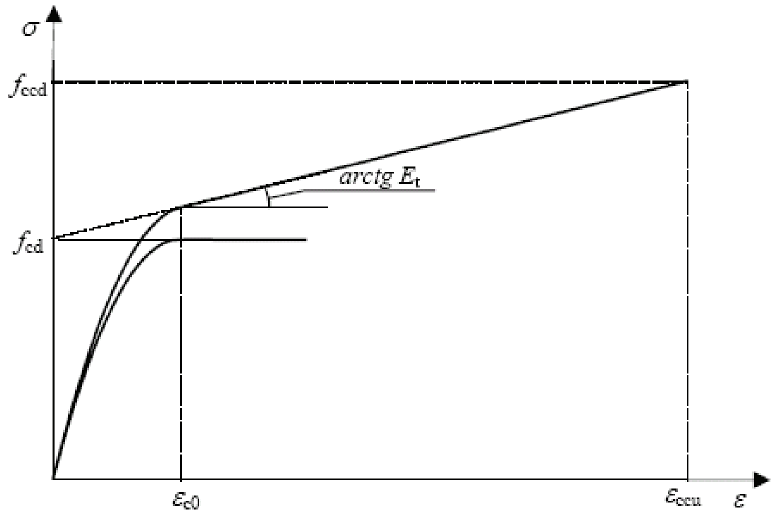


Figure 9-1 – Stress-strain model of FRP-confined concrete.

The mathematical expression of such relationship can be written as follows:

- (parabolic branch) $$\frac{f_{\mathrm{c}}}{f_{\mathrm{cd}}} = a \cdot \bar{\varepsilon} - \bar{\varepsilon}^2 \quad \text{for } 0 \le \bar{\varepsilon} \le 1 \tag{9.1}$$

- (linear branch) $$\frac{f_{\mathrm{c}}}{f_{\mathrm{cd}}} = 1 + b \cdot \bar{\varepsilon} \quad \text{for } 1 \le \bar{\varepsilon} \le \frac{\varepsilon_{\mathrm{ccu}}}{\varepsilon_{\mathrm{c0}}} \tag{9.2}$$

where:
- $\bar{\varepsilon}$ is a non-dimensional coefficient defined as follows:

$$\bar{\varepsilon} = \frac{\varepsilon_{\mathrm{c}}}{\varepsilon_{\mathrm{c0}}} \tag{9.3}$$

- f_{cd} and $\varepsilon_{\mathrm{c0}}$ are the design strength of unconfined concrete and the concrete strain at peak (typically assumed equal to 0.2%), respectively.
- $\varepsilon_{\mathrm{ccu}}$ is the design ultimate strain of confined concrete corresponding to the design strength f_{ccd} (chapter 4).

- the coefficients a and b are taken as follows:

$$a = 1 + \gamma\,, \quad b = \gamma - 1 \tag{9.4}$$

where (see Figure 9-1):

$$\gamma = \frac{f_{\mathrm{cd}} + E_{\mathrm{t}} \cdot \varepsilon_{\mathrm{c0}}}{f_{\mathrm{cd}}} \tag{9.5}$$

$$E_{\mathrm{t}} = \frac{f_{\mathrm{ccd}} - f_{\mathrm{cd}}}{\varepsilon_{\mathrm{ccu}}} \tag{9.6}$$

10 APPENDIX E (EXAMPLES OF FRP STRENGTHENING DESIGN)

In this Appendix, numerical examples of non-seismic FRP strengthening of RC members are provided. It is assumed that FRP strengthening is necessary due to increase of applied loads. Design is only performed at ultimate limit state. Serviceability limit state design is not carried out because similar to traditional RC members theory.

10.1 GEOMETRICAL, MECHANICAL, AND LOADING DATA

The building considered for design is shown in Figure 10-1. Structural elements are defined as follows:

- Main rectangular beams with cross-section of 30 cm x 50 cm (concrete cover d_1=d_2=3 cm).
- Secondary rectangular beams with cross-section of 30 cm x 40 cm (concrete cover d_1=d_2=3 cm).
- Rectangular columns with cross-section of 20 cm x 30 cm (concrete cover d_1=d_2=3 cm).

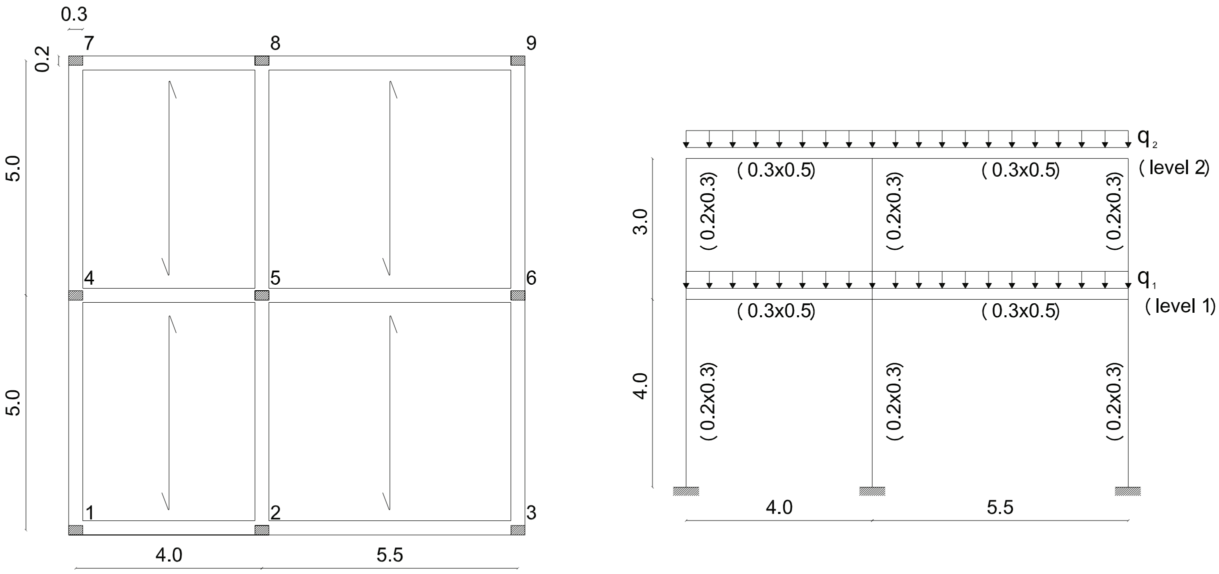


Figure 10-1 – Building geometry (dimensions in m).

Material mechanical properties are as follows:

- Concrete: R_{ck} = 20 N/mm^2.
- Steel: FeB32k (f_{yk}=31.5 N/mm^2).

Loading conditions are defined as follows:

- Live load at level 1: $a_1 = 2.00\,\text{kN}/\text{m}^2$.
- Live load at level 2: $a_2 = 0.50\,\text{kN}/\text{m}^2$.
- Snow (zone III, height $a_s < 200\,\text{m}$): $b = 0.75\,\text{kN}/\text{m}^2$.
- Dead load due to flooring (for each level): $g = 6.00\,\text{kN}/\text{m}^2$.

Factored loads acting at ULS can be evaluated as follows:

- Level 1: q_1 = 62.25 kN/m.
- Level 2: q_2 = 55.00 kN/m.

Figure 10-2 shows existing steel bar arrangement for beams and columns.

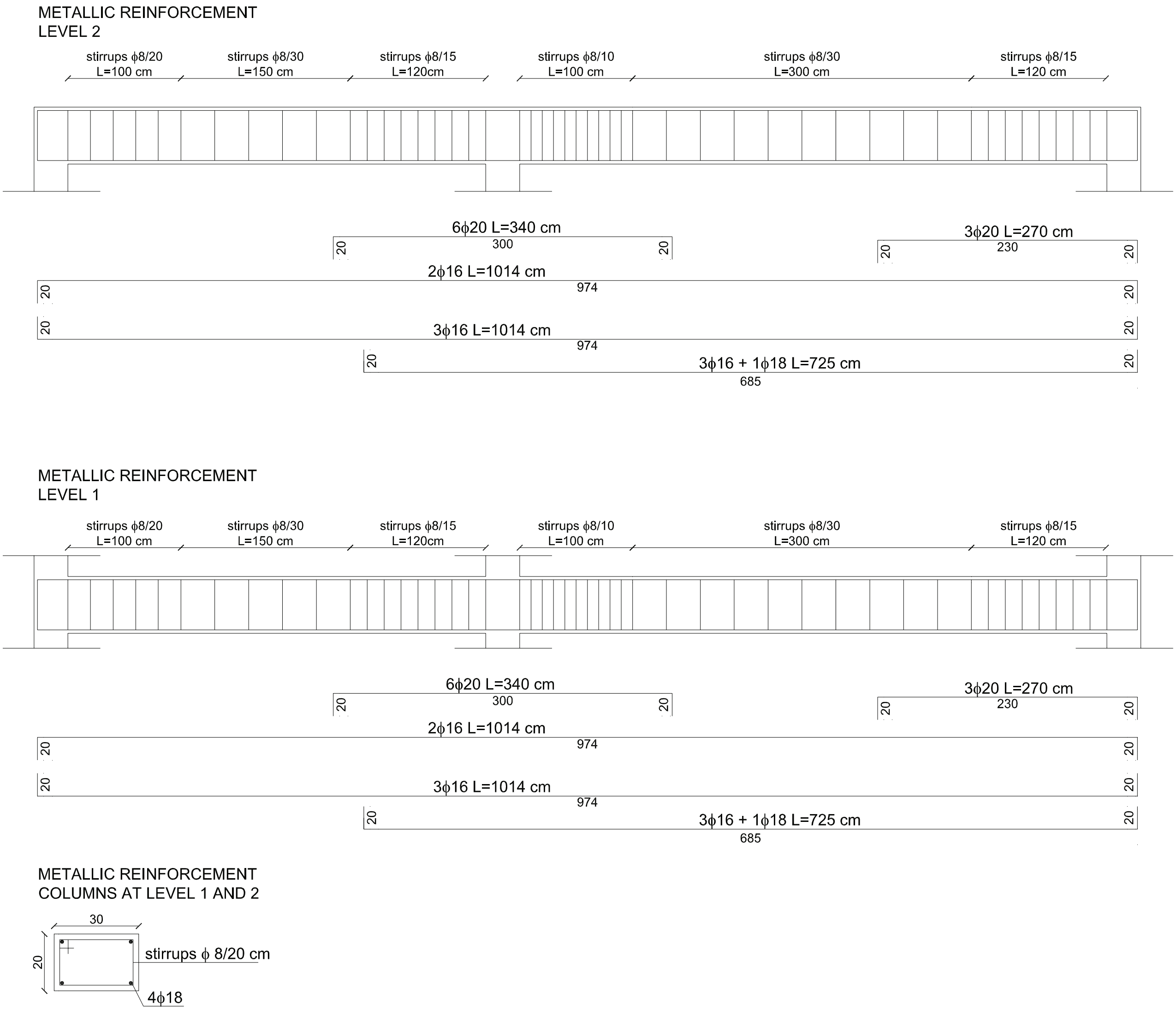


Figure 10-2 – Steel bars location for beams and columns.

10.2 INCREASE OF APPLIED LOADS

New loads are defined as follows:

- Level 1: a_1 = 6.00 kN/m^2.
- Level 2: a_2 = 4.00 kN/m^2.

New factored loads acting at ULS can be evaluated as follows:

- Level 1: q_1 = 92.25 kN/m.
- Level 2: q_2 = 81.20 kN/m.

10.3 DESIGN OF FLEXURAL REINFORCEMENT

Design material properties are determined as follows:

- Concrete ($f_{\mathrm{ck}} = 16.60$ N/mm^2, γ_c=1.60, f_{cd}=10.38 N/mm^2, $f_{\mathrm{ctm}} = 1.95$ N/mm^2, $f_{\mathrm{ctd}} = 0.7 \cdot f_{\mathrm{ctm}} / \gamma_{\mathrm{c}} = 0.85$ N/mm^2)

- Steel (f_{yk} = 315.00 N/mm^2, γ_s = 1.15, f_{yd} = 274.00 N/mm^2)

The following relationship shall be met:

$$M_{Sd} \le M_{Rd} \tag{10.1}$$

As indicated in Table 10-1, Equation (10.7) is not met at mid-span of both levels for 5.5 m long beams.

Table 10-1

Level	Span [m]	Section	M_{Sd} [kN m]	A_{s1} [cm^2]	A_{s2} [cm^2]	M_{Rd} [kN m]	Eq. (10.7) satisfied
1	4.0	left support	-49	4.02	6.03	-49.3	Yes
	4.0	mid-span	69	6.03	4.02	73.2	Yes
	4.0	right support	-195	22.90	14.60	-272.4	Yes
	5.5	left support	-242	22.90	14.00	-272.4	Yes
	5.5	mid-span	182	14.60	4.02	168.4	No
	5.5	right support	-99	13.40	14.60	-162.1	Yes
2	4.0	left support	-35	4.02	6.03	-49.3	Yes
	4.0	mid-span	65	6.03	4.02	73.2	Yes
	4.0	right support	-175	22.90	14.60	-272.4	Yes
	5.5	left support	-207	22.90	14.00	-272.4	Yes
	5.5	mid-span	173	14.60	4.02	168.4	No
	5.5	right support	-67	13.40	14.60	-162.1	Yes

FRP flexural strengthening is performed by installing carbon fiber reinforcement using the wet-lay-up method with the following geometrical and mechanical characteristics (mode 1, Section 2.3.3.2:α_{fE}=α_{ff}=0.9):

- CFRP thickness: $t_{f,1}$= 0.167 mm.
- CFRP width: b_f = 240.0 mm.
- CFRP Young modulus of elasticity in fibers direction (beam axis):
  $E_f = \alpha_{fE} \cdot E_{fib}$ = 0.9·300000 N/mm^2 = 270000 N/mm^2.
- CFRP characteristic strength: $f_{fk} = \alpha_{ff} \cdot f_{fib}$ = 0.9·3000 N/mm^2 = 2700 N/mm^2.

For Type A application, the partial factors γ_f and $\gamma_{f,d}$ are 1.10 and 1.20, respectively (Table 3-2, Section 3.4.1). The environmental conversion factor, η_a, is set equal to 0.95 (Table 3-4, Section 3.5.1).

A trial and error procedure is initiated for the determination of the number of CFRP plies, n_f, needed to satisfy Equation (10.7). Therefore, assuming n_f=1, the maximum CFRP design strain, ε_{fd}, can be calculated as follows (Equation (4.19):

$$\varepsilon_{fd} = \min\left\{\eta_a \cdot \frac{\varepsilon_{fk}}{\gamma_f}, \varepsilon_{fdd}\right\} = \varepsilon_{fdd} = 0.0054 \tag{10.2}$$

where:

$$\varepsilon_{fk} = \frac{f_{fk}}{E_f} = 0.01 \tag{10.3}$$

$$\varepsilon_{\text{fdd}} = \frac{f_{\text{fdd,2}}}{E_{\text{f}}} = 0.0054 \tag{10.4}$$

CFRP design strength, $f_{\text{fdd,2}}$, when failure mode 2 (debonding) controls and when k_{cr} from Equation (4.6) is set equal to 3.0, can be calculated as follows (Equations (4.3), (4.2), and (4.6)):

$$k_{\text{b}} = \sqrt{\frac{2 - \frac{b_{\text{f}}}{b}}{1 + \frac{b_{\text{f}}}{400}}} \geq 1 \tag{10.5}$$

$$\Gamma_{\text{Fk}} = 0.03 \cdot k_{\text{b}} \cdot \sqrt{f_{\text{ck}} \cdot f_{\text{ctm}}} = 0.17 \text{ N/mm}^2 \tag{10.6}$$

$$f_{\text{fdd,2}} = \frac{k_{\text{cr}}}{\gamma_{\text{f,d}} \cdot \sqrt{\gamma_{\text{c}}}} \cdot \sqrt{\frac{2 \cdot E_{\text{f}} \cdot \Gamma_{\text{Fk}}}{n_{\text{f}} \cdot t_{\text{f,1}}}} = 1469 \text{ N/mm}^2 \tag{10.7}$$

CFRP flexural failure mechanism may be of two types, depending whether CFRP maximum tensile strain, ε_{fd}, or concrete maximum compressive strain, ε_{cu}, is reached (Figure 10-3).

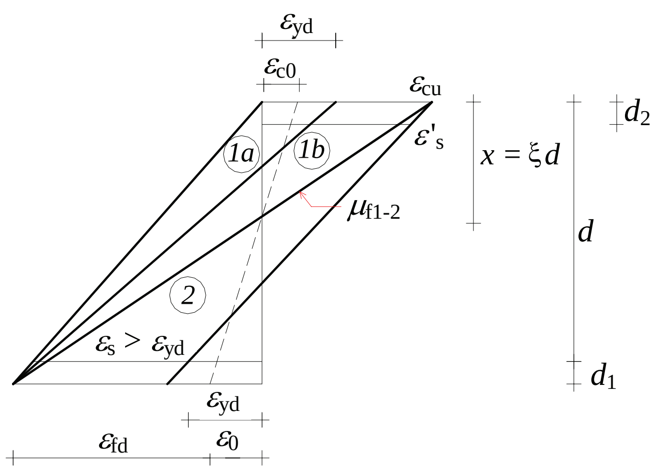


Figure 10-3 – Failure regions of RC members strengthened with FRP.

To identify the failure mode for this particular case, the CFRP mechanical ratio, μ_{f}, is computed:

$$\mu_{\text{f}} = \frac{b_{\text{f}} \cdot (n_{\text{f}} \cdot t_{\text{f,1}}) \cdot f_{\text{fdd,2}}}{\overline{f_{\text{cd}}} \cdot b \cdot d} \tag{10.8}$$

and compared with the balanced mechanical ratio, $\mu_{\text{f1-2}}$, defined as follows:

$$\mu_{\text{f1-2}} = \frac{0.8 \cdot \varepsilon_{\text{cu}} \cdot \frac{h}{d}}{\varepsilon_{\text{cu}} + \varepsilon_{\text{fd}} + \varepsilon_0} - \mu_{\text{s}} \cdot (1 - u) \tag{10.9}$$

In Equation (10.8) $\overline{f_{cd}}$ is equal to the design concrete compressive strength, f_{cd}, suitably reduced if it is necessary; in Equation (10.9), μ_s is defined in Equation (8.2), u represents the ratio between steel compression, A_{s2}, and tension, A_{s1}, area; ε_0 is the initial strain of the tension side of concrete, evaluated as:

$$\varepsilon_0 = \frac{M_{gk}}{0.9 \cdot d \cdot E_s \cdot A_{s2}} \tag{10.10}$$

where M_{gk} is the moment due to dead loads at SLS.

If $\mu_f \le \mu_{f1-2}$, failure takes place in region 1; if $\mu_f > \mu_{f1-2}$, failure takes place in region 2 (Figure 10-3). Once the failure mode is known, the position, x, of the neutral axis can be identified from Equation (4.20). The flexural capacity, M_{Rd}, can be calculated from Equation (4.21), assuming that the partial factor, γ_{Rd}, is set equal to 1.00 (Table 3-3, Section 3.4.2). The calculated flexural capacity, M_{Rd}, for a single layer CFRP reinforcement (Table 10-2) is greater than the applied moment, M_{Sd}.

Table 10-2

Level	Span [m]	Section	M_{Sd} [kN m]	n_f	ε_{fd}	$\mu_{f.1}$	$\mu_{f.1\text{-}2}$	Region	x [m]	M_{Rd} [kN m]	l_e [m]
1	5.5	mid-span	182	1	0.0054	0.047	0.099	1	0.14	195	0.11
2	5.5	mid-span	173	1	0.0054	0.047	0.099	1	0.14	195	0.11

If Equation (10.1) is not satisfied, the number, n_f, of CFRP plies shall be progressively increased, iterating the design procedure. CFRP strengthening shall be installed along the beam axis until Equation (10.1) is not met. Proper anchorage length shall be provided to CFRP reinforcement according to Section 4.2.2.5. Table 10-2 also summarizes the value of the optimal bond length, l_e, calculated according to Equation (4.1):

$$l_e = \sqrt{\frac{E_f \cdot t_f}{2 \cdot f_{ctm}}} = \sqrt{\frac{E_f \cdot n_f \cdot t_{f,1}}{2 \cdot f_{ctm}}} \tag{10.11}$$

10.4 DESIGN OF SHEAR REINFORCEMENT

The following relationship shall be met:

$$V_{Sd} \le V_{Rd} \tag{10.12}$$

where V_{Sd} is the design applied shear force, and V_{Rd} represent the shear capacity to be calculated as follows:

$$V_{Rd} = \min\left\{V_{Rd,ct} + V_{Rd,s},\ V_{Rd,max}\right\} \tag{10.13}$$

where $V_{Rd,ct}$ and $V_{Rd,s}$ represent concrete and steel existing contribution to the shear capacity, respectively; and $V_{\mathrm{Rd,max}}$ is the maximum shear capacity representing the resistance of the concrete compressed strut.

According to the current building code, the above mentioned quantities may be expressed as follows:

$$V_{\mathrm{Rd,max}} = 0.3 \cdot f_{\mathrm{cd}} \cdot b \cdot d \tag{10.14}$$

$$V_{\mathrm{Rd,ct}} = 0.6 \cdot f_{\mathrm{ctd}} \cdot b \cdot d \cdot \delta \tag{10.15}$$

$$V_{\mathrm{Rd,s}} = \frac{A_{\mathrm{sw}}}{s} \cdot f_{\mathrm{ywd}} \cdot 0.9 \cdot d \tag{10.16}$$

where:
- $\delta = 1$ when axial force is negligible, $\delta = 0$ for tension axial force, $\delta = 1 + M_0/M_{\mathrm{Sd}}$ for compression axial force (M_0 is the decompression moment referred to the extreme fiber of the cross section evaluated along the beam axis where M_{Sd} acts);
- A_{sw}, s, and f_{ywd} represent area, spacing, and steel stirrups yield strength, respectively.

According to the current building code, steel contribution to the shear capacity shall be as follows:

$$V_{\mathrm{Rd,s}} \geq \frac{V_{\mathrm{Sd}}}{2} \tag{10.17}$$

Table 10-3 summarizes the as-built shear capacity. As it can be seen, all members require shear strengthening.

Table 10-3

Level	Span [m]	Section	V_{Sd} [kN]	$V_{\mathrm{Rd,max}}$ [kN]	A_{sw} [cm^2]	s [cm]	$V_{\mathrm{Rd,s}}$ [kN]	$V_{\mathrm{Rd,ct}}$ [kN]	V_{Rd} [kN]	Eq. (10.18) satisfied
1	4.0	left support	148	438	1.00	20	58.2	73.6	131.8	NO
	4.0	right support	221	438	1.00	15	77.6	73.6	151.2	NO
	5.5	left support	280	438	1.00	10	116.5	73.6	190.1	NO
	5.5	right support	228	438	1.00	15	77.6	73.6	151.3	NO
2	4.0	left support	127	438	1.00	20	58.2	73.6	131.8	NO
	4.0	right support	198	438	1.00	15	77.6	73.6	151.2	NO
	5.5	left support	248	438	1.00	10	116.5	73.6	190.1	NO
	5.5	right support	197	438	1.00	15	77.6	73.6	151.3	NO

FRP shear strengthening is performed by installing U-wrap carbon fiber reinforcement having the following geometrical and mechanical characteristics (mode 1, Section 2.3.3.2: $\alpha_{\mathrm{fE}}=\alpha_{\mathrm{ff}}=0.9$):

- CFRP thickness: $t_{\mathrm{f,1}} = 0.167$ mm.
- CFRP width: $w_{\mathrm{f}} = 150.0$ mm.
- CFRP Young modulus of elasticity: $E_{\mathrm{f}} = \alpha_{\mathrm{fE}} \cdot E_{\mathrm{fib}} = 0.9 \cdot 300000$ N/mm^2 = 270000 N/mm^2.
- CFRP characteristic strength: $f_{\mathrm{fk}} = \alpha_{\mathrm{ff}} \cdot f_{\mathrm{fib}} = 0.9 \cdot 3000$ N/mm^2 = 2700 N/mm^2.

For Type-A application, the partial factors γ_{f} and $\gamma_{\mathrm{f,d}}$ are 1.10 and 1.20, respectively (Table 3-2, Section 3.4.1).

The following fiber orientations with respect the axis of the beam are considered:

- Level 1: $\beta = 45°$.

- Level 2: $\beta = 90°$.

The design shear capacity of the strengthened member may be evaluated from Equation (4.24):

$$V_{\mathrm{Rd}} = \min\left\{ V_{\mathrm{Rd,ct}} + V_{\mathrm{Rd,s}} + V_{\mathrm{Rd,f}}, V_{\mathrm{Rd,max}} \right\} \tag{10.18}$$

where:
- $V_{\mathrm{Rd,ct}}$ is the concrete contribution to the shear capacity taken from Equation (10.15).
- $V_{\mathrm{Rd,s}}$ is the steel contribution to the shear capacity taken from Equation (10.16).
- $V_{\mathrm{Rd,f}}$ is the CFRP contribution to the shear capacity taken from Equation (4.26) as follows:

$$V_{\mathrm{Rd,f}} = \frac{1}{\gamma_{\mathrm{Rd}}} \cdot 0.9 \cdot d \cdot f_{\mathrm{fed}} \cdot 2 \cdot t_{\mathrm{f}} \cdot \left(\cot\theta + \cot\beta\right) \cdot \frac{w_{\mathrm{f}}}{p_{\mathrm{f}}} \tag{10.19}$$

with $\gamma_{\mathrm{Rd\ set}}$ equal to 1.2 (Table 3-3, Section 3.4.2), and θ=45°.
- $V_{\mathrm{Rd,max}}$ is the maximum shear capacity representing the resistance of the concrete compressed strut taken from Equation (10.14).

For CFRP U-wrap reinforcement, the effective design strength, f_{fed}, can be evaluated from Equation (4.30) as follows:

$$f_{\mathrm{fed}} = f_{\mathrm{fdd}} \cdot \left[1 - \frac{1}{3} \cdot \frac{l_{\mathrm{e}} \cdot \sin\beta}{\min\left\{0.9 \cdot d, h_{\mathrm{w}}\right\}} \right] \tag{10.20}$$

where:
- h_{w} is the beam depth.
- l_{e} is the effective bond length from Equation (10.11).
- f_{fdd} is the bond strength for mode 1 from Equation (4.4):

$$f_{\mathrm{fdd}} = \frac{1}{\gamma_{\mathrm{f,d}} \cdot \sqrt{\gamma_{\mathrm{c}}}} \cdot \sqrt{\frac{2 \cdot E_{\mathrm{f}} \cdot \Gamma_{\mathrm{Fk}}}{n_{\mathrm{f}} \cdot t_{\mathrm{f,1}}}} \tag{10.21}$$

Assuming the CFRP strip width, w_{f}, equal to 150 mm, both center-to-center spacing, p_{f}, and number of CFRP plies, n_{f}, can be determined with a trial and error procedure until Equation (10.12) is satisfied.
Table 10-4 and Table 10-5 as well as Table 10-6 and Table 10-7 summarize the performed shear design of CFRP U-wrapped strengthened member for both level 1 and 2, respectively.

Table 10-4

Span [m]	Section	n_f	p_f [mm]	Γ_{Fk} [N/mm^2]	f_{fdd} [N/mm^2]	l_e [mm]	f_{fed} [N/mm^2]	$V_{Rd,f}$ [kN]
4.0	left support	1	200	0.17	492	106	463	82
4.0	right support	2	200	0.17	348	150	319	112
5.5	left support	2	150	0.17	348	150	319	150
5.5	right support	1	150	0.17	492	106	463	109

Table 10-5

Span [m]	Section	$V_{Rd,ct}$ [kN]	$V_{Rd,s}$ [kN]	$V_{Rd,max}$ [kN]	V_{Rd} [kN]	V_{Sd} [kN]
4.0	left support	73.6	58.0	438.8	214	148
4.0	right support	73.6	78.0	438.8	264	220
5.5	left support	73.6	116.0	438.8	340	280
5.5	right support	73.6	78.0	438.8	261	227

Table 10-6

Span [m]	Section	n_f	p_f [mm]	Γ_{Fk} [N/mm^2]	f_{fdd} [N/mm^2]	l_e [mm]	f_{fed} [N/mm^2]	$V_{Rd,f}$ [kN]
4.0	left support	1	150	0.17	492	106	450	53
4.0	right support	2	150	0.17	348	150	307	72
5.5	left support	3	150	0.17	284	184	243	86
5.5	right support	3	150	0.17	284	184	243	86

Table 10-7

Span [m]	Section	$V_{Rd,ct}$ [kN]	$V_{Rd,s}$ [kN]	$V_{Rd,max}$ [kN]	V_{Rd} [kN]	V_{Sd} [kN]
4.0	left support	73.6	58	438.86	185	127
4.0	right support	73.6	78	438.86	224	198
5.5	left support	73.6	116	438.86	276	248
5.5	right support	73.6	78	438.86	238	197

10.5 CONFINEMENT OF COLUMNS SUBJECTED TO COMBINED BENDING AND SLIGHTLY ECCENTRIC AXIAL FORCE

A P-M diagram according to the current building code is built based on the specified material mechanical properties. As summarized in Table 10-8, there are two cases for each level where columns require strengthening.

Table 10-8

Level	Column	Section	N_{Sd} [kN]	M_{Sd} [kN m]	n_{sd}	m_{sd}	μ_s	Strengthening required
1	left side	bottom	-290	-10	0.61	0.08	0.29	NO
	left side	top	-282	16	0.59	0.12	0.29	NO
	central	bottom	-962	-9	2.02	0.07	0.29	YES
	central	top	-954	15	2.00	0.12	0.29	YES
	right side	bottom	-441	16	0.93	0.12	0.29	NO
	right side	top	-432	-34	0.91	0.26	0.29	NO
2	left side	bottom	-134	-34	0.28	0.26	0.29	NO
	left side	top	-128	35	0.27	0.27	0.29	NO
	central	bottom	-453	-32	0.95	0.25	0.29	NO
	central	top	-447	33	0.94	0.26	0.29	NO
	right side	bottom	-204	66	0.43	0.51	0.29	YES
	right side	top	-198	-67	0.42	0.52	0.29	YES

Because the central column of level 1 is subjected to a slightly eccentric axial force, FRP confinement is performed to ensure that the following equation is met:

$$N_{Sd} \le N_{Rcc,d} \tag{10.22}$$

A continuous CFRP wrapping of the column is carried out assuming the following parameters (Section 2.3.3.2: α_{fE}=α_{ff}=0.9):

- CFRP thickness: $t_{f,1}$= 0.167 mm.
- CFRP Young modulus of elasticity: E_f = 0.9·300000 N/mm^2 = 270000 N/mm^2.
- CFRP characteristic strength: f_{fk} = 0.9·3000 N/mm^2 = 2700 N/mm^2.

For Type-A application, the partial factors γ_f and $\gamma_{f,d}$ are set equal to 1.10 (Table 3-2, Section 3.4.1) and 1.20 (Table 3-3, Section 3.4.2), respectively. The environmental conversion factor, η_a, is set equal to 0.95 (Table 3-4, Section 3.5.1). A trial and error procedure is initiated for the determination of the number of CFRP plies, n_f, needed to satisfy Equation (10.29). Therefore, assuming n_f=1, the design axial capacity, $N_{Rcc,d}$ can be written as follows (Equation (4.40)):

$$N_{Rcc,d} = \frac{1}{\gamma_{Rd}} \cdot A_c \cdot f_{ccd} + A_s \cdot f_{yd} \tag{10.23}$$

where:
- γ_{Rd} is the partial factor for the resistance model, equal to 1.10 (Table 3-3, Section 3.4.2).
- A_c is the concrete cross section area.
- f_{ccd} is the design strength of confined concrete.
- A_s is the area of steel existing reinforcement.
- f_{yd} is the design strength of steel existing reinforcement, calculated according to the current building code.

The design strength, f_{ccd}, for confined concrete may be evaluated according to Equation (4.41):

$$\frac{f_{ccd}}{f_{cd}} = 1 + 2.6 \cdot \left(\frac{f_{l,eff}}{f_{cd}} \right)^{2/3} \tag{10.24}$$

where f_{cd} is the design strength of unconfined concrete according to the current building code, and $f_{l,eff}$ is the effective confinement pressure, depending upon member cross section shape and type of FRP application. The latter is given by Equation (4.42) as follows:

$$f_{l,eff} = k_{eff} \cdot f_l = k_{eff} \cdot \left(\frac{1}{2} \cdot \rho_f \cdot E_f \cdot \varepsilon_{fd,rid} \right) \quad (10.25)$$

where:

- k_{eff} (≤ 1) is the coefficient of efficiency defined by Equation (4.44):

$$k_{eff} = k_H \cdot k_V \cdot k_\alpha \quad (10.26)$$

- ρ_f is the CFRP geometric ratio; for rectangular cross section confined with continuous FRP reinforcement it may be written as follows:

$$\rho_f = \frac{2 \cdot t_f \cdot (b+d)}{b \cdot d} \quad (10.27)$$

where b and d are dimensions of the column cross-section.
- E_f represents CFRP Young modulus of elasticity in the fibers direction.
- $\varepsilon_{fd,rid}$ is the CFRP reduced design strain, taken from Equation (4.47):

$$\varepsilon_{fd,rid} = \min\{\eta_a \cdot \varepsilon_{fk} / \gamma_f ;\ 0.004\} = 0.004 \quad (10.28)$$

The coefficient of vertical efficiency, k_V, as well as the k_α coefficient can be set equal to 1 when continuous wrapping with fiber running perpendicular to the member axis is performed. The coefficient of horizontal efficiency, k_H, for rectangular cross sections can be written as follows (Equation (4.51)):

$$k_H = 1 - \frac{b'^2 + d'^2}{3 \cdot A_g} \quad (10.29)$$

where b' and d' are the dimensions shown in Figure 4-12 of Section 4.5.2.1.2, and A_g is the member cross sectional area.

The calculated axial capacity, $N_{Rcc,d}$, of the CFRP confined column is summarized in Table 10-9.

Table 10-9

Section	n_f	K_H	k_{eff}	ρ_f	$f_{1,eff}$ [N/mm^2]	f_{ccd} [N/mm^2]	$N_{Rcc,d}$ [kN]
bottom	1	0.48	0.48	0.0029	0.75	15.10	1102
top	1	0.48	0.48	0.0029	0.75	15.10	1102

Before CFRP wrapping takes place, cross section edges shall be rounded to 20 mm according to Equation (4.49).

10.6 CONFINEMENT AND FLEXURAL STRENGTHENING OF COLUMNS SUBJECTED TO COMBINED BENDING AND AXIAL FORCE WITH LARGE ECCENTRICITY

In this paragraph, design of CFRP strengthening for level 2 of the right side column subjected to combined bending and axial force is performed. Flexural CFRP reinforcement is carried out assuming the following geometrical and mechanical parameters (mode 1, Section 2.3.3.2: $\alpha_{fE}=\alpha_{ff}=0.9$):

- CFRP thickness: $t_{f,1}$= 0.167 mm.
- CFRP Young modulus of elasticity: $E_f = \alpha_{fE} \cdot E_{fib}$ = 0.9·300000 N/mm^2 = 270000 N/mm^2.
- CFRP characteristic strength: $f_{fk} = \alpha_{ff} \cdot f_{fib}$ = 0.9·3000 N/mm^2 = 2700 N/mm^2.

In addition, in the regions of the column close to the beams, the same CFRP material is applied as column wrapping as suggested in Appendix C. For Type A application, the partial factors γ_f is set equal to 1.10 (Table 3-2, Section 3.4.1). The environmental conversion factor, η_a, is set equal to 0.95 (Table 3-4, Section 3.5.1).

Due to confinement, the concrete design compressive strength can be written as follows:

$$f_{ccd} = f_{cd} \cdot \left[1 + 2.6 \cdot \left(\frac{f_{l,eff}}{f_{cd}} \right)^{2/3} \right] = 15.43 \text{ N/mm}^2 \tag{10.30}$$

A trial and error procedure is initiated according to Appendix C by calculating the non-dimensional coefficients:

$$n_{Sd} = \frac{N_{Sd}}{\overline{f}_{ccd} \cdot b \cdot d} \tag{10.31}$$

$$m_{Sd} = \frac{M_{Sd}}{\overline{f}_{ccd} \cdot b \cdot d^2} \tag{10.32}$$

where b and d are the dimensions of the column cross-section; $\overline{f}_{ccd}$ is equal to the design strength of confined concrete, f_{ccd}, suitably reduced if it is necessary.

Design is satisfied when the number, n_f, of CFRP plies is equal to 3 (Table 10-10, Table 10-11).

Table 10-10

Section	n_{Sd}	m_{Sd}	μ_s	u	n_f	u	Γ_{fk} [N/mm^2]	ε_{fd}	f_{fd} [N/mm^2]	μ_f
bottom	0.29	0.35	0.2	1	3	1	0.2	0.0086	2590.9	0.29
top	0.28	0.35	0.2	1	3	1	0.2	0.0086	2590.9	0.29

Table 10-11

Section	η_0	η_1	η_2	η_3	η	Failure mode	m	m_{Sd}	Checked
bottom	-0.19	0.12	0.23	0.73	0.72	2	0.009	0.43	YES
top	-0.19	0.12	0.23	0.73	0.70	2	0.011	0.43	YES

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