# **Background to European seismic design provisions for the retrofit of R.C. elements using FRP materials**

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## Abstract

This paper is a comprehensive state of the art background document on European seismic design provisions which was assembled in support of the development of design guidelines by the fib Committee 5.1 regarding the use of externally applied Fiber Reinforced Polymers (FRP) materials in the seismic retrofitting of reinforced concrete structures. In the context of developing design guidelines, the underlying mechanistic models that support the derivation of provisions were assembled after critical evaluation of the existing proposals and with careful reference to the available experimental evidence, the comparative assessment of past models in the literature, and requirements established from first principles.

**Keywords:** Seismic Retrofit; FRPs; Performance-Based Design; Rehabilitation

## Introduction

Fiber reinforced polymers have been introduced in Civil Engineering practice in the early 1990s, but they only became popular after they became known for their effectiveness as a fast remedy in retrofitting of damaged reinforced concrete and masonry structures in the wake of the catastrophic earthquakes in the end of that decade (Northridge 1994, Kocaeli 1999, Athens, 1999). Since that time extensive research undertaken to support design procedures of retrofits with FRP wraps and laminates, leading to several versions of design guidelines (ACI440, Fib Bulletin 14, 35 etc.). A large part of the research effort was directed towards development of confinement models, whereas all other actions were primarily considered for static loads (shear and anchorage); earthquake retrofit detailing was hampered by the need to also address global structural response issues in order to determine the retrofit priorities, whereas the literature on models that could support the development of guidelines was already marked by significant discord regarding the deformation-indices of retrofitted behavior thereby complicating the detailing process.

This paper aspires to establish a new-generation framework for the design of seismic retrofits using FRP materials. Following the prevailing earthquake and design practice, the paper establishes performance-based criteria for global and local retrofit requirements, so that the rehabilitated structure may develop acceptable, repairable levels of damage in a severe earthquake, and minimal (limited) levels of damage in the frequent event; FRP retrofits are designed aiming both for the enhancement of strength, as well as the deformation capacity and mode of failure control of the structure and its individual structural members. It is intended that this paper serves as the background for the development of European seismic retrofit provisions, using FRPs.

## 2. Global considerations

Structures get damaged during earthquakes when the displacement demand exceeds the displacement capacity of the individual members for the limit state considered. In general, the more localized the demand, the greater the severity of damage. These two conditions occur frequently in older reinforced concrete structures, where system deficiencies in lateral stiffness distribution (e.g. soft-storeys) are combined with lightly reinforced concrete members that possess little or no ductility and a negligible deformation capacity.

Ample experimental evidence exists today illustrating that externally applied FRPs, when used as confining jackets on lightly reinforced concrete members, are effective measures to increase ductility and deformation capacity by suppressing premature failure modes that usually occur in such members in the absence of proper seismic detailing. However, it is not possible to effect significant changes to the strength – and therefore the translational stiffness – of a reinforced concrete member by the mere addition of transverse FRP jackets. Strength and stiffness both depend on the amount and arrangement of primary (i.e., longitudinal) reinforcement[[1]](#footnote-1). Therefore rehabilitation of reinforced concrete (RC) members with FRP jacketing cannot affect the demand side of the design equation (apart from suppressing premature failure modes that might have otherwise controlled the response), whereas it can significantly enhance the supply. In this context, retrofitting through FRP jacketing is considered a *local* intervention in seismic rehabilitation of RC structures.

Note that FRP strips or laminates may be added as longitudinal reinforcement along the member length – most effectively when used as NSM reinforcement (Barros et al. 2006, De Lorenzis and Teng 2007, Yost et al. 2007, Novidis and Pantazopoulou 2008, Bournas and Triantafillou 2009, Bilotta et al. 2011, 2015, Dalfre and Barros 2013, Dias and Barros 2013, Sharaky et al. 2013, Coelho et al. 2015). Because strength and stiffness of the member is increased by the addition of the longitudinal FRP reinforcement, thereby affecting the global characteristics of the structure, this technique may be classified as a *global* intervention. Note here, that, in order to qualify as a global intervention for seismic applications, the added FRP strips should be anchored adequately beyond the critical cross sections where maximum flexural moments (i.e., demand for force development in the added reinforcements) are expected to occur. Such sections for example are in the ends of beams and columns, and at the base of structural walls. When this intervention is used in practice, it ought to be accompanied with transverse FRP jackets which would be needed in order to supplement the shear strength of the member, to prevent buckling of these added longitudinal reinforcements and to also support their anchorage.

If on first assessment it is deemed necessary to also moderate the demand, the retrofit solution should include *global* measures by which to increase the effective stiffness of the structure, *Keff*. Note that by increasing *Keff* the demand may be reduced in two different ways: (a) a higher effective stiffness results in a lower predominant period tending towards the left in the displacement spectrum, i.e. in the range of lower relative displacements; (b) through a more uniform distribution of deformation demand in the structure, which secures that the magnitude of deformation demanded of individual members is lowered[[2]](#footnote-2). For seismic response the effective stiffness is calculated from the contribution of the stiffness of the individual storeys of the structure by idealizing the structural system as a generalized single degree of freedom system that oscillates in its fundamental mode of vibration, *Φ* (Clough and Penzien 1993). The procedure practically evaluates the strain energy stored in the system during this vibration through the expression (i.e., work equivalent stiffness):

 (1)

where *Ki* is the lateral stiffness of the i-th floor and **  is the relative displacement that occurs in the i-th floor when the structure translates laterally according with its fundamental mode, assuming a unit displacement at the top (thus, ** is a normalized value). From Eq. (1) it is evident that it is more effective to increase *Keff* by optimizing the distribution of ** than by increasing the individual storey stiffness values.

## 3. Practical implementation of global measures

From the preceding discussion it follows that in practical implementation the displacement demand and the pattern of its distribution may be essential prerequisites to the application of a local intervention through addition of FRP jackets. The steps in this direction are defined in the following subsections.

**3.1 Determination if a global intervention is required**

The issue of whether stiffness additions are needed may be addressed easily by estimating the effective translational period and corresponding translational mode of the structure in the direction of interest. The following criteria apply:

(a) *Keff* should be increased if the effective translational period *Teff* (based on secant to yielding sections analysis) exceeds by more than 25% the empirical reference value prescribed by EN1998-1 (2004), which estimates *Tref*, as:

 (2)

where *Htot* is the total building height, measured from the crest of a box-type rigid basement or otherwise from the level of the foundation.

(b) *Keff* should be increased if, by inspection of the translational shape of vibration there is evidence of localization of deformation in few storeys only, or if there is significant discrepancy in ductility demands between members that belong to a given floor.

**3.2 Target for an improved period estimate, *Ttrg***

For the vast majority of structures the period after retrofit will lie between the milestone values *TB* and *TD* of the EN1998-1 (2004) Type I earthquake design spectra. For this period range the elastic spectral displacement demand may be estimated from:

 (3)

In the preliminary stage of calculation, it may be assumed that this displacement will be increased by about 20% when transferring from the spectrum to the actual structure. The displacement value may be further increased from the above value if inelasticity occurs. The total average elastic drift ratio (denoted as *θdem* for drift demand) for the retrofitted structure is approximated by:

 (4)

The target or improved period, *Ttrg*, of the retrofitted structure may be selected based on experience, as a value between *Tref* [from Eq. (2)] and the initial *Teff*. A note of caution is that the cost of the intervention increases as *Ttrg* is reduced getting closer to *Tref*. Alternatively, *Ttrg* may be selected by requiring that the average drift demand *θdem* of the structure [Eq. (4)] will not exceed a preset limit value, which after substitution of Eq. (3) in Eq. (4) will yield the required value for *Ttrg*. Such preset limit values may be 0.5% (for performance limit A: Damage Limitation, *μθ*≈1), 1.25% (B: Repairable Damage, *μθ*=2.5) and 2% (C: Collapse Prevention or Life Safety, *μθ*>3.5). It is not advisable to allow for *μθ*>2.5 for retrofitted structures.[[3]](#footnote-3) (Figure 1 plots the base shear, *V*, against the lateral drift ratio of the structure, given as a multiple of the value at yielding; the ductility factor *θ* is the multiplier of *y*).

**3.3 Target for improved shape of the fundamental mode**

**θy=0.5%**

**2.5θy**

**3.5θy**

**A: Immediate occupancy**

**B:Repairable damage**

**C: Collapse prevention**

**Base Shear, V**

*Figure 1: Performance limits.*

**Drift, θ**

With the selection of the target shape the objective is to achieve an optimum distribution of deformation throughout the structure. A number of simple displacement patterns may be used as benchmark for selecting a target shape for the fundamental vibration shape in retrofitting an existing, seismically deficient structure (Fig. 2). The closer towards a triangular or flexural shape the greater the extent of the required intervention and thus the associated cost. The shear type shape could serve as an acceptable compromise in lower cost retrofits, where a possible soft storey formation may be re-engineered towards this option for moderate improvement. The selection of the target response shape could be:

* Shear – type response: The shape is approximated by *Φ(zi)=sin(πzi/(2Htot))* simplified to *Φi=sin(π·i/2n)* for equal storey heights, where *n* is the total number of storeys (*zi* is measured, as in the case of *Htot*, from the crest of box-type basement or from the level of foundation to the storey of interest, parameters *n* and *Htot* have been defined in Eqs. (1) and (2)). The tangential drift (*dΦ/dz*, Fig. 3a) is more moderate in the lower floors above the first storey of the structure. Buildings with this fundamental response shape have a natural tendency for localization of damage in the lower floors. Significant beam rotation demands and potential plastic hinge formation are only expected in the lower floors.
* Flexural – type response: The trigonometric approximation for this pattern is *Φ(zi)=1-cos(π·zi/(2Htot)),* simplified to *Φi=1-cos(π·i/2n)* for equal storey heights. Beam rotations in structures of this type follow the distribution of the tangential drift (*dΦ/dz,* Fig. 3b) thus, damage in beam plastic hinge regions is expected to be maximum in the upper floors, whereas in the case of walls and columns, plastic hinging is expected at the base (*i=0, z=0*), consistently with the anticipated maximum base shear value.

**Φ(zi)=sin(πzi/(2Htot))**

**Φ(zi)=zi/Htot**

**Φ(zi)=1-cos(πzi/(2Htot))**

**(a) shear**

**(b) triangular**

**(c) flexural**

**Retrofitted building**

*Figure 2: Lateral displacement profiles; (a) shear; (b) triangular; (c) flexural.*

* Triangular response shape: It is described by *Φ(zi)= zi /Htot* which may be simplified to *Φi=i/n* for equal storey heights equal to *hst* (*n·hst=Htot*). It represents the ideal scenario of constant interstorey drift ratio throughout the height of the structure and the best possible case for even damage distribution throughout the structure. However, it is difficult to be actually achieved in practice.

***θ­i***

**hst**

**(b)**

***(dΦ/dz)|z=zi + hst***

**hst**

***θ­i***

**(a)**

***(dΦ/dz)|z=zi + hst***

*Figure 3: (a) Shear-type and (b) flexural-type response floor rotations.*



**(a)**

**(b)**

**(c)**

**(d)**

**(e)**

**(f)**

*Figure 4: (a), (b), (c): Stiffness to mass ratio for the first storey, K1/m, versus period for up to 8-storey frames. To obtain required K1 values multiply the ordinate with the mass m (in tons); (d), (e), (f): Floor stiffness ratios ki (=Ki:K1) for different lateral deflection shape patterns for 2- up to 8-storey frame buildings.*

**3.4 Determining required stiffness**

Engineered modification of the fundamental mode of lateral vibration is achieved through a weighted distribution of added stiffness along the height of the building. In case of the three benchmark cases (triangular, shear and flexural shapes in Fig. 2) the solution is provided in the charts of Fig. 4. The charts were derived considering a minimum storey height, *hst*=3m and unit storey mass, *m*=1 ton; they can be used in order to define for a target period and chosen deflection shape (after the user selects the target deflection shape between the triangular, shear and flexural); then, from the charts of Fig. 4a, b or c, the required stiffness of the first storey is obtained directly, along with the required distribution of stiffness along the height of the retrofitted building (from the charts of Fig. 4d, e, or f, and given the number of floors in the structure, obtain the required stiffness for all floors as a fraction of the first storey stiffness)[[4]](#footnote-4).

## 4. Practical implementation in retrofit design

The procedure described in Section §3 enables estimation of the required storey stiffness for a given building (i.e., with known distribution of mass) so as to achieve the specified target period and fundamental mode of vibration characteristics according with the designer’s choice. The last step in the procedure involves the selection of the global intervention method and the detailing of the actual members of the building in order to achieve the stiffness addition defined in Section §3.4.

Global intervention methods include but are not limited to the following:

(1) Addition of FRP longitudinal reinforcement (NSM or externally bonded FRP laminates)

(2) R.C. jacketing of selected columns in the building

(3) Addition of R.C. wall elements

(4) Addition of steel X-braces

(5) Addition of masonry infills (not common in North America).

Note that FRP jacketing is only pertinent for *local* interventions and is not included in the *global* strategy of the retrofit.

The required storey stiffness *Ki* of the retrofitted structure that comprises *ℓcRC* R.C. jacketed columns, *ℓw* R.C. walls, *ℓX* spans of X-brace metallic pairs, *ℓmw*masonry walls, and *ℓcf* columns strengthened with longitudinal FRP laminates (EBR or NSM) is equal to:

 (5)

In Equation (5), variables *ℓcRC, ℓw, ℓX, ℓmw, ℓcf* are defined as follows:

*ℓcRC* is the number of columns retrofitted with RC jackets in a single floor; *ℓw* is the number of walls added for stiffening the structure in the direction of action; *ℓX* is the number of X-brace pairs added in the floor to add stiffness – in the direction of action; *ℓmw* is the number of infill panels added in the floor in the direction of action; and *ℓcf* is the number of columns strengthened with longitudinal FRP strips (externally bonded or NSM).

The contributions of each of these techniques/elements to the storey stiffness, *Ki*, are listed in Thermou and Pantazopoulou (2014) and are summarized here for completeness in Appendix. Only the possible contribution of FRP to the stiffness terms, *Kjf*, are considered in the following detailing paragraphs (in the main body of the paper).

## 5. Detailing of FRP interventions for seismic applications

Seismic retrofitting of RC structures with FRP may be used in order to upgrade a variety of structural deficiencies, if upon assessment according with the established code framework (EN 1998-3, 2005) it is shown that seismic safety may be compromised at the design performance limit state. Both for evaluating the structure’s safety and in defining the retrofit objectives, reference is made to verification of acceptable limit states as described in the reference code document.

Similarly, the seismic hazard considered for the retrofit is identical to that used for new designs, unless the National Standards enable through special provisions, to assign a different importance level category to the retrofitted structure so as to account for a residual service life different from the 50-year standard.

Analysis of the retrofitted structure may be used to check against the established acceptance criteria, following the methods of analysis used in the assessment procedure.

Material Safety factors refer to usual FRP materials typically used today (GFRP, CFRP and AFRP with strengths ranging from 1500 to 3500MPa, and nominal rupture strains from 2.5% down to 1.5%). For retrofit design these are: (a) For existing concrete and steel reinforcement, the confidence factors are used to divide mean material strength values depending on the knowledge level attained (EN 1998-3, 2005). (b) For FRP: The material safety factor depends on the development method of the FRP material and the member classification (primary or secondary as per EN 1998-1, 2004) as listed in Table 1.

*Table 1: FRP material redundancy safety factors*

|  |  |  |
| --- | --- | --- |
| FRP is anchored on: | Primary member | Secondary Member |
| (a) Brittle substrate | f = 3 | f=2.3 |
| (b) Fully wrapped FRP layer (i.e. anchorage by lap-splicing the ends of the layer in closed jacket) | f = 1.5 | f=1.25 |

**5.1 Strategies in FRP retrofitting**

The FRP material to be used in the retrofit solution and its arrangement depend on the overall objectives of the retrofit design. A general guideline is to aim for a uniform distribution of strength and stiffness among members in any given floor, in order to minimize the risk of disproportional damage of any single element[[5]](#footnote-5). The implication is that during an earthquake, for any given magnitude of lateral displacement, members with different aspect ratios in a single floor reach very different states of damage. The same effect is observed if the structure has plan irregularities that cause torsional response. Clearly, major building irregularities cannot be eliminated using FRP as a strengthening technique, although the addition of FRP strips as longitudinal reinforcement belongs to the global interventions as it can be used to increase strength and stiffness of individual members.

Thus, a good strategy is to selectively retrofit members that belong to the lateral load resisting system so as to achieve similar relative drift ratios at yielding, and to also enhance deformation capacity through confinement. It is essential to eliminate, through FRP jacketing, brittle failure modes, so that the flexural capacity of the member may be fully developed and sustained up to the ductility level required by the design.

Extensive experimental evidence supports the use of FRPs as a pertinent material in seismic retrofitting applications, particularly in reinforced concrete beams, columns, walls and beam-column connections. FRP retrofit-schemes that are well documented and support the establishment of detailing rules include the following solutions:

(1) Increasing the flexural stiffness and strength of a linear member by using externally bonded or near-surface mounted FRP strips in the role of primary reinforcement, that is, by the addition of reinforcement running parallel to the longitudinal axis and attached near the tension side of the strengthened member.

(2) Increasing the member shear capacity by using FRP material with fibers running orthogonal to the direction of the axis of the strengthened member.

(3) Increasing the ductility of end sections of beams and/or columns by using FRP material wrapped around the member cross section.

(4) Improving the efficiency of lap splices by using FRP material wrapped around the member cross section.

(5) Delaying the occurrence of buckling of steel longitudinal bars by using FRP material wrapped to the member cross section.

(6) Increasing the capacity of beam-column joints to diagonal tension by using FRP material installed with fibers located along the principal tensile stresses.

As interventions (2) – (6) listed above cannot significantly alter the reference flexural strength and stiffness of the retrofitted member, all these techniques are classified as *local measures* (or *local interventions*). It is required that localized strengthening shall not reduce the overall ductility of the structure.

In detailing the retrofit solutions each retrofitted member is designed using principles of Capacity Design. To secure adequate ductility, flexural yielding should control the response of the retrofitted member. So the member retrofit details should be proportioned with reference to flexural overstrength. The shear force associated with flexural yielding of the member, from the static relationship depicted in Fig. 5, is referred to as flexural shear demand, *Vflex*.

When considering individual members, case (1) in the list of intervention measures given is a global measure, effectively increasing *Vflex*. On the other hand, local strengthening schemes of individual linear members, i.e. cases (2) – (5) above, have relatively little effect on *Vflex*, and they depend on the confining action of the FRP reinforcement. Thus, the efficacy of the strengthening scheme in these cases depends on the magnitude of the confining pressure. To illustrate the procedures for detailing, the role of the FRP properties in each mechanism of resistance associated with the strengthening objectives of individual members listed above, will be reviewed briefly in the following.

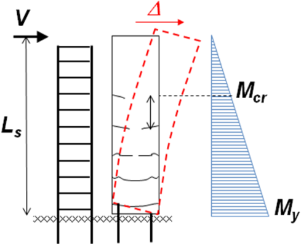
In order to perform the necessary design calculations and to control the occurrence of the modes of failure, a static model is envisioned for each member undergoing retrofit depicted in Fig. 5. In this static model, the member develops constant shear force along its length reaching maximum flexural moment at the end points of their deformable length where partial restraint to rotation may exist (for example, during lateral sway, a column is considered to develop maximum moment at the base and at the top cross section at the beam soffit; a beam is considered to develop maximum moments at the end supports, at the face of the columns; a flexural wall is considered to develop maximum moment at the base).

To implement this step, global displacement demands need to be determined and to be subsequently converted to local deformation demands of the members to be retrofitted through the above scenarios.

### 5.2 Determining the displacement demand of the individual structural members

**H**

**db**



**Htot**

**2/3Htot**

**LV**

**Vflex**

**(a)**

**(b)**

**(c)**

*Figure 5: (a) Static model used for beam-column elements undergoing lateral sway. (b) The cantilever part has the same moment distribution as the swaying column over the length from contra-flexure to the face of the support. Owing to this similarity the cantilever member is used to illustrate the concept of shear span, Lv = H/2 and the relation between shear demand and flexural moment strength: V=M/LV. (c) Static model for structural walls. The red line defines an “equivalent” linear moment diagram to relate shear demand with moment strength: V=M/(2/3Htot).*

The global retrofit objectives are defined in terms of target fundamental period, target response shape, and global behavior factor, *q*.

With reference to Fig. 1, it is recommended that *q* should not exceed the value of 2.5 for ordinary structures (for the performance limit B: Repairable damage *q≈=*2.5). Higher values should be avoided if the structure has been designed to previous standards, or has irregularity in plan or in height. Lower values of *q* are acceptable.

To illustrate how global considerations of the retrofit design may be used to determine local design requirements, the following steps are considered:

- the required displacement ductility *μΔ* of the structure may be estimated from:

 (6)

where *TC* is the end of the plateau of the Type I spectrum for the design soil conditions (see EN1998-1 2004). Depending on the target response shape, required displacements at the individual floors of the structure are determined from (EN 1998-3 2005):

 (7)

where, *Φi* is the coordinate of the target response shape in the *i*-th floor, and *ΔΦi =Φi - Φi-1*.

-The required curvature ductility at the critical sections of members in the *i*-th floor may be obtained with reasonable approximation from:

 (8)

whereas the maximum compression strain demand for the columns, *εcu,c*, may be estimated from (KANEPE 2013):

 (9)

In Eq. (9), parameter *νd,max* is the maximum axial load ratio of the typical column for the seismic combination (defined in Section 6.1)[[6]](#footnote-6) and *εsy* is the yield strain of steel.

**5.2.1 Increasing of the local rotational capacity of R.C. members**

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, the term *i*in Fig. 3) at a distance equal to the shear span: *Lv=M/V*. (In buildings with a “shear” type mode of lateral deflection this rotation is also equal to the ratio of the relative displacement between the two above mentioned sections to the shear span, referred to as relative storey drift ratio; however, if beams also participate in the deformation of the storey, then the relative drift ratio defined in the preceding far exceeds the column rotation due to the rigid body rotation of the base as depicted later in Fig. 10).

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.

**5.2.2 Capacity design criterion**

The application of the capacity design criterion (hierarchy of resistance) implies the adoption of mechanisms of behaviour in the structure such as to prevent by design the formation of all potential plastic hinges in the columns. In “weak column-strong beam” situations, which are typical of 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.

## 6. FRP as a means of enhancing strength

### 6.1 Increasing flexural strength and stiffness of R.C. members by adding longitudinal FRP reinforcement

With the addition of FRP strips on the tension side of a member parallel to its longitudinal axis the objective is to enhance flexural strength (and therefore, stiffness) of the member. In this capacity the FRP reinforcement functions as tension reinforcement; if upon reversal of the load, the FRP layers fall within the compression zone of the member, then their contribution to strength may be neglected, until adequate data exist to corroborate any other design decision.

Steps for detailing the retrofit: Consider the cross-section shown in Fig. 6.1: A prismatic cross section with known initial geometry and material properties is given. The cross section carries an average design axial load, *NG+0.3Q-E≤NG+0.3Q≤NG+0.3Q+E* obtained from the seismic design combination. For this axial load, which is referred to as the axial load ratio, [*Ed*=*NG+0.3Q/(fcdbd)*], and the longitudinal reinforcement ratios, *s1* and *s2*, the reference flexural strength of the cross section is *MRdo* (moment and axial load are considered to be acting at the centroid of the cross section).

To increase the flexural strength to a required value *ΜEd*, FRP strips shall be either externally bonded or embedded in near-surface grooves in the tension sides of the member as shown in Fig. 6.2. The added reinforcement may be confined by a transverse jacket (this ought to be pursued if the shape of the cross section allows it). Figures 6.1 and 6.2 depict all possible combinations for illustration purposes – i.e., NSM or EBR longitudinal laminates, either confined by a transverse jacket (see top region in the sections in Fig. 6.1), or left unconfined (see bottom region in the sections of Fig. 6.2). The maximum allowable stress of the longitudinal FRP depends on the chosen arrangement as explained below.

Essential requirements for this type of retrofit are:

(a) The extreme layer of embedded longitudinal tension steel reinforcement should undergo excessive yielding at the ultimate limit state (*s1,min>εyd*).

(b) Maximum compressive strains in unconfined concrete in the compression zone shall not exceed *εc,u*.

(c) Tension strains in the FRP longitudinal reinforcement, *f*, shall not exceed the design limit *f,max=fu /f*, where *f* is taken from Table 1 for primary reinforcement on a brittle substrate.

**x**

**b**

**h**

**d**

**d2**

***εc,u≤*0.0035**

***εs1,min≥ εyd***

FRP jacket (fibers transverse to the long. axis of member)

***Fs1***

***Fs2***

***Fc***

***NEd***

***MEd***

***fcd***

***0.85fcd***

**β⋅x**

**≈**

*Figure 6.1: Various types of flexural strengthening of prismatic column / beam cross-section. The red line illustrates the jacket arrangements implied by the various values of a1, and a2 in Eq. (10) – the yellow line marks the adhesive layer. Clearly this should not be interpreted as a recommended arrangement of the jackets. Wherever possible the jacket ought to be wrapped fully around the cross section.*

possible transverse NSM bar for strengthening of slab beam connection

Shear bolt

FRP jacket (fibers transverse to the long. axis of member)

Anchor device

1. **(b)**

*Figure 6.2: Various types of flexural strengthening of T- beam cross-sections by addition of longitudinal NSM or EBR reinforcement. The outer solid line illustrates jacket arrangements required to secure the flexural intervention. Due to the presence of slab longitudinal reinforcement addition of top reinforcement is more rarely needed (see (a)); here it is important to provide transverse top reinforcement to secure the participation of slab reinforcement in beam flexural strength (see dashed line in (b)).*

For dimensioning of the reinforcement, the axial tensile strain *fd* in the FRP layer shall not exceed the limit:

 (10)

*a*1=1 for EBR-FRP layer,

*a*1=1.4 for NSM-FRP layer

*a2*=1.0 if no transverse reinforcement has been applied over the FRP reinforcement

*a2*=1.4 if transverse reinforcement has been applied over the FRP reinforcement in the form of jacketing (Figs. 6.1 and 6.2), or if clamping of the FRP-layer is achieved by means of chemical or other anchors (marked by blue arrows in Fig. 6).

For the sake of illustration, the dimensioning procedure is presented below, for the case where concrete crushing *c*=*cu* is prioritized as the limiting failure mode after reinforcement yielding. The procedure for the case of FRP failure follows similar principles. Requirements (a) – (c) above may be expressed as limits on the normalized depth of compression zone of the cross section after retrofit, *=x/d*:

 (11)

To calculate the required area of the added tension reinforcement *Af=f ·(b·d)*, the values of ** and *f* are solved from the equilibrium requirements:

- Sum of axial forces = 0 (*Ed >*0 for compression)

 (12)

Parameter **is the depth of the equivalent rectangular stress block of concrete compressive stress, normalized by the depth of compression zone, *x* (see Fig. 6.1; the intensity of the stress block is 0.85*fcd*). Parameters *s1* and *s2* are the available areas of tension and compression reinforcement in the cross section, respectively, calculated as the ratios of total bar area in each side to the effective cross section area, i.e. *s1*=*n1Db12/(4hd)*, where *n1* and *Db1* are the number and the diameter of the tension bars.

-An upper limit is obtained for *f* by substituting *=bal* in the above[[7]](#footnote-7).

-For the retrofit to be possible it is also required that:

 (13)

-To find the required area of the FRP layer in order for the strengthened cross section to have a flexural strength of *MRd*>*MEd* in the presence of a design axial load *NEd =νEdbdfcd*, the following procedure is used: First, the sum of moments is considered about the centroid of the FRP layer:

 (14a)

Note that both *MEd,f* and the normalized value *Ed,f* are defined with reference to the centroid of the FRP layer (this is the significance of the subscript *f*); *MEd,f* is calculated first, given the design values of moment and axial load of the retrofitted cross section. Next, the design value of*Ed,f* is obtained from *MEd,f* after normalizing with *bd2fcd*. This is set equal to the normalized moment of internal forces about the same point of reference, given by:

 (14b)

where term *Ro* in Eq. (14.b) only depends on geometric characteristics of the original cross section. The FRP layer is calculated so that the total value for *Ed,f* meets the strength demand of the retrofitted cross section. The first term in the right-hand side of Eq. (14.b), given by:

 where,  (14c)

was tabulated for easy reference in Table 2 for usual values of (*d2/d*).

*Table 2: Normalized moment Ed,f for various values of maximum allowable FRP strain, fd*

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| *d2/d*= | **0,05** | | **0,10** | | **0,15** | |
| ***εfd*** | *ξ* | ***ΔμEd,f*** | *ξ* | ***ΔμEd,f*** | *ξ* | ***ΔμEd,f*** |
| **0,0022** | 0,644737 | **0,347276** | 0,675439 | **0,381137** | 0,70614 | **0,416573** |
| **0,0024** | 0,622881 | **0,339206** | 0,652542 | **0,372281** | 0,682203 | **0,406894** |
| **0,0026** | 0,602459 | **0,331431** | 0,631148 | **0,363748** | 0,659836 | **0,397567** |
| **0,0028** | 0,583333 | **0,323944** | 0,611111 | **0,355531** | 0,638889 | **0,388586** |
| **0,003** | 0,565385 | **0,316737** | 0,592308 | **0,347621** | 0,619231 | **0,379941** |
| **0,0035** | 0,525 | **0,29988** | 0,55 | **0,32912** | 0,575 | **0,35972** |
| **0,004** | 0,49 | **0,284553** | 0,513333 | **0,312298** | 0,536667 | **0,341334** |
| **0,0045** | 0,459375 | **0,270595** | 0,48125 | **0,296979** | 0,503125 | **0,324591** |
| **0,005** | 0,432353 | **0,257855** | 0,452941 | **0,282998** | 0,473529 | **0,309309** |
| **0,0055** | 0,408333 | **0,246198** | 0,427778 | **0,270203** | 0,447222 | **0,295326** |
| **0,006** | 0,386842 | **0,235501** | 0,405263 | **0,258464** | 0,423684 | **0,282495** |
| **0,0065** | 0,3675 | **0,22566** | 0,385 | **0,247663** | 0,4025 | **0,270689** |
| **0,007** | 0,35 | **0,21658** | 0,366667 | **0,237698** | 0,383333 | **0,259798** |
| **0,0075** | 0,334091 | **0,208181** | 0,35 | **0,22848** | 0,365909 | **0,249723** |
| **0,008** | 0,319565 | **0,200392** | 0,334783 | **0,219932** | 0,35 | **0,24038** |
| **0,0085** | 0,30625 | **0,193152** | 0,320833 | **0,211985** | 0,335417 | **0,231695** |
| **0,009** | 0,294 | **0,186405** | 0,308 | **0,204581** | 0,322 | **0,223602** |
| **0,0095** | 0,282692 | **0,180105** | 0,296154 | **0,197667** | 0,309615 | **0,216045** |
| **0,01** | 0,272222 | **0,17421** | 0,285185 | **0,191197** | 0,298148 | **0,208973** |
| **0,0105** | 0,2625 | **0,168683** | 0,275 | **0,18513** | 0,2875 | **0,202343** |
| **0,011** | 0,253448 | **0,16349** | 0,265517 | **0,179431** | 0,277586 | **0,196114** |
| **0,0115** | 0,245 | **0,158603** | 0,256667 | **0,174068** | 0,268333 | **0,190252** |

Values in Table 2 were calculated by taking *β* equal to 0.8 which corresponds to the ultimate strain of the extreme compressed fiber of the cross section, *εcu*=0.0035. Ranges of parameters outside the table represent cases where the resulting normalized depth of compression zone, **, does not satisfy the limits set by Eq. (11), and therefore this type of strengthening would not be advisable as it will,

(a) embrittle the cross section, for ** values above the upper limit of Eq. (11), in the shaded part of Table 2[[8]](#footnote-8), or

(b) lead to debonding along the anchorage of the added FRP reinforcement, for **values below the lower limit of Eq. (11).

The Table 2 is entered with the value of *d2/d* and the required normalized moment increase *(calculated about the centroid of the longitudinal FRP layer)*. The estimated (from the Table 2) value of **, is entered in the expression for *f* (Eq. 12) to yield the required area of FRP tension reinforcement. The strain that develops in the FRP layer is given in the left column, which may be checked against the allowable *fd* value determined from Eq. (10).

**6.1.1 Additional requirements**

FRP reinforcements used as post-installed primary reinforcement to strengthen RC structural members for seismic applications shall be anchored so as to be able to support their force at the critical section where moment is maximum (e.g. at the upper and lower cross sections over the free length of a column, at the end cross sections of a beam, and at the base cross section of a structural wall). The required effective anchorage length *Le*, available on each side of a critical section, shall satisfy the design requirements for development length given in MC2010 (2010).

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.

The increased flexural strength corresponds to an increased flexural shear demand, *VEd=MRd /Lv* (Fig. 5). The retrofitted member shall be checked in shear, and additional shear reinforcement shall be provided to ensure that factored shear resistance, *VRd*, shall exceed *VEd*.

The stiffness increase attained by the addition of the FRP reinforcement may be quantified by the magnitude of the effective *EIj* of the j-th member’s strengthened cross section, associated with the onset of yielding of the embedded longitudinal tension steel. Thus, the translational stiffness of a column is:

 (15)

Note that (*Hj /h*) is the aspect ratio of the member; *My,j* is the moment resistance of the j-th strengthened member, at the onset of yielding of longitudinal reinforcement.

**6.1.2 Additional detailing considerations**

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

Longitudinal fibers used for strengthening R.C. members subjected to combined bending and axial load shall be properly confined to avoid debonding and concrete spalling under cyclic loads.

### 6.2 Increasing the deformation capacity of R.C. members through FRP jacketing

To increase the deformation capacity of an RC member any type of undesirable brittle failure should be eliminated. The member shall be designed to develop ductility during seismic load reversals. Ductility is achieved if the longitudinal steel reinforcement of the member is engaged in post-yielding response prior to the occurrence of any of the following:

(a) Delamination of concrete cover in the compression zone

(b) Failure of lap-splices or reinforcement anchorages

(c) Diagonal tension failure of the member’s web (shear)

(d) Control of bar buckling in the compression zone of a member

(e) Disintegration of the confined concrete core under high compression strain demands

FRP jacketing may be used to effectively eliminate these occurrences and to also enhance the deformation and ductility capacity of a reinforced concrete member. The term FRP jacketing refers to any type of application of the material where the primary fibers are oriented transversally to the longitudinal axis of the upgraded member and at a minimum of three faces (properly anchored U-shaped and -shaped types exclusively) of the member’s cross section so as to facilitate confining action against any dilation of the concrete (i.e. due to axial load, shear transverse tension or dilation produced by the bond action of a ribbed bar). Interventions that may be necessary to achieve this objective were termed local measures, and were listed in Section §4.1. A critical design parameter in all cases is the confining pressure introduced by the FRP jacket.

**6.2.1 Calculation of confining pressure in FRP-encased concrete**

The confining pressure exerted by the FRP jacket encasing a reinforced concrete member is estimated with reference to Fig. 7a (-shaped FRP types exclusively). FRP stresses and confinement exerted on the encased cross section vary from the corners to the center. The average confining pressure *σx*, acting along the *x* axis, may be estimated considering equilibrium on a plane intersecting the cross section along line A-A. Similar is the calculation of the average confining pressure, *σy* acting in the *y*-direction.

 (16a)

 (16b)

where, *fw-χ* is the FRP web reinforcement ratio (geometric ratio) provided in the *x* direction (*2tf /h*) for continuous jacket having an effective thickness of *tf*. Similarly, *fw-y=(2tf /b)*.

The effective thickness is estimated from the number of FRP layers placed in the jacket, *n*, and the thickness of a single layer, *to*. Therefore, *tf =to·n0.85* for *n≥4* else *tf =to·n* for *n<*4 [[[9]](#footnote-9)] (KAN.EPE 2013)*.*

Parameter *εfd* is the design value for strain capacity of the transverse jacket, defined in Section §6.2.3 below.

Unconfined regions

Dilation owing to outwards bending of stirrups

Confined zone

Stirrups in deformed state

**b**

**A**

**ff**

**bo**

**ho**

**h**

**h'**

**y**

**χ**

**R**

Square section:

***αf=0.5***

Rectangular section of aspect ratio > 3: ***αf≈0.0***

*Figure 7:* *a) Definition of terms for estimation of confining pressure and b) -shaped FRP types exclusively*

**(a)**

**(b)**

Parameter *sw-χ* is the transverse (web) steel reinforcement ratio in the *x*-direction: for links oriented in the *x*-direction placed along the member length at a clear spacing of *s*, having a total sectional area of *Asw-x*, this is defined by: *sw-x =Asw-x/(s·ho)*. Similarly, *sw-y=Asw-y/(s·bo)*.

A uniform lateral pressure is assumed to confine the FRP-encased concrete in compression. This pressure, denoted by *σlat* is the average of *σχ* and *σy* defined above. To account for the reduced efficiency of confinement in rectangular cross sections (Fig. 7b) an effectiveness coefficient, *f*, is used in order to moderate the FRP-component of confining stress. This is similar to the effectiveness coefficient, *w*, used for stirrup-generated confinement (EN 1998-1, 2004):

 (17a)

Parameters *ρfv* and *ρsv* are the volumetric ratios of transverse reinforcement (Fig. 7a):

 (17b)

**6.2.2 Confinement effectiveness coefficients *f*, *w***

The confinement effectiveness coefficient is the volume ratio of the encased member that is effectively confined. With reference to Fig. 7a, *w*is defined for stirrup confinement according with EN 1998-1 (2004) as:

 (18)

Similarly, the effectiveness of confinement provided by FRP jackets is obtained as the volume ratio of the effectively confined part of the member (*fib* Bulletins 14 (2001), 35 (2006), 40 (2007)):

 (19)

Parameter *ρg* is the longitudinal reinforcement ratio of the member’s cross section, *b’* and *h’* are the straight sides of the rectangular cross section encased by the jacket after chamfering the corners with a radius *R*. By definition, the effectiveness coefficient is always less than 1. In lightly reinforced members that are considered for retrofit with FRP jacketing, the contribution of the stirrups may be neglected with no significant loss of accuracy. The effectiveness coefficient of the FRP jacket, *a****f***, decays fast with increasing aspect ratio (*b/h*) of the member’s dimensions (Fig. 7b). Further reduction occurs if the FRP jackets are placed in strips and not continuous over the member length.

* For members with a circular cross section and continuous jacketing (i.e., no strips) *a****f****=*1.
* For members with a square cross section and continuous jacketing *****f****=*0.5.
* For cross sections with an aspect ratio *>*3 the confinement effectiveness is practically negligible: *****f≈***0. However, FRP jacketing in these cases is a very effective means of providing web reinforcement (e.g. in structural walls).

**6.2.3 Design tensile strain of the FRP jacket, *εfd***

The allowable tensile strain of the jacket, *εf*, shall not exceed the design limit, *f,max=fu/f*, where *f* is taken from Table 2 depending on the jacketing arrangement:

(a) Fully wrapped retrofit arrangement refers to closed jackets ( -shaped) that fully encases the member.

(b) Anchorage on brittle substrate refers to open jackets (U-shaped) that do not enclose the member on all sides.

For proportioning of the FRP jacket, the axial tensile strain *fd* in any FRP layer shall not exceed the limit[[10]](#footnote-10):

*Figure 8:* *Definition of factor n1 vs the larger member cross section side for several values of R and Db*

 (20)

Factor *1* accounts for the radius of chamfer *R*, at the corners of the member (also known as strain efficiency factor, see Pantazopoulou et al. 2015, Tastani et al. 2006, Pellegrino and Modena 2010, see also Fig. 8):

 (21)

Parameter *Db* is the embedded corner bar diameter. Equation (21) is valid only for rectangular cross sections (*b’* the largest cross sectional side), for circular members *n1*=1.

Factor *2* accounts for the development length of the wrap:

 (22)

where *lbmin* is the minimum required overlap length of the exterior jacket layer (i.e. as it is calculated by implementing Eq. (23) or Eq. (24)) and *lbavail* is the available length of the cross section side where the FRP is to be anchored.

Factor *3* accounts for the redundancy of the jacket against debonding failure.

* For fully wrapped jackets *3=*1.0
* For U-type arrangements with special details in the ends in order to secure the jacket against debonding (e.g. adhesive anchors, NSM details, etc.) *3=*1.0
* For U-type arrangements without special measures against debonding, *3=*0.85
* For straight layers with special details in the ends in order to secure the jacket against debonding (e.g. adhesive anchors, NSM details, transverse confining wraps), *3=*0.9
* For straight layers (parallel to the web height) without special measures against debonding, *3=*0.6. (Note that this arrangement is discouraged by most relevant design codes due to the high risk of debonding (e.g. ACI440, fib 9.3 Bulletin 35, KAN.EPE 2013)); however it may be improved by the application of chemical anchors or other effective clamping means)

**6.2.4 Stress-strain law for FRP- confined concrete**

The confined concrete strength *fcc* and the corresponding strain at attainment of peak stress, *εcc*, in the compression zone of the encased cross section may be calculated from the classical confinement model of Richart et al. (1928) moderated to account for the greater compliance of jackets as compared to conventional stirrups[[11]](#footnote-11):

 (25)

By substitution of Eq. (17a) in Eq. (25), and assuming *εco*=0.002(strain at peak stress of unconfined concrete), the following are obtained:

 (26)

The failure strain of confined concrete, *εcu,c* corresponding to a compression strength reduction in excess of 15% is obtained from (Pantazopoulou et al 2015, Fig. 9(b)):

 (27)

Coefficient ** varies linearly between the two bounds for intermediate axial strain values. This parameter accounts for the reduced jacket effectiveness when a very high confinement is present: at such a very high limit axial compaction of confined concrete accounts for part of the observed axial strain capacity, without engaging the jacket through dilation of the core. Note that material safety factors are not used for concrete characteristic strength in determining the compression stress-strain law. Such a safety factor may affect unfavorably the estimated hierarchy of failure in establishing capacity design principles. It is recommended that a safety factor may be applied on the calculated member strength after retrofit to account for uncertainties.

A note of caution is in order regarding the available confinement models: all models listed in the literature have been calibrated against a very large database of tests conducted on axially compressed members [Fig. 9(a)]. Specimens were either reinforced or unreinforced. Tests conducted correspond to the red point in the axial-load moment interaction diagram plotted in Fig. 9(c). The derived stress-strain relationships do not account for the strain gradient effects that occur due to flexural moments. Using stress-strain relationships obtained from axial load tests, in order to model the stress-strain behavior of concrete in the confined compression zone of members under combined axial load and moment (range marked by green in Fig. 9(c)) is a point of inconsistency in the FRP-related literature.

**(b)**

Reference point for confinement

**M**

**N**

**Nbal**

Axial

strain

**(a)**

***cu,c***

Strength reduction by 15%

Range of practical interest

Axial stress

***cc***

**(c)**

***axial***

***lat***

**fcc**

**fcc,u**

*Figure 9: (a) Typical test of FRP confined concrete in compression. (b) Nomenclature for the stress-strain milestone points. (c) Axial-Load Moment interaction diagram of typical prismatic element.*

## 7. Acceptance criteria and safety evaluation

### 7.1 Rotation capacity and displacement ductility of FRP confined members

Based on ample experimental documentation, RC beams, columns and walls, retrofitted with FRP jackets in the critical regions, can develop significant rotation capacity and displacement ductility.

Rotation capacity refers to the maximum angle that may sustained between the chord of the member in the displaced position and the normal to the end cross sections (Fig. 10).

The ultimate chord rotation, *θ­u*, of members strengthened with FRP confinement, may be estimated using one of the following procedures:

(a) From basic mechanics:

*Figure 10: Definition of chord rotation  when the transverse elements (i.e. beams) a) do and b) do not participate in the deformation of the storey.*

**θ**

**θ**

**(b)**

**θ**

**θ**

**(a)**

 (30)

- *el* is equal to 1.5 for primary and 1.0 for secondary members, respectively (according with EN1998-1 (2004) “secondary members” are those members whose stiffness and resistance accounts for less than 15% of the total floor stiffness, and hence they may be neglected in the response analysis, even though they shall be designed to withstand the deformations of the structure under the design seismic loads without loss of vertical load carrying capacity).

- *θy* is the chord rotation attained at yielding of longitudinal tension reinforcement (KANEPE 2013, Biskinis and Fardis 2013):

 (31)

where *h* is the section height,  is the (average) diameter of the longitudinal bars,  and are concrete compressive strength and steel yield longitudinal strength (in MPa), respectively, obtained from in-situ tests of the existing materials. Term *avz* represents the tension shift of the bending moment diagram (EN1992-1-1, 2004).

-*φu* is the ultimate curvature of the end section, evaluated by assigning at the concrete ultimate strain,  the value defined by Eq. (8-27) [or alternatively by Eq. (8-28), or (8-29)].

- *φy* represents the curvature exhibited by the end section at the onset of yielding of tension reinforcement (this may be approximated as *2εsy/h*).

- *ℓpl* is the length of the plastic hinge estimated, according with (EN 1998-3, 2005) from:

 (32a)

or according with Biskinis and Fardis (2013) (for cyclic loads),

 (32b)

(b) Empirically from the following expressions:

The *εcu,c* value determined from Eq. (27) [or Eq. (29)] is used to quantify the curvature ductility by reversing Eq. (9):

 (33)

*θy* may be estimated from:, where *H/h* is the aspect ratio of the member (*H* is the member height and *h* the cross section height of the member).

Using Eq. (8) calculate the available *μθ*:

 and  (34)

(A simplification made here was to assume that *ℓp≈0.5h*, and *H/h≈6*.) The value estimated from Eq. (34) shall be multiplied by 1.5 in order to account for the contribution of reinforcement pullout to the rotation capacity.

(c) Based on calibrated expressions obtained through correlation with an extensive database of tests as follows (Biskinis and Fardis 2013):

(35)

where, *ω=ρs1fsy/fc* is the mechanical reinforcement ratio of the tension reinforcement (including any vertical reinforcement between the tension and the compression chord of the RC section), *ω’=ρs2fsy/fc* is the mechanical reinforcement ratio of compression reinforcement, *Lv* is the shear span (≈0.5*H* for columns), *ρd* is the ratio of diagonal reinforcement (if available in each diagonal direction). The last term *(αρfu/fc)f,eff* of Eq. (35) may be calculated using one of the following expressions, all of which are proposed as alternative options.

 (35a)

 (35b)

 (35c)

where *ρf* is the geometric ratio of the FRP in the direction of loading and *αf* is calculated from Eq. (19) by neglecting the term (1-*ρg*). Alternatives given by Eq. (35a,b,c) account for the pre-damage on *θu*, among which Eq. (35c) better agrees with the experimental database; the first alternative is included in EN1998-3 (2005).

### 7.2 Safety requirements

#### **7.2.1 Ductile members and mechanisms -** **combined bending and axial load**

According with Section §6.2.4, FRP jacketing may increase the effective strength of concrete in compression, through confining action.

A modest increase in the flexural strength, *MRd*, of the FRP-jacketed member may be estimated when accounting for the increased strength *fcc*, of the compression zone. This strength increase corresponds to a commensurate increase of *VEd*, which is used as reference in capacity-based proportioning of the retrofit. If axial load is present, the most conservative estimate for *MRd* shall be obtained in order to assess the available over-strength.

#### **7.2.2 Brittle members and mechanisms** **-** **shear**

The shear strength of FRP jacketed RC members, *VRd*, shall exceed the retrofitted flexural strength, *VEd* = *MRd/Lv* (from Section § 6.1*)* so as to preclude shear failure.

Shear strength, *VRd*, of the retrofitted member comprises contributions of the original member, *VRd,o* and of the FRP jacket, *VRd,f*:

 (36)

The cyclic shear resistance, *VRd,o* of the original member, decreases with the plastic part of ductility demand, expressed in terms of ductility factor of the transverse deflection of the shear span or of the chord rotation at member end: *μθ,pl*= *μθ*-1. For this purpose *μθ,pl* may be calculated as the ratio of the plastic part *θpl* of the total chord rotation, *θu*, normalized to the chord rotation at yielding, *θy*, calculated in accordance with Section §7.1.

The following expression (KAN.EPE 2013) may be used for the shear strength, as controlled by yielding of the embedded stirrups, accounting for the above reduction (with units: MN and meters):

 (37a)

In Eq. (37a), *γel* is equal to 1.15 for primary seismic elements and 1.0 for secondary seismic elements.

Terms *VRd,c* and *VRd,s* represent the contributions of the concrete compression zone and the web reinforcement to the shear strength of the original member (prior to retrofitting with FRP jacketing). Term *VRd,s*, as represented in the established codes of practice, is used in Eq. (37a). Note that the expression corresponds to a 45o angle shear truss. Term *VRd,c* is taken reduced from the code expressions in recognition of the recent understanding that only the compression zone of a cross section participates in shear transfer (Tureyen and Frosch, 2003); thus, the strength contribution is taken,

 (37b)

Term *x=ξd* represents the depth of compression zone at the state of sectional equilibrium at ultimate flexural capacity (accounting for the simultaneous action of the design axial load value for the seismic combination, *NG+0.3Q±E*)*.* In Eq. (37b), *f*c is the concrete compressive strength measured from in-situ tests; for primary seismic elements *f*c should further be divided by the partial confidence factor for concrete in accordance with EN 1998-1 (2004) in §5.2.4. In general, mean material properties from *in-situ* tests and from additional sources of EN 1998-3: 2005 information, should be used in the calculations.

- For primary seismic elements, the mean material strengths in addition to being divided by the appropriate confidence factors based on the Knowledge Level, should also be divided by the partial factors for materials in accordance with EN1998-1 (2004) in §5.2.4.

- If the member has sustained damage during previous loading, the residual, rather than the full contributions of core concrete and web reinforcement shall be considered. The value of *μθ,pl* used in Eq. (37a) to calculate the post-retrofit shear strength of the member will be the minimum of the plastic ductility demand suffered during previous events, and the target value used for redesign.

Term *ρsw-y* was defined in Eq. (16a) as the web reinforcement ratio in direction parallel to the shear force (here it is assumed that the design shear is acting in the *y-* direction of the member’s cross section):

 (37c)

The contribution of the FRP jacket, *VRd,f* is calculated similar to *VRd,s* as follows:

 (38)

The value of *ffwd* depends of the type of the externally applied fiber reinforcement (closed or -shaped, three sided or U-shaped, two sided or ll-shaped, the latter, being the weakest of all alternatives, is usually prohibited by several codes, i.e. ACI440, fib 9.3 Bulletin 35, KAN.EPE 2013), as determined through the pertinent value of *εfd* (see Section §6.2.3). Term *ρf-y* was defined in Eq. (16a) as the FRP jacket reinforcement ratio in direction parallel to the shear force (here it is assumed that the design shear is acting in the *y-*direction of the member’s cross section):

 (39a)

If the FRP reinforcement is applied in strips of a width *bf*, at a longitudinal spacing *sf* (on centers, o.c.) the reinforcement ratio is defined by:

 (39b)

The above equations assume that the fibers of the FRP jacket are placed at an angle of 90o with respect to the longitudinal axis of the member. If the jacket is applied at an angle *αo* with respect to the longitudinal axis of the member, Eq. (38) shall be modified as follows:

 (39c)

The shear strength estimated according with Eq. (36) shall not exceed the following limit value for shear, *VRd,max* which corresponds to crushing of the diagonal compression struts in the web of the member modified to account for the confined concrete strength (EN 1992-1-1, 2004):

 (40)

#### **7.2.3 Brittle members and mechanisms – lap Splices**

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

FRP wrapping over the embedment length of bar anchorages provides clamping, resisting propagation of cover splitting thereby enhancing the frictional mechanism of bond resistance.

FRP jacketing enables attainment of high strain demands in the tension reinforcement at the critical section. The increased demand for bar development capacity cannot always be met by the anchorage/lap splice which is often inadequate in substandard construction or inaccessible to rehabilitation.

The required FRP jacket layers are intended to enhance bond strength in order to develop yielding of the embedded lapped reinforcement at the critical sections near the support.

In existing construction, where the available lap length *Lo* is known, the required bond stress may be evaluated from:

 (41)

Bond strength of lapped bars in the retrofitted member comprises contributions from the concrete cover, the web reinforcement and the added FRP jacket (Tastani and Pantazopoulou 2008):

 (42)

* *Nb* is the number of tension bars (or pairs of tension spliced bars if reinforcement is spliced) laterally restrained by the transverse pressure. For example if in a cross section there are 8 bars evenly distributed around the perimeter (3 bars on each side) then *Nb*=3 (at the most tensioned region of the cross section) whereas if there are 8 pairs of spliced bars around the perimeter then again *Nb*=3.
* c is the concrete cover.
* *Ast* is the area of stirrup legs enclosing the *Nb* lapped bars (the area of legs crossing the splitting plane).
* *s* is the stirrup spacing along the member length. For sparse stirrups the “stirrup term” of Eq. (42) may be neglected for safety.

The effective strain, of the FRP jacket is linked to the degree of acceptable damage along the splice length, which is reflected in the value of the coefficient of friction.

***pcr***

***ft***

**Side splitting**

concrete

***δo***

Compressed concrete due to *ur,o*

***ur,o***

***hr***

***sr***

***δo***

***Hoop strain: =ur/r***

***,o=ur,o/Rb***

***,c= slf=ur,o/(Rb+c)***

***c***

***Db***

***,o***

***slf***

Bar segment

***ur,o=0.5δo***

***Slip δo***

***Efslf***

***ft***

**Face splitting**

***pcr***

**V-shaped splitting**

***ft***

***pcr***

*Figure 11: (a) Radial displacement and surface hoop strain in lap-splice pull-out. (b) Definition of the crack-path length pc (red line).*

**(a)**

**(b)**

Based on *fib* Model Code 2010, bond stress reaches bond strength at a slip value of 0.1 mm. For that limit, damage to the anchorage is negligible, and the corresponding coefficient of friction =1. For higher slip values, the value of  degrades due to plastification or cracking in the lapped length.

Based on experimental results by Tastani and Pantazopoulou (2010), the outwards radial displacement *ur,o*, that occurs at the concrete – bar interface when a bar slips along its axis by an amount *δο*, is related to *δo* by virtue of the inclined profile of the lugs (initially) and by the slope of the sliding plane formed by the crushed concrete under the lugs at higher levels of slip (Fig. 11a):

 (43a)

Hoop strain *εθ,o* at the interface is equal to *ur /r*. For the performance limit considered (i.e., a value of slip *δο*=0.1 mm with=1) the corresponding value of the effective jacket strain is: (Tastani and Pantazopoulou 2008, Pantazopoulou et al. 2015):

 (43b)

where the cover *c* and the bar diameter *Db* are both in mm (Fig. 11a). Equation (43) is valid regardless of the material type (GFRP or CFRP) used in the FRP jacketing. Equation (42) may be used to determine the required confining jacket thickness, *tf* (for securing the lap splice capacity of longitudinal reinforcement). In this case the required jacket thickness over the lap-splice length is estimated from (with *Ab=πDb2/4)*:

 (44)

If the member has sustained damage during previous loading, and the lap splices show signs of distress, then it is advisable to patch repair the damaged cover by replacing it with repairing mortar. If no such repair is possible, then the residual, rather than the full contributions of the cover concrete shall be considered in Eq. (42). In this case, it is sufficient to reduce the concrete term in Eqs. (42) and (44) to 1/3 of its initial, reference value.

Note here that because the *sl f* is very small, usually the calculated number of FRP layers is *n*≥4, thus the effective jacket thickness should be *tf =to·n0.85*. Also in Eq. (44) term *pcr* is used instead of *2c* that appeared in the initial Eq. (42) since the potential splitting mechanisms are modified as shown in Fig. 10b in light of the confining action of the jacket.

Term *pcr* is referred to the length of cracking produced by a single bar or a pair of spliced bars at bond failure (see Fig. 11b). If a V-shaped crack pattern is adopted (see Fig. 11b) then  where *c* the vertical cover. Note that if *Nb⋅pcr>(b-2ch-DbNb)* or *Nb⋅pcr>(b-2ch-2DbNb)* for bars or pairs of spliced bars respectively (*ch* is the side/horizontal cover width), then the critical splitting plane is the horizontal one that crosses all the bars. In this case, the value of *ch+ 0.5⋅(b-2ch-DbNb)/(Nb-1)* or *ch+ 0.5⋅(b-2ch-2DbNb)/(Nb-1)* may be used as *pcr* in Eq. (44) for bars or pairs of spliced bars respectively.

## 8 Detailing provisions to eliminate brittle failure of the jacket

### 8.1 Buckling of longitudinal bars

In lightly reinforced RC members, the compression strain capacity of longitudinal reinforcement is often limited by premature buckling owing to the large unsupported length of the bars.

Bar slenderness ratio of compression reinforcing bars supported laterally by stirrups is defined by the parameter *λ*=*s/Db*. Recommended values of *λ* for high to moderate ductility structures are in the range of 6 to 8.

Values of *λ* greater than 10 are excessive. The bar may undergo elastic buckling prior to yielding. In such cases, the susceptibility of the FRP jacket to stress concentrations limits its effectiveness as lateral support to longitudinal reinforcement after it reaches critical conditions for buckling.

FRP jacketing may delay but cannot preclude eventual buckling of compression reinforcement. The confinement induced by jacketing provides lateral support to the cover concrete, so delamination is not prevented.

The critical buckling load of compression bars diminishes after yielding in compression.

Through FRP confinement, the concrete in the compression zone develops large deformation capacity. So redistribution is possible from the longitudinal reinforcement to concrete when the former reaches conditions of instability.

Two alternative options are considered in order to calculate the required FRP confinement to **(a)** eliminate the occurrence of buckling, or **(b)** to increase the deformation capacity of reinforcement in the compression zone of concrete members.

**(a)** Jacket thickness shall be evaluated from the requirement in reinforced concrete design (Priestley at al. 1996) according to which, the restraint needed to avoid buckling over a critical length that involves several hoops, of a longitudinal bar in the strain hardening range of axial compression, is given by the volumetric ratio of transverse reinforcement as follows:

 (45a)

* *n* is the total number of compressed longitudinal bars restrained by the jacket(e.g. all the bars in the compression zone of a member cross section),
* *Et* the modulus of elasticity of transverse reinforcement,
* *Er* is the double modulus of the longitudinal reinforcement at the onset of bar buckling at axial compressive stress in the bar equal to *fs* (where *fs>fsy*), given by Eq. (45b); here *Es* and *Ei* are the elastic and the secant (*fs* to *fu*, see Fig. 12b) moduli of existing steel compression bars after yielding, respectively. The double modulus is intended to account for the fact that upon nonlinear bar bending outwards due to buckling, a part of the bar cross section is unloading from compression into tension:

 (45b)

Equation (45a) is used also to consider the restraining action by FRP jacketing; the required jacket thickness *tf*, may be obtained by setting the left hand side of Eq. (45a) equal to the product of the volumetric ratio of FRP jacket by the confinement effectiveness coefficient: *af* ⋅*ρfv= af* ⋅*2tf (b+h)/(bh)* (in the case of square member cross section it is: *af* ⋅*ρfv= af* ⋅*4tf /h*). Assuming as critical condition the onset of longitudinal bar yielding (*fs=fsy*), Eq. (45a) is then solved for *tf* (Triantafillou 2004):

*Figure 12: a) Symmetric buckling of fully supported steel bar, b) stress – strain diagram*

***s***

***w***

**FRP jacket**

*w*

*w*

*R*



*bs*

**(a)**

**(b)**

**Bas stress,  *fs***

**Strain, *εs***

***εsy***

***εh***

***fsy***

***fu***

***Es***

***εs,crit***

***fs,crit***

***Et***

***Ehο***

***εu***

***Ei***

For square cross section:  (45c)

For orthogonal cross section:  (45d)

Thus Equations (45c,d) estimate the required jacket thickness to secure that buckling of the longitudinal compressed reinforcement will be avoided up to the yielding strain (and not up to a specific value of strain into the hardening range).

**(b)** In plastic hinge regions sideways buckling is the usual form of compression reinforcement failure due to lateral shear distortion of the member in that region. A criterion for design of the required lateral restraint to be provided by the jacket is the requirement that the strain capacity of confined concrete, *εcu,c* shall exceed the critical strain, *εs,crit*, at the onset of reinforcement buckling. In this case (where *εcu,c*>*εs,crit*), redistribution between the compressed bars at incipient buckling and the encased concrete is possible, thereby postponing buckling to occur at a higher strain level (Tastani et al. 2006, Tastani et al 2010).

The critical *s/Db* ratio that corresponds to rebar critical stress *fs,crit* is given by,

 (46)

* *Et* is the tangent modulus of steel at the stress level considered (see Fig. 12b and fib Bulletin 24, 2003),
* *ψ* parameter that accounts for the buckling length (*ψ*=/4 for symmetric buckling and *ψ*=/2 for sideways buckling, see Fig. 13).

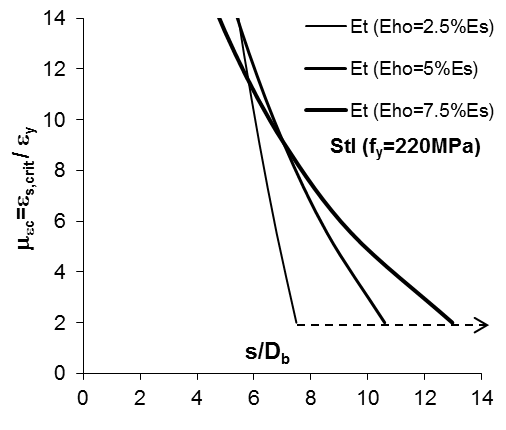
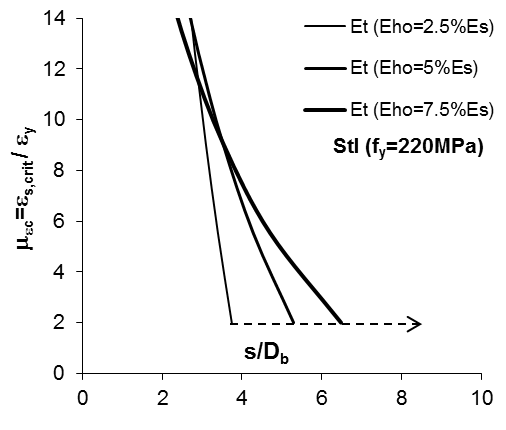
Given the full stress-strain law of the longitudinal bars in compression (which is often assumed identical to that in tension for lack of detailed data) the limiting strain ductility *μεc*=*εs,crit /εsy* is plotted against the *s/Db* ratio (see for example, Fig. 13). Parameter *εs,crit* is the strain at which the bar will become unstable. Therefore, buckling of any individual bar segment is controlled by its strain ductility *μεc*- *s/Db* curve, unless the dependable deformation capacity of encased concrete, ε*cu,c* (as defined by any preferred confinement model – for example, Eq. (27)) exceeds the *εs,crit* value corresponding to the available *s/Db* ratio.

An important consideration in detailing the FRP jacket is to ensure that the target displacement ductility of the member after upgrading *μΔ,req* may be attained prior to buckling of primary reinforcement. The steps to achieve this are as follows:

* Estimate the target displacement ductility demand at the design performance limit state *θreq*=*θu,target/θy*
* Estimate the curvature ductility demand *μφ,req* (where *μφ* =*φu*/*φy*) in the plastic hinge region of the member, using the relationship between *μθ,* and *μφ* from Eq. (8) and (9):

 (47)

* From *req* find the compression strain ductility demand, *μεc,req* of compression reinforcement: . Estimate the required jacket confinement, to ensure , *cu,c* from Eq. (27). [[12]](#footnote-12)

Et (Eho=2.5%Es)

Et (Eho=5%Es)

Et (Eho=7.5%Es)

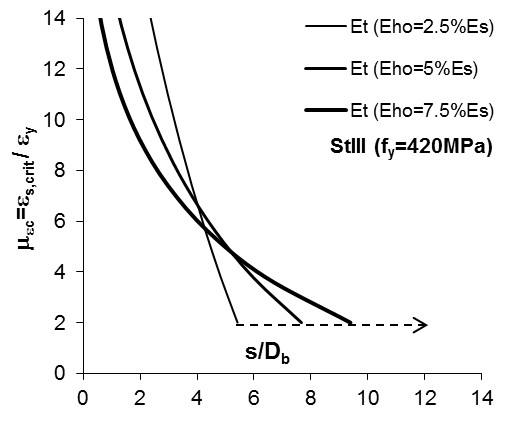
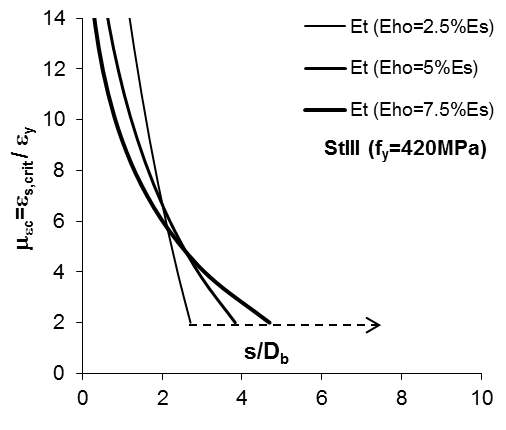
Et (Eho=2.5%Es)

Et (Eho=5%Es)

Et (Eho=7.5%Es)

**s**

**s**

Et (Eho=2.5%Es)

Et (Eho=5%Es)

Et (Eho=7.5%Es)

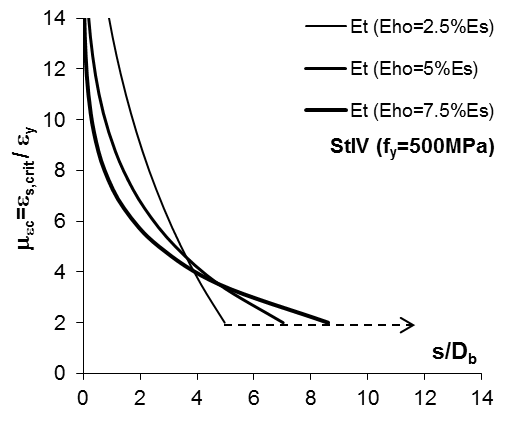
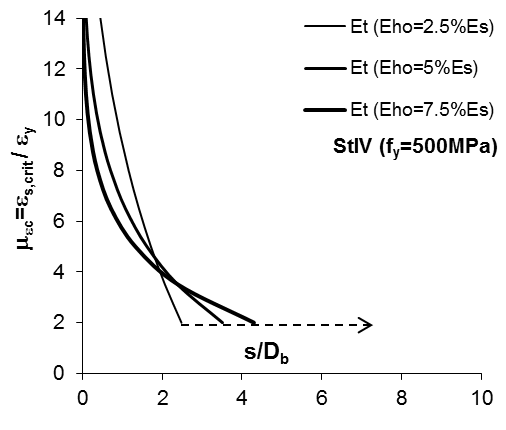
Et (Eho=2.5%Es)

Et (Eho=5%Es)

Et (Eho=7.5%Es)

**s**

**s**

Et (Eho=2.5%Es)

Et (Eho=5%Es)

Et (Eho=7.5%Es)

Et (Eho=2.5%Es)

Et (Eho=5%Es)

Et (Eho=7.5%Es)

**s**

**s**

*Figure 13: Compressive strain ductility c =s,crit/y versus stirrup spacing s/Db for steel categories StI, StIII and StIV and two buckling lengths (buckling curves).*

### 8.2 Displacement ductility of FRP jacketed r.c. members

Through FRP jacketing all failure modes but flexural yielding of reinforcement may be suppressed. The available displacement ductility, * Δ* as a function of transverse confining pressure *lat*is estimated from (Tastani and Pantazopoulou 2008):

 (48a)

In Equation (48a) the lower limit value of **=1.3 recognizes that lightly reinforced r.c. elements that overcome any premature elastic mode of failure, are able to develop some limited displacement ductility.

The above is simplified by neglecting the contribution of stirrups (if their arrangement is deemed non-conforming to modern standards):

 (48b)

where *f,eff=fu,d* and *fu,d=fu*/*f,* and *f* the FRP material redundancy coefficient.

By recalling here the expressions for the chord rotation at yielding (Eq. (31)) and for the plastic component of drift capacity (Eq. (35)) by Biskinis and Fardis (2013) where *θu= θy+ θupl* the displacement ductility related to the confinement provided by the FRP jacketing is defined by Eq. (48) below; if the displacement ductility demand is known, the equation below may be used to extract the required jacket thickness (implicit in the exponent of the numeral factor 25) through iteration:

 (48c)

## 9. Joints

Beam-columns joints are regions of very high shear stress demand. The design shear force which is input to the beam-column joint during seismic excitation may be estimated from the moment reversal which occurs between the end faces of the joint region, as the slope of the moment diagram over the depth of the beam or column (Fig. 14a).

Joint failure occurs due to inadequate shear reinforcement, or by crushing failure of the diagonal compressive strut that forms in the body of the joint (Fig. 14b).

Reinforced concrete joints in beam-columns connections can be effectively strengthened with pertinent arrangement of FRP externally bonded strips. Because of the shear action effects which cause diagonal tension failure within the joint panel, shear reinforcement in the form of FRP strips is an appropriate method of retrofit. In this case, externally bonded FRP reinforcement either confining the joint on all free faces, or placed with the fibers running in the direction of principal tensile stresses is needed. To determine the required amount, the jacket thickness *tf*may be estimated from the following expression:

-If the FRP fibers are oriented in the horizontal direction, then, *tf = tf,h*:

 (49a)

-If the FRP fibers are oriented in the vertical direction, then, *tf = tf,v* :

 (49b)

With *Rd* equal to 1.5.

In Eq. (49), *Vj,h* and *Vj,v* are the design shear forces in the joint, estimated to act on a horizontal, and a vertical plane through the joint, respectively. Parameter is the allowable design value of FRP tensile strain which, for the case considered, shall not be taken higher than *4‰*. An essential requirement is proper anchorage of the FRP strips. When FRP reinforcement is not properly anchored, FRP strengthening shall not be considered effective.

For calculating the design values of *Vj,h* and *Vj,v* the following procedure is followed with reference to Fig. 12a: First the sums of yield moments in the beams and in the columns framing into the joint in consideration are calculated: Here, *ΣΜyb* is the sum of yield moments of the beams that frame into the joint, *ΣΜyc* is the sum of yield moments of the columns that frame into the joint.

-If *ΣΜyb< ΣΜyc*, then the horizontal shear force *Vjh* is derived from the slope of the column moment diagram as follows:

 (50)

while the vertical shear force acting in the joint, (*Vjv*), is obtained from:

 (51)

If *ΣΜyc< ΣΜyb* then the vertical shear force *Vjv* is derived by,

 (52)

while the horizontal shear force (*Vjh*) is obtained from:

 (53)

In the above equations, *jdb* is the internal lever arm of the beam section and *jdc* the internal lever arm of columns; *Vg+ψq,b,r, Vg+ψq,b,ℓ* are the shear forces of the beams to the right (*r*) and to the left (*ℓ*) of the joint due to vertical loads that act at the same time with the seismic action. *Lb,n* and *Lb* are the theoretical and clear half span of the beams; *Hn* and *H* are the theoretical and clear storey heights.

**hc**

**Point of zero moment**

**Lb**

**H/2**

**db**

**hb**

***C1* = *Ts1***

***Vcol***

***Vcol***

***Vb1***

***Vb2***

***Ts2=1.25Asfy***

***C2* = *Ts2***

***Ts1=1.25Asfy***

1. **(b)**

*Figure 14: (a) Calculation of joint shear force, Vj from the gradient of flexural moments along the column or the beam line in the joint region. (b) Diagonal strut and definition of confinement requirements.*

**10. Conclusions**

A performance-based framework for design of retrofits of R.C. Buildings using FRP materials was developed and presented in detail. Consistent models and approaches were weaved together to cover the entire range of design considerations, including global stiffness requirements, strength hierarchies to satisfy capacity-design objectives in the retrofitted structure, and deformation capacities of the individual structural members to meet the performance objectives of the retrofit. Interestingly it is shown that all performance indices may be linked to measures of the lateral confining stress exerted by FRP jackets on the encased members; however, the supporting database of experiments and attendant calibrated confinement models are particularly biased, being obtained solely from uniaxial confinement tests with or without embedded reinforcement. It was found that information is scarce regarding the performance and deformation capacity of members retrofitted by FRPs when these are subjected cyclic moment-shear-axial load reversals, the result being some over-conservatism in defining design values for these parameters. Thus, rotation capacity, improved anchorage of confined reinforcement, and shear strength of retrofitted structural members are all subjects that warrant further investigation; detailing the anchorage of FRP strips and jackets in retrofits of beam-column joints is another open issue which, although addressed analytically and with design expressions in the present work, would require particular attention during implementation in order to secure efficient confinement of the diagonal compressive struts that support the function of moment and shear transfer in this type of elements.

**Appendix**

(a) Reinforced concrete jacketing (Thermou and Pantazopoulou 2014): The storey stiffness contribution owing to a total of *ℓcRC* RC jacketed columns is equal to:

 (A-1)

where *Ec* is the elastic modulus of concrete, *Af* is the floor area, *Hst,I* is the storey height, *hJ,ave­* is the average height of the RC jacketed cross section and *ρc,*i is the column’s area ratio in the floor plan at i-th storey.

(b) Addition of RC walls: The storey stiffness owing to *ℓw* walls oriented in the direction of the earthquake, is expressed by:

 (A-2)

Where *Ec* is the elastic modulus of concrete, *Af* is the floor area, *Hst,i­* is the storey height, *lw,ave­* is the average length of RC walls, and *ρwc,i* is thedimensionless area ratio of RC walls at the i-*th* storey.

(c) Metallic cross braces: The stiffness increase of a frame bay with an encased pair of metallic cross braces along its diagonals may be estimated by:

 (A-3)

where, *Dj* is the length of the brace diagonals, *Es* is the elastic modulus of the steel braces, *Abr,j* is the area of the braces and *φ* (=*atan(Hst/L*)) is the angle forming between the brace and the x-axis (*L* is the clear span and *Hst* the clear height).

(d) Infill masonry walls: The stiffness provided by the masonry wall is:

 (A-4)

where *Af* is the floor area, *Hst* *­*is the storey height, *fbc* and *fmc* is the compressive strength of the bricks and the mortar, respectively, *μymw* is the drift ductility of the masonry walls, *θymw* is the drift of masonry walls at yield, and *ρmw,i* is the dimensionless area of masonry infill walls at i-th storey.

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**Notation**

**(i)-Latin symbols**

*Ast=*area of transverse reinforcement legs crossing the splitting plane

*D*= diameter of cylindrical specimen

*EIj*=effective sectional stiffness of the j-th member (secant at yielding).

*Ei*=inelastic, tangent modulus of steel reinforcement after yielding

*Er*=Reduced, or double modulus

Ef= elastic modulus of FRP

*Htot=*total building height measured from the crest of a box-type basement or otherwise, from the level of foundation

*Hj* = member length (j corresponds to the j-*th* member)

*Hn*, and *H* = theoretical and clear column lengths (in the context of beam column joints)

*K1*=stiffness of the first floor

*Keff* =Generalized lateral stiffness of the structure (stiffness of the equivalent single degree of freedom system)

*Ki*=the translational stiffness of the i-th floor

*LV*= Shear Span of a frame member (*LV=M/V*; in case of fixed end conditions of a column under lateral sway, *LV=H/2*)

*lbmin* =development length of the wrap

*lo*=lap-splice length of bar-pair

*Lb,n* and *Lb* = theoretical and clear half span of the beams;

*MRd,o*=Flexural strength prior to retrofit (capacity)

*MEd=*Design value of required flexural strength (demand)

*ΔΜEd* =Required flexural strength increase through retrofitting (demand)

*MRd*= retrofitted flexural strength (capacity)

*My,j*=Yield moment of the j-th member

*Nb*=number of lapped bar pairs favored by the confining pressure that is exerted on the splitting plane

*NG+0.3Q±E* = Column axial load for the seismic design combination according with EC8-I (2004) (common residential buildings)

*R*=Corner chamfering radius

*S=1.2* (soil class according to EC8-I 2005)

*T=*structural period

*Teff* = the effective translational period (based on secant to yielding sections analysis).

*Tref* = the empirical reference value prescribed by EN1998-I (2005), as given here by Eq. 8-2.

*Ttrg* = the target or improved period of the retrofitted structure.

*TB, TC, TD* = milestone period values corresponding to the Type-I earthquake hazard, the values of which correspond to the characteristic points (see EN1998-I 2005).

*V=*Base Shear

*VEd*= Shear force required to cause flexural yielding of the member at the end (see static model in Fig. 8-5; referred to as flexural shear demand=*MRd/Lv)*.

*VRd* =Retrofitted shear strength

*VRd,o* =Member shear strength prior to retrofit

*VRd,c* =Concrete contribution term to shear strength

*VRd,s* =Web reinforcement contribution to shear strength

*VRd,f* =Contribution of web FRP (transverse jacket) to shear strength of the member

*VRd,max* =Shear strength that corresponds to crushing of the diagonal compression struts

*Vj,v* and *Vj,h* = Vertical and horizontal joint shear force

*ag=*Peak ground acceleration

*av*=coefficient to account for the tension shift of the bending moment diagram due to shear (1 or 0)

*bf*=width of transverse strips

*b,h* = width and height of rectangular cross-section - prismatic member; subscript c refers to columns, whereas subscript b refers to beams (in beam column joints)

*bo, ho*=dimensions of confined core

*b’, h’*=straight sides of rectangular cross-section after corner chamfering

*d*=effective depth of cross section (measured from the extreme compressive fiber to the centroid of tension reinforcement)

*d2*=effective cover (to the centroid) of compression reinforcement

*c*=clear cover thickness

*fj,eff* = the effective jacket stress

*fctk0.05*=characteristic tensile strength

*hst*=storey height (from top of slab to top of slab in the next floor)

*ki*=floor stiffness ratios

*ℓcRC* = the number of columns retrofitted with RC jackets in a single floor;

*ℓw* = the number of walls added for stiffening the structure in the direction of action;

*ℓX* = the number of X-brace pairs added in the floor to add stiffness – in the direction of action;

*ℓmw* = the number of infill panels added in the floor in the direction of action;

*ℓcf* = the number of columns strengthened with longitudinal FRP strips (externally bonded or NSM).*n*=total number of floors

*ℓpl*=the length of plastic hinge

*n*=in the context of jacket thickness: number of layers of the FRP jacket,

*n*=in the context of total height of the structure, *n* is the number of storeys

*pc*=length of splitting crack path

*q=*behavior factor

*z*=web-height of the equivalent truss resisting shear in reinforced concrete elements (in the context of shear resistance)

*fco*= unconfined concrete strength

*fsy, εsy*= yield strength and associated yield strain of longitudinal reinforcement

*ffu*= ultimate strength of FRP

*fcc*: ultimate strength of confined specimen

*fyk*=characteristic steel yield strength

*fy,st*= yield strength of transverse reinforcement

*sao* =slip of the adhesive at shear failure

*s*=spacing of stirrups

*sf*=c.o.c. spacing of FRP strips

*tf* =thickness of the FRP jacket;

*to*=thickness of a single layer

*tf,v* and *tf,h* = jacket thickness in the vertical and horizontal directions

*wcr*=crack width at the limit of serviceability

*z*=the height coordinate measured from the same reference point as Htot (in the context of shape function and lateral deflection calculations)

*x=*depth of compression zone

***(ii)-Greek Symbols***

*Φi* = the coordinate in the i-th floor of the fundamental translational shape of building response, normalized to its peak value;

*ΔΦi* = the change in Φi between two successive floors.

*αf*=confining effectiveness coefficient of the FRP jacket

*αw*=confining effectiveness coefficient of the stirrups

*γf=*redundancy safety factor (for FRP materials)

*γfb=*concrete substrate safety factor (for open-FRP layers)

*γFRP=*1 and 2 for CFRP and GFRP respectively according with the KANEPE (2013) model

*γel*=safety factor for chord rotation capacity

*γRd*= overstrength coefficient for joint shear

*εf=*FRP strain at the cross section examined

=effective confining strain used in jacket design *f,eff=fu,d* and *fu,d=fu*/*f,* and *f* the FRP material redundancy coefficient)

*εs,crit*=critical strain (onset of buckling) of compressed reinforcement

*εcc*=confined specimen’s axial strain at peak stress

*εlat,ccu*= confined specimen’s lateral strain at 85% capacity

*εfu,d*= design ultimate strain of FRP

*εc,u* = strain capacity (at 85% residual post-peak axial compressive strength) of unconfined concrete (in the range from 0.003 to 0.004),

*εs1,min=*minimum required strain in tension reinforcement

*εfu*= nominal ultimate strain capacity of the FRP (characteristic value)

*εf,max*= *εfu*/γf the maximum usable strain of the FRP

*εfd*= design strain of the FRP

*εcu,c* = Post-peak “strain capacity”, is the axial strain sustained by the compressed specimen at a residual axial compressive strength equal to 85% of peak.

*ζ*=a coefficient to account for the reduced confining effectiveness of the jacket at very high axial compression strain values

*θdem* =the total average elastic drift demand ratio for the retrofitted structure

*θy=*chord rotation required to yield the column at its critical section

*θu*=ultimate chord rotation capacity

=plastic rotation capacity

*η=*damping factor related to the damping coefficient *ξ*

*λ*=reinforcing bar slenderness ratio

*μεc* = compression bar strain ductility

*μΔ, μθ =* displacement or rotational ductility (demand or supply depending on context)

*μφ=*curvature ductility

*μθ,pl*=plastic rotation ductility (=*μθ-1*)

*μRd ; μEd*=normalized moment ratio: capacity, and demand: = (*MRd/(bd2fcd*); *MEd/(bd2fcd*), respectively

*νd,max=* maximum value of the column axial load ratio for the seismic design combination *(Ed* =*NG+0.3Q/(fcdbd)*)

*ξ*=normalized depth of compression zone (=*x/d*)

*ξbal*=normalized depth of compression zone at balanced failure

*ρs1*=tension reinforcement ratio (=*As1/bd*)

*ρs2*=compression reinforcement ratio (=*As2/bd*)

*ρd*=diagonal reinforcement ratio (=*Ad/bd*)

*ρf*=area ratio of FRP tension reinforcement (=*Af/bd*)

*ρg*=total longitudinal reinforcing ratio

*ρfv*=volumetric ratio of FRP transverse reinforcement (for orthogonal cross section *ρfv=2tf (b+h)/(bh*), for circular *ρfv=4tf /D*)

*ρsv*=volumetric ratio of embedded stirrup reinforcement (=(*ρsw-x+ρsw-y*)/2)

*ρsw-x, ρsw-y*=web reinforcement ratios in the x, and the y directions, respectively (stirrups)

*ρfw-x, ρfw-y* =web reinforcement ratios in the x, and the y directions, respectively (FRP jacketing)

*σy*= average confining pressure, acting in the y-direction

*σx*=average confining pressure, acting in the x-direction

*σlat*=average lateral (confining) pressure

*a* =the shear strength of the adhesive at the stage of plastification

*b1* =the design bond strength

=ultimate curvature

=the curvature exhibited by the end section when the steel is yielded

*ωwd* =the mechanical ratio of confining stirrup reinforcement (=*ρvfy,st/fc*)

*ω* =the mechanical ratio of tension reinforcement

*ω’*=the mechanical ratio of tension reinforcement

1. Shear strength is supposed to always exceed flexural strength, through proper design measures. The response ought to be controlled by flexural strength which is also analogous to stiffness, because of the advantages of ductility secured by yielding of primary reinforcement. Because flexural strength depends mainly on the yield strength of longitudinal reinforcement and on the axial load acting on the member, evidently it cannot be increased through jacketing. If shear strength is inadequate, a design objective in using FRP jackets for retrofit would be to suppress premature shear failure that could prevent the longitudinal reinforcement from developing its full yield capacity. Such a measure would ensure that flexural strength will always control failure after retrofit through jacketing, and thus, in such a retrofit scenario, the demand side of the design equation will only depend on the available longitudinal reinforcement). [↑](#footnote-ref-1)
2. Note: Definition of some terms used for structural engineers and not exclusively related with FRP can be found in the notation section at the end of the paper. [↑](#footnote-ref-2)
3. Example: The effective translational period of a 5 storey structure has been determined as *Teff=*1.1s. The seismic hazard is defined by *S=*1.2 (soil class B), *TB=*0.15s, *TC=*0.5s, *TD=*2s, *ag=*0.2*g*, *η=*1 (**5%), *o=*2.5, *H=*15m. Based on Eq. (2) it is estimated that *Tref=*0.572s. Thus, global interventions are needed to reduce the period value from 1.1s to a value *Ttrg* closer to 0.572s. For a preset drift ratio limit of 1.25% and a ductility of *μθ*=2.5, it follows that the corresponding elastic drift limit would be *μθ=*1.25%*/θdem* thus  *θdem=*1.25%/2.5*=*0.50% and the required value of *Ttrg*:

   This may be considered the maximum acceptable value for the retrofitted structure, which will be designed with a behaviour factor *q=μθ=*2.5 and will therefore develop significant, but repairable damage in the design earthquake. Selecting lower target values for *Ttrg* will generally lead to less damage and better overall performance. Note that for a given structure, the lower the period, the greater the fraction of deformation demand that will be developed in the beams rather than the columns, therefore, it is a good practice to aim for a lower value of the period, as close as possible to *Tref*; but the downside of this choice is increased rehabilitation cost. [↑](#footnote-ref-3)
4. Example: For a 5-storey building with *hst*=3.5m storey height (*Htot*=17.5m) and the selection of triangular response shape a retrofit scenario could be as follows: The target period is equal to 0.64s according to Eq. (2). It follows that the required first floor stiffness is *K1/m*=1446kN/m according to Fig. 4a (see the red dashed lines). The required storey stiffnesses should be decreasing towards the upper floors according with the ratios: *K2*=0.93*K1*, *K3*=0.80*K1*, *K4*=0.60*K1*, *K5*=0.33*K1* (see Fig. 4d, follow the red dashed lines) where *K1* is the first floor stiffness. Thus, through this very simple approach, the required stiffnesses to achieve the desired pattern of drift distribution and the structural period in the retrofit are completely defined. After implementation, the success of the retrofit design in approaching toward the chosen lateral shape response and target period may be evaluated through assessment. [↑](#footnote-ref-4)
5. The curvature at yielding of a linear RC member is approximately equal to  where *h* the cross section depth and *sy* the characteristic yield-strain of the reinforcing steel. Thus the chord rotation (see definition in § 4.2.1)at yielding, is, , where *H/h* is the aspect ratio of the member (*H* is the member clear height and *h* the cross section height of the member). Therefore, two members having very different aspect ratios, yield at very different relative drift ratios. [↑](#footnote-ref-5)
6. Using the calculated compression strain demands, the required amount of confining reinforcement may be obtained from pertinent stress-strain models for FRP-confined concrete, which relate the thickness of the FRP jacket with the compression strain capacity of encased concrete. [↑](#footnote-ref-6)
7. For *fyk*=500MPa, ; for *fyk*=400MPa, 

   for *fyk*=220MPa, ) [↑](#footnote-ref-7)
8. The shaded part indicates concrete crushing, for embedded tensile reinforcement with *fsy* = 500 MPa (B500C); values are increasingly conservative in the range of higher FRP strains, i.e., for lower ** values, to account for the fact that the actual stress in compression reinforcement is lower than the assumed yielding limit. More strict criteria (e.g. a minimum required value of tensile strain in the extreme layer of reinforcement – *εs1,min*=0.004) will extend the shaded portion of the table downwards. [↑](#footnote-ref-8)
9. The jacket layers are calculated as follows: From *tf* *=b⋅f-y*/2 calculate *n=tf /to*, if *n*<4 then the calculated number of layers is applied, else if *n*>4 then recalculate the increased number of layers by applying *n=[tf/to]1/0.85*. As the number of layers is increased, the effective strain of the exterior layers is reduced due to the increased stiffness of the jacket – therefore the choice of alternative strengthening schemes that make a better use of material resources ought to be considered. [↑](#footnote-ref-9)
10. The usable design FRP strain is limited in order to protect the retrofit from premature local failures such as:

    (a.1) Rupture of the FRP at the corners.This mode of failure occurs mostly due to lateral dilation of concrete under high compression strains in the compression zone of confined members. To delay the occurrence of local rupture due to high compressive pressures the corners of the cross section should be chamfered by a radius of *R*, according with the requirements of Section §5.2.3.

    (a.2) Rupture may also occur due to buckling of embedded compression reinforcement. The axial compression strain that is allowed to occur in the compression zone of the member at the ultimate limit state shall be limited according with Section §7.1.1.

    (b.1) Debonding failure of the FRP in a closed jacket arrangement ( - type). The most critical for debonding is the external layer, since the shear strength of the adhesive in interior layers is enhanced by friction due to confinement. The minimum required overlap length of the exterior jacket layer, *lbmin* is:

     (23)

    where *τ* the shear strength of the adhesive at the stage of plastification and so the slip of the adhesive at brittle shear failure (data for the adhesive are needed by the adhesive provider). For an adhesive that exhibits ductile shear response up to su, the coefficient 1.6 may be eliminated and su be used in lieu of so in (23).

    (b.2) Debonding failure of the FRP in an open (U – type) FRP jacket arrangement (i.e., in case of anchorage on brittle substrate such as the concrete cover). The minimum development length measured from the critical section where ε*fd* will be developed – at the point where the FRP intersects a flexural or shear crack of width *wcr* – is:

     (24)

    The design bond strength is*, b1* = *fctk0.05* /*γfb*, where *fctk0.05* is the characteristic tensile strength of the concrete substrate and *γfb* =1.5 the concrete material safety factor. Design calculations may be performed for *wcr=*0.5 mm. The jacket effective thickness *tf* is defined in Section §5.2.1. [↑](#footnote-ref-10)
11. Three additional alternative stress-strain models are also relevant and might be considered in a final design guideline from among the many confinement models available in the literature:

    (a) The model adopted by EN 1998-3 (2005): Here the confined concrete strength and the associated strain (*εcc*) as well as the strain capacity (*εcu,c*), where ε*f,eff* is lower than the jacket strain at rupture (*εfu* – suggested values are 0.015 for CFRP and AFRP, and 0.02 for GFRP):

     (28a)

    where, for circular section *eff=*1, for rectangular and for strips , where *sf* is the strip spacing (o.c.) and *D* the maximum cross sectional dimension.

    (b) Based on correlation of a large database of tests Biskinis and Fardis (2013) have proposed a revision of Eq. (28a) as follows:

    (28b)

    where, *ρf* is the geometric ratio of the FRP in the direction of loading (i.e. *ρf=2tf/D*, see Section §5.2.1) *εu,f* is the failure strain of the FRP, and *keff* an FRP effectiveness factor equal to 0.6 for CFRP, AFRP or GFRP; *εco=*0.002.

    (c) *KAN.EPE 2013 (Greek code for Retrofitting, in English, GRECO 2014):* the confined concrete strength and the associated strain (*εcc*) are obtained from:

    (29)

    *fc,d= fck/*1.5, *fcc,d* is the design confined concrete compressive strength, *γFRP*=1 and 2 for CFRP and GFRP respectively, whereas *αf* is the coefficient of confinement effectiveness [see Eq. (19)], and *ωwd* is the mechanical ratio of confining reinforcement (*tf* is the jacket thickness according with Section §5.2.1, and *d* the size of the cross section for continuous jackets), *fj,eff* is the effective jacket stress which is taken equal to the nominal strength of the jacket material (i.e., *εfd=εf,max*).

    [↑](#footnote-ref-11)
12. Example: Consider a square RC cross section of *h*=300 mm where four compressed longitudinal bars are restrained by the jacket(*n*=4). The axial load ratio is *vd,max*=0.4. The FRP jacketing system (here GFRP) has *Ef*=75 GPa, design strain *εfd*=0.02/*γf* (*γf* =1.5) and layer thickness *to*=0.16 mm, whereas the confinement effectiveness is *f*=0.53.

    Option (a) At critical buckling condition where onset of longitudinal bar yielding occurs the implementation of Eq. (45a) results in *tf*=0.30 mm (this corresponds to two plies).

    Option (b) By requiring that the RC element can develop a displacement and drift ductility ratio, *μΔ*=2, the curvature ductility demand is estimated from Eq. (47) as *μφ,req*=3 and the corresponding concrete compression strain is . Because the strain demand in the level of compression reinforcement, *εs2*, is *εs2*<*εcu,creq* but also *εs2>εsy* (depending on the values of *d2* and *x* variables of the cross section for *μφ,req*=3, see Fig. 6.1), the compression reinforcement would be required to yield in compression and to be able to sustain post-yield strains in the strain hardening branch). For stirrup arrangement representative of older practice (i.e. for *s/Db*=10 with StIII, see Fig. 13, diagram at right) the value of *εs,crit ≤εsh*, thus *ec*<2, leading to buckling upon zero bar stiffness or even worse to elastic buckling of longitudinal reinforcement. To increase the deformation capacity of compression reinforcement beyond the limit of zero stiffness buckling or elastic buckling (by raising the strain capacity from the limit value *εs,crit* to the demand value ), Eq. (27) is used by substituting where  (assuming values for all other parameters as: *fck*=20MPa, *εc,u*=0.0035, and *αf*=0.53, *fd*=*n1⋅n2⋅n3⋅fu /f =*0.69*⋅*1*⋅*1*⋅*0.02/1.5=0.0092, *R*=25mm, *Db*=16mm). The resulting required volumetric ratio for the FRP is *ρfv*=0.0067, which corresponds to *tf* =0.505 mm (i.e. four plies, each ply of 0.16 mm thickness). If the value of  was chosen slightly higher than *εsy* (i.e. 0.0025) so that, at the level of the compression bars it would be *εs2=εsy* (critical condition for option a) then the required jacket thickness is calculated as *tf* =0.35mm (i.e. three plies).

    Comparing the two options at the same critical reinforcement strain, namely compression yielding of the longitudinal bars, option b) is deemed more conservative. Also option b) will secure the strain capacity of the reinforcement deeper into the hardening range where the critical conditions for buckling may occur (thus postponing the occurrence of buckling up to or beyond the exhaustion of the strain capacity of confined concrete). [↑](#footnote-ref-12)