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Background to the European seismic design provisions for retrofitting RC elements using FRP materials

This paper is a comprehensive background document on the state of the art in European seismic design provisions which was assembled by fib committee 5.1 to support the development of design guidelines regarding the use of externally applied fibre reinforced polymer (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 following critical evaluation of the existing proposals and with careful reference to the experimental evidence available, the comparative assessment of past models in the literature and requirements established from first principles.

Keywords: seismic retrofit, FRPs, performance-based design, rehabilitation

1 Introduction

Fibre reinforced polymers (FRP) were introduced into civil engineering practice in the early 1990s, but they only became popular after they became known for their effectiveness as a fast remedy when retrofitting damaged reinforced concrete and masonry structures in the wake of the catastrophic earthquakes at the end of that decade (Northridge, 1994, Kocaeli, 1999, Athens, 1999). Since that time, extensive research has been undertaken to support design procedures for retrofits with FRP wraps and laminates, leading to several versions of design guidelines (ACI 440.2R-08 [1], *fib* Bulletins 14 [2], 35 [3], etc.). A large part of the research effort was directed towards the 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 address global structural response issues as well 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 behaviour, thus complicating the detailing process.

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

2 Global considerations

Structures are 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 frequently occur 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 to illustrate that externally applied FRPs, when used as confining jackets on lightly reinforced concrete members, are effective measures for increasing ductility and deformation capacity by suppressing the 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 through the mere addition of transverse FRP jackets. Strength and stiffness both depend on the amount and arrangement of primary (i.e. longitudinal) reinforcement. (Shear strength is always supposed to 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 the yielding of primary reinforcement. As flexural strength depends mainly on the yield strength of the longitudinal reinforcement and the axial load acting on the member, evidently, it cannot be increased through jacketing. If shear strength is inadequate, a design objective when using FRP jackets for retrofits 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 a jacketing retrofit. Therefore, in such a retrofit scenario, the demand side of the design equation will only depend on the longitudinal reinforcement available.) 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 with FRP jacketing is considered to be a **local** intervention in the seismic rehabilitation of RC structures.

It should be noted that FRP strips or laminates may be added as longitudinal reinforcement along the member length – most effectively when used as NSM reinforcement ([4–14]). As the strength and stiffness of the member is increased by the addition of the longitudinal FRP reinforcement, thus 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 FRP strips added should be anchored adequately beyond the critical cross-sections where maximum flexural moments (i.e. demand for force development in the added reinforcement) are expected to occur. Such sections are, for example, at the ends of beams and columns and the bases 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 the additional longitudinal reinforcement and also to support their anchorage.

If on first assessment it is deemed necessary to moderate the demand, too, the retrofit solution should include **global** measures aimed at increasing the effective stiffness of the structure K_{eff}. Note that by increasing K_{eff}, 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 ensures that the magnitude of deformation demanded of individual members is lowered[1].

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 ([15]). The procedure practically evaluates the strain energy stored in the system during this vibration through the following expression (i.e. work-equivalent stiffness):

[1] Note: Definitions of some terms used by structural engineers and not exclusively related to FRP can be found in the notation section at the end of this paper.

$$K_{eff} = \sum_{i=1}^{n} K_i \cdot \Delta\Phi_i^2 \tag{1}$$

where K_i is the lateral stiffness of the ith floor and $\Delta\Phi_i$ is the relative displacement that occurs at the ith floor when the structure translates laterally according to its fundamental mode, assuming a unit displacement at the top (thus, $\Delta\Phi_i$ is a normalized value). From Eq. (1) it is evident that it is more effective to increase K_{eff} by optimizing the distribution of $\Delta\Phi_i$ 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 for the application of a local intervention through the addition of FRP jackets. The steps in this direction are defined in the following subsections.

3.1 Determining whether 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) K_{eff} should be increased if the effective translational period T_{eff} (based on secant to yielding sections analysis) is more than 25 % higher than the empirical reference value prescribed by EN 1998-1 [16], which estimates T_{ref} as

$$T_{ref} = 0.075 \cdot H_{tot}^{3/4} \tag{2}$$

where H_{tot} is the total building height measured from the crest of a box-type rigid basement or otherwise from the level of the foundation.

b) K_{eff} should be increased if an inspection of the translational shape of vibration reveals evidence of localization of deformation in a 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 T_{trg}

For the vast majority of structures, the period after retrofit will lie between the milestone values T_B and T_D of the EN 1998-1 [16] type I earthquake design spectra. For this period range, the elastic spectral displacement demand may be estimated using

$$\begin{aligned} T_B \le T \le T_C&: S_d(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot \frac{T^2}{40} \\ T_C \le T \le T_D&: S_d(T) = a_g \cdot S \cdot \eta \cdot \beta_o \cdot \left(\frac{T_C T}{40}\right) \end{aligned} \tag{3}$$

At 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

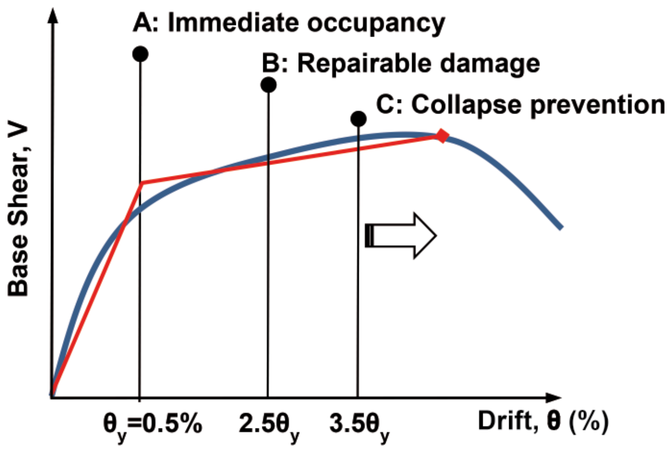

Fig. 1. Performance limits

from the above value if inelasticity occurs. The total average elastic drift ratio (denoted as θ_{dem} for drift demand) for the retrofitted structure is approximated by

$$\theta_{dem} = 1.2 \cdot S_d(T)/H_{tot} \tag{4}$$

Based on experience, the target or improved period T_{trg} of the retrofitted structure may be selected as a value between T_{ref} (from Eq. (2)) and the initial T_{eff}. A note of caution: the cost of the intervention increases as T_{trg} is reduced, getting closer to T_{ref}. Alternatively, T_{trg} may be selected by requiring that the average drift demand θ_{dem} of the structure (Eq. (4)) will not exceed a preset limit value, which after substituting Eq. (3) in Eq. (4) will yield the required value for T_{trg}. Such preset limit values may be 0.5 % (for performance limit A: damage limitation, $\mu_\theta \approx 1$), 1.25 % (B: repairable damage, $\mu_\theta = 2.5$) or 2 % (C: collapse prevention or life safety, $\mu_\theta > 3.5$). It is not advisable to allow for $\mu_\theta > 2.5$ for retrofitted structures. (Fig. 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 μ_θ is the multiplier of θ_y).

(**Example:** The effective translational period of a five-storey structure has been determined as $T_{eff} = 1.1$ s. The seismic hazard is defined by $S = 1.2$ (soil class B), $T_B = 0.15$ s, $T_C = 0.5$ s, $T_D = 2$ s, $a_g = 0.2\ g$, $\eta = 1$ ($\xi = 5$ %), $\beta_o = 2.5$, $H = 15$ m. Based on Eq. (2), it is estimated that $T_{ref} = 0.572$ s. Thus, global interventions are needed to reduce the period value from 1.1 s to a T_{trg} value closer to 0.572 s. For a preset drift ratio limit of 1.25 % and a ductility $\mu_\theta = 2.5$, it follows that the corresponding elastic drift limit would be $\mu_\theta = 1.25\,\%/\theta_{dem}$; thus, $\theta_{dem} = 1.25\,\%/2.5 = 0.50\,\%$ and the T_{trg} value required is

$$0.50\% = \frac{1.2}{15m} 0.2 \cdot 10 \frac{m}{s^2} \cdot 1.2 \cdot 1 \cdot 2.5 \cdot \left(\frac{0.5 \cdot T_{trg}}{40} \right) \Rightarrow T_{trg} = 0.83\,sec$$

This may be considered the maximum acceptable value for the retrofitted structure, which will be designed with a behaviour factor $q = \mu_\theta = 2.5$ and will therefore develop significant, but repairable, damage in the design earthquake. Selecting lower target values for T_{trg} will generally lead to less damage and better overall performance. Note that for a given structure, the lower the period, the greater is 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 for the period, as close as possible to T_{ref}; but the downside of this choice is increased rehabilitation costs.)

3.3 Target for an improved shape of the fundamental mode

The object of selecting the target shape is to achieve an optimum distribution of deformation throughout the structure. A number of simple displacement patterns may be used as benchmarks for selecting a target shape for the fundamental vibration shape when retrofitting an existing, seismically deficient structure (Fig. 2). The closer to a triangular or flexural shape, the greater is the extent of the intervention required and thus the associated costs. 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 $\Phi(z_i) = sin(\pi z_i/(2H_{tot}))$, simplified to $\Phi_i = sin(\pi \cdot i/2n)$ for equal storey heights, where n is the total number of storeys (as in the case of H_{tot}, z_i is measured from the crest of a box-type basement or from the foundation level to the storey of interest; parameters n and H_{tot} have been defined in Eqs. (1) and (2)). The tangential drift ($d\Phi/dz$, Fig. 3a) is more moderate on the lower floors above the

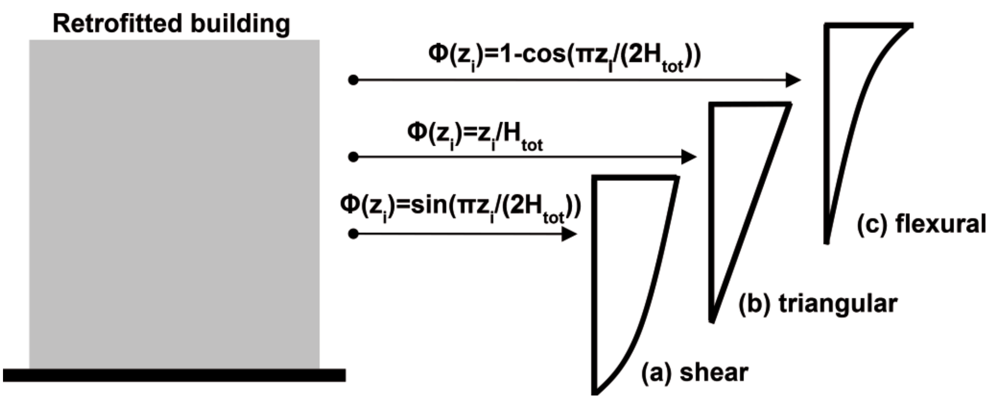

Fig. 2. Lateral displacement profiles: a) shear, b) triangular, c) flexural

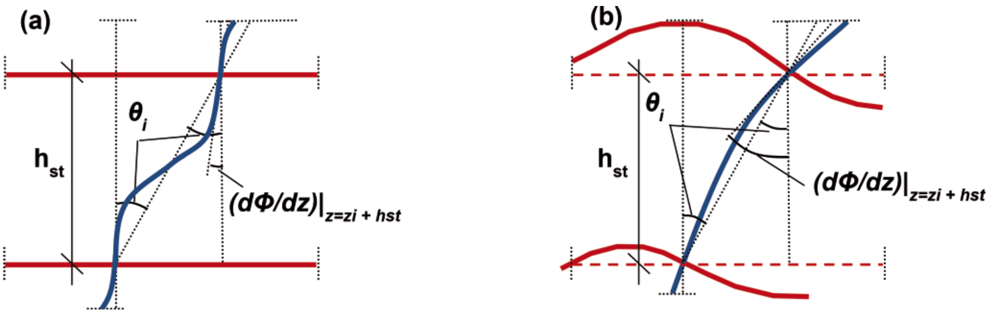

Fig. 3. a) Shear-type and b) flexural-type response floor rotations

first storey of the structure. Buildings with this fundamental response shape have a natural tendency towards damage localization in the lower storeys. Significant beam rotation demands and potential plastic hinge formation are only expected on the lower floors.

- **Flexural-type response:** The trigonometric approximation for this pattern is $\Phi(z_i) = 1\text{-}cos(\pi \cdot z_i/(2H_{tot}))$, simplified to $\Phi_i = 1\text{-}cos(\pi \cdot i/2n)$ for equal storey heights. Beam rotations in structures of this type follow the distribution of the tangential drift ($d\Phi/dz$, Fig. 3b), thus damage in beam plastic hinge regions is expected to be maximum on the upper floors, whereas in the case of walls and columns, plastic hinging is expected at the base ($i = 0,\ z = 0$), consistent with the anticipated maximum base shear value.
- **Triangular response shape:** This is described by $\Phi(z_i) = z_i/H_{tot}$, which may be simplified to $\Phi_i = i/n$ for equal storey heights equal to $h_{st}(n \cdot h_{st} = H_{tot})$. It represents the ideal scenario of a 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 actually achieve this in practice.

3.4 Determining the required stiffness

Engineered modification of the fundamental mode of lateral vibration is achieved through a weighted distribution of added stiffness over the height of the building. In the case of the three benchmark cases (triangular, shear and flexural shapes in Fig. 2), the solution is provided in the charts of Fig. 4. These charts were derived considering a minimum storey height h_{st} = 3 m and unit storey mass m = 1 tonne; they can be used to define a target period and chosen deflection shape (after the user selects the target deflection shape from triangular, shear or flexural). Then, using the charts of Fig. 4a, 4b or 4c, the stiffness required for the first storey can be obtained directly, along with the required distribution of stiffness over the height of the retrofitted building. (Using the charts of Fig. 4d, 4e or 4f and given the number of floors in the structure, it is possible to obtain the required stiffness for all floors as a fraction of the first storey stiffness.)

(**Example:** For a five-storey building with h_{st} = 3.5 m storey height (H_{tot} = 17.5 m) and a triangular response shape selected, a retrofit scenario could be as follows: The target period is 0.64 s according to Eq. (2). It follows that the required first floor stiffness is K_1/m = 1446 kN/m according to Fig. 4a (see red dashed lines). The required storey stiffnesses should decrease towards the upper floors in accordance with the ratios $K_2 = 0.93\ K_1$, $K_3 = 0.80\ K_1$, $K_4 = 0.60\ K_1$, $K_5 = 0.33\ K_1$ (see Fig. 4d, follow red dashed lines), where K_1 = first floor stiffness. Thus, through this very simple approach, the stiffnesses required to achieve the desired pattern of drift distribution and the structural period in the retrofit are fully defined. After implementation, the success of the retrofit design in approaching the chosen lateral shape response and target period may be evaluated through assessment.)

4 Practical implementation in retrofit design

The procedure described in section 3 enables an estimate of the storey stiffness required for a given building (i.e. with known distribution of mass) in order to achieve the specified target period and fundamental mode of vibration characteristics according to the designer's choice. The last step in the procedure involves selecting 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) RC jacketing of selected columns in the building
3) Addition of RC 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 K_i of the retrofitted structure, which comprises ℓ_{cRC} RC jacketed columns, ℓ_w RC walls, ℓ_X spans of X-brace metal pairs, ℓ_{mw} masonry walls and ℓ_{cf} columns strengthened with longitudinal FRP laminates (EBR or NSM), is equal to

$$K_i = \sum_{j=1}^{\ell_{cRC}} K_j^J + \sum_{j=1}^{\ell_w} K_j^w + \sum_{j=1}^{\ell_X} K_j^X + \sum_{j=1}^{\ell_{mw}} K_j^{mw} + \sum_{j=1}^{\ell_{cf}} K_j^f \qquad (5)$$

where:

ℓ_{cRC} number of columns retrofitted with RC jackets on a single floor

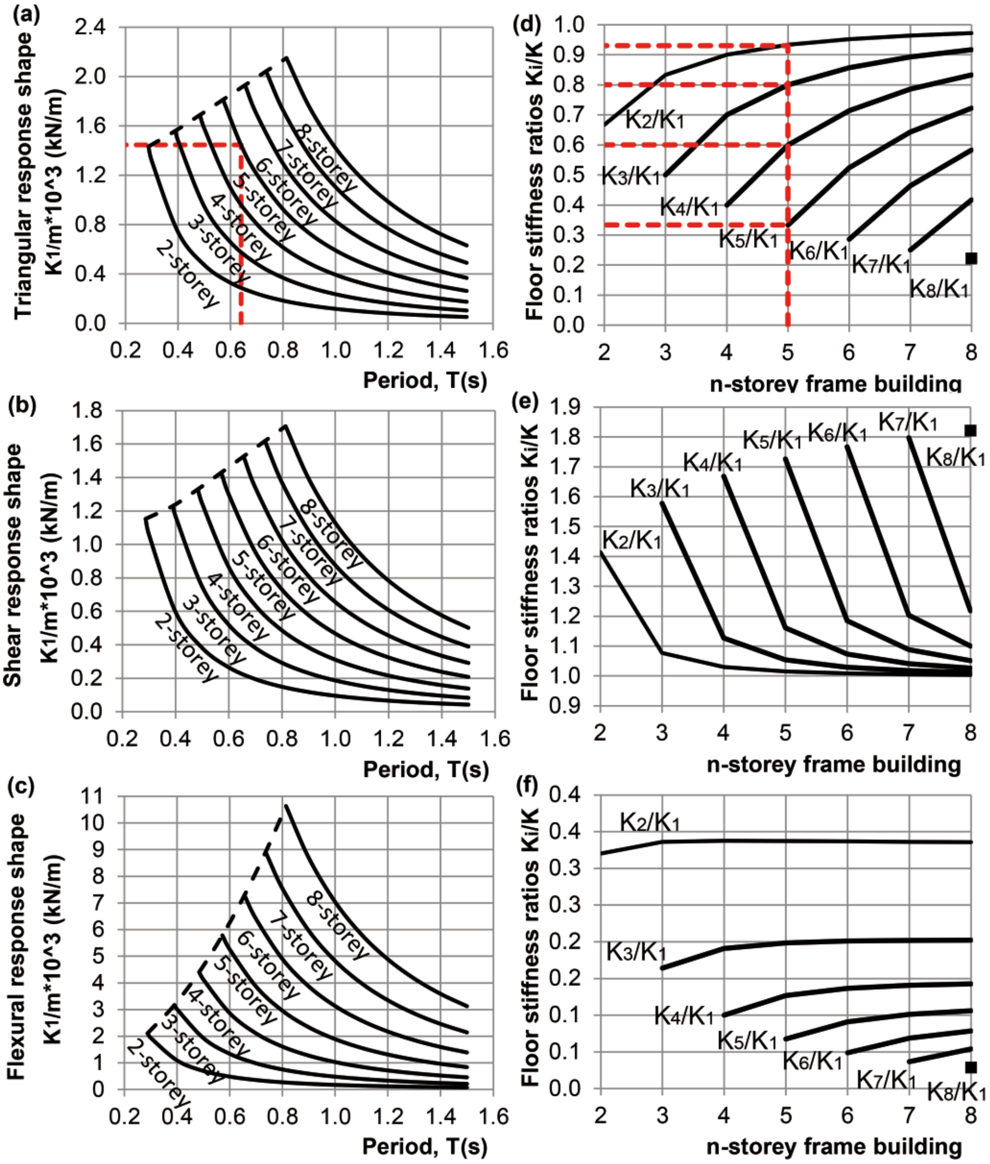

Fig. 4. a), b), c): stiffness-to-mass ratio for first storey K_1/m vs. period for up to eight-storey frames. To obtain the required K_1 values, multiply the ordinate with the mass m (in t); d), e), f): floor stiffness ratios k_i (= K_iK_1) for different lateral deflection shape patterns for frame buildings with two to eight storeys

ℓ_w number of walls added for stiffening the structure in the direction of action

ℓ_X number of X-brace pairs added on the floor to add stiffness in the direction of action

ℓ_{mw} number of infill panels added on the floor in the direction of action

ℓ_{cf} number of columns strengthened with longitudinal FRP strips (externally bonded or NSM)

The contributions of each of these techniques/elements to the storey stiffness K_i are listed in Thermou and Pantazopoulou [17] and are summarized here for completeness in the Appendix. Only the possible contribution of FRP to the stiffness terms K_j^f are considered in the following detailing sections.

5 Detailing of FRP interventions for seismic applications

Seismic retrofitting of RC structures with FRP may be carried out in order to upgrade a variety of structural deficiencies if upon assessment according to the established code framework (EN 1998-3, [18]) it is shown that seismic safety may be compromised at the design performance limit state. For evaluating the structure's safety and for 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 – through special provisions – the national standards enable a different importance level category to be assigned to the retrofitted structure in order to account for a residual service life different from the 50-year standard.

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

Material safety factors refer to the FRP materials typically used today (GFRP, CFRP and AFRP with strengths ranging from 1500 to 3500 MPa and nominal rupture strains from 2.5 down to 1.5 %). For retrofit design these are:

Table 1. FRP material redundancy safety factors

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

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, [18]).
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, [16]) as listed in Table 1.

5.1 Strategies for 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 on any given floor in order to minimize the risk of disproportionate damage to any single element. (The curvature at yielding of a linear RC member is approx. $\phi_y = 2\varepsilon_{sy}/h$, where h = cross-section depth and ε_{sy} = characteristic yield strain of reinforcing steel. Thus, the chord rotation (see definition in section 5.2.1) at yielding is $\theta_y = 1/6\phi_y H = 1/3\varepsilon_{sy}[H/h]$, where H/h is the aspect ratio of the member (H = member clear height and h = cross-section depth of member). Therefore, two members having very different aspect ratios yield at very different relative drift ratios.) The implication is that, during an earthquake, for any given magnitude of lateral displacement, members with different aspect ratios on a single floor reach very different states of damage. The same effect is observed if the structure has plan irregularities that cause a torsional response. Clearly, major building irregularities cannot be eliminated using FRP as a strengthening technique, although the addition of FRP strips as longitudinal reinforcement can be counted as a global intervention as they can be used to increase the strength and stiffness of individual members.

Thus, a good strategy is the selective retrofitting of members that belong to the lateral load-resisting system in order to achieve similar relative drift ratios at yielding and also enhance deformation capacity through confinement. It is essential to eliminate brittle failure modes through FRP jacketing 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 for 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, i.e. 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 fibres 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 around the member cross-section.
6) Increasing the diagonal tension capacity of beam-column joints by using FRP material installed with fibres located along the line of the principal tensile stresses.

As interventions 2) to 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 a requirement that localized strengthening should not reduce the overall ductility of the structure.

In detailing the retrofit solutions, each retrofitted member is designed using capacity design principles. 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 V_{flex}.

When considering individual members, case 1) in the list of intervention measures is a global measure, effectively increasing V_{flex}. On the other hand, local strengthening schemes for individual linear members, i.e. cases 2) to 5) above, have relatively little effect on V_{flex} and depend on the confining action of the FRP reinforcement. Thus, the efficacy of a 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 resistance mechanism 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 control the occurrence of the modes of failure, a static model is envisioned for each member undergoing retrofit as depicted in Fig. 5. In this static model, the member develops a constant shear force along its length, reaching a maximum flexural moment at the end points of

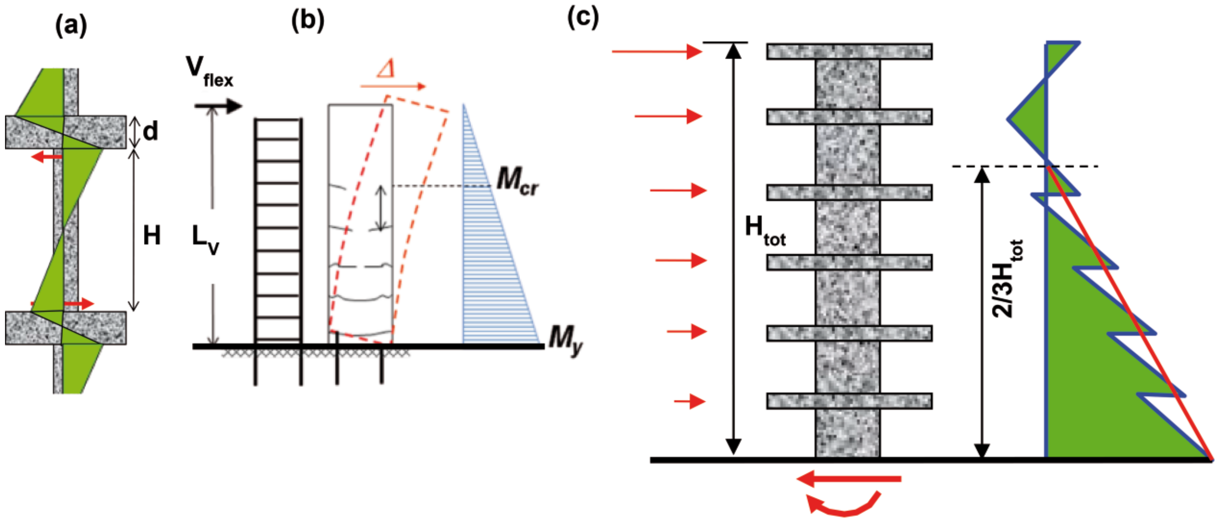

Fig. 5. a) Static model used for beam-column elements undergoing lateral sway. b) The cantilever part has the same moment distribution as the swaying column over the length from point of contraflexure to face of support. Owing to this similarity, a cantilever member is used to illustrate the concept of shear span $L_V = H/2$ and the relation between shear demand and flexural moment strength $V = M/L_V$. c) Static model for structural walls. The red line defines an "equivalent" linear moment diagram to relate shear demand to moment strength $V = M/(2/3H_{tot})$

their deformable length where partial restraint to rotation may exist. (For example, during lateral sway, a column is considered to develop a maximum moment at its base and at its top cross-section at the beam soffit; a beam is considered to develop maximum moments at its end supports, at the face of the columns; a flexural wall is considered to develop a maximum moment at the base.)

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

5.2 Determining the displacement demand of the individual structural members

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

With reference to Fig. 1, it is recommended that q should not exceed 2.5 for ordinary structures (for performance limit B: repairable damage $q \approx \mu_\theta = 2.5$). Higher values should be avoided if the structure has been designed to previous standards or has irregularity on plan or over its 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 μ_Δ of the structure may be estimated using

$$\mu_\Delta = \begin{cases} q & for\ T > T_C \\ 1 + \dfrac{T_C}{T} \cdot (q-1) & for\ T < T_C \end{cases} \quad (6)$$

where T_C is the end of the plateau of the type I spectrum for the design soil conditions (see EN 1998-1 [16]). Depending on the target response shape, the displacements required at the individual floors of the structure are determined from EN 1998-3 [18]:

$$\mu_{\Delta,i} = \mu_{\theta,i} = \frac{\Delta_i}{\Delta_{y,i}};\ \Delta_i = (\mu_\Delta \cdot \Delta_y) \cdot \Delta\Phi_i \quad (7)$$

where Φ_i is the coordinate of the target response shape on the ith floor and $\Delta\Phi_i = \Phi_i - \Phi_{i-1}$.

– The required curvature ductility at the critical sections of members on the ith floor may be obtained with reasonable approximation using

$$\mu_{\phi,i} = 2\mu_{\theta,i} - 1 \quad (8)$$

whereas the maximum compression strain demand for the columns $\varepsilon_{cu,c}$ may be estimated from (KANEPE [19]):

$$\varepsilon_{cu,c} = 2.2 \cdot \mu_\phi \cdot \varepsilon_{sy} \cdot \nu_{d,max} \geq 0.0035 \quad (9)$$

where $\nu_{d,max}$ is the maximum axial load ratio of a typical column for the seismic combination (defined in section 6.1)[2] and ε_{sy} is the yield strain of the steel.

5.2.1 Increasing of the local rotational capacity of RC 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, θ_i in Fig. 3) at a distance equal to the shear span $L_V = M/V$. (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 aforementioned sections to the shear span, referred to as relative storey drift ratio; however, if beams

[2] Using the calculated compression strain demands, the amount of confining reinforcement required may be obtained from pertinent stress-strain models for FRP-confined concrete, which relate the thickness of the FRP jacket to the compression strain capacity of the encased concrete.

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 behaviour mechanisms 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 underdesigned due to the lack of longitudinal reinforcement. In such a case it is deemed necessary to increase the column capacity under combined bending and axial load towards a "strong column-weak beam" situation.

6 FRP as a means of enhancing strength

6.1 Increasing the flexural strength and stiffness of RC members by adding longitudinal FRP reinforcement

The aim of adding FRP strips on the tension side of a member parallel to its longitudinal axis is to enhance the flexural strength (and hence 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. The cross-section carries an average design axial load $N_{G+0.3Q-E} \le N_{G+0.3Q} \le N_{G+0.3Q+E}$ obtained from the seismic design combination. For this axial load, which is referred to as the axial load ratio $\nu_{Ed} = N_{G+0.3Q}/(f_{cd}bd)$ and the longitudinal reinforcement ratios ρ_{s1} and ρ_{s2}, the reference flexural strength of the cross-section is M^o_{Rd} (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 M_{Ed}, FRP strips should 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 (a method that ought to be pursued if the shape of the cross-section allows it). Figs. 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 in Fig. 6.2). The maximum allowable stress in 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 ($\varepsilon_{s1,min} > \varepsilon_{yd}$).

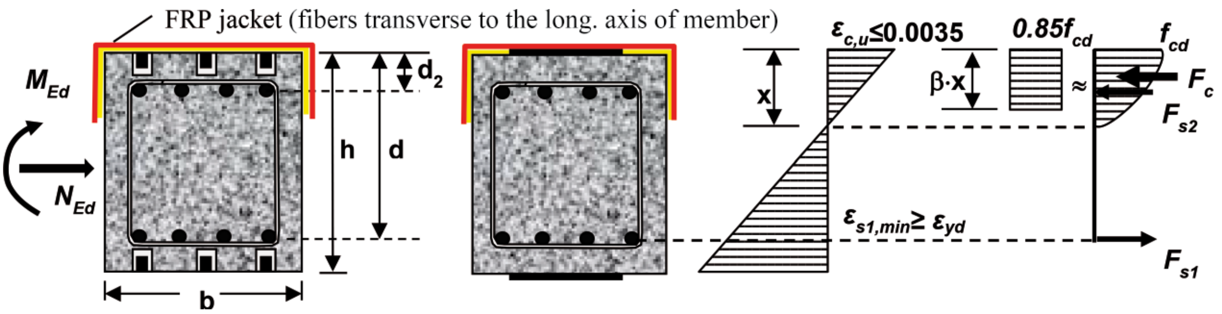

Fig. 6.1. Various types of flexural strengthening of prismatic column/beam cross-section. The red line illustrates the jacket arrangements implied by the various values of a_1 and a_2 in Eq. (10). The yellow line marks the adhesive layer. Clearly, this should not be interpreted as a recommended arrangement of the jackets. Wherever possible, the jacket ought to be wrapped fully around the cross section.

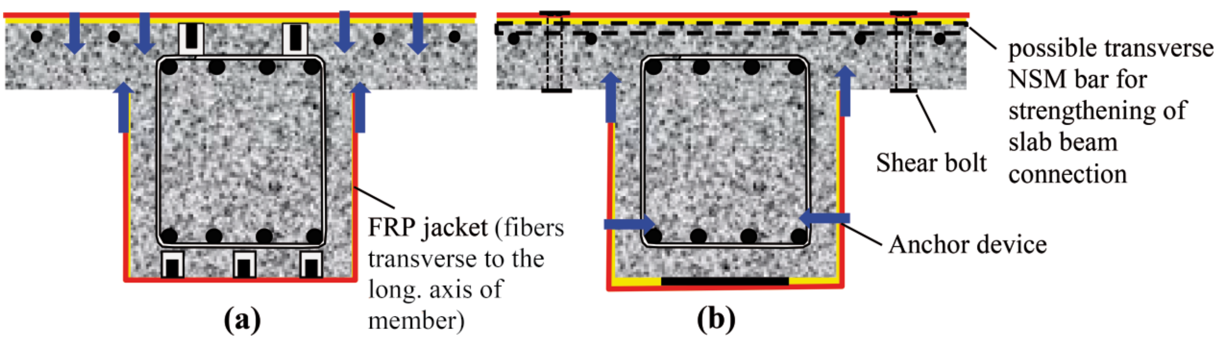

Fig. 6.2. Various types of flexural strengthening for T-beam cross-sections secured through the addition of longitudinal NSM or EBR measures. The outer solid line illustrates jacket arrangements required to secure the flexural intervention. Owing to the presence of slab longitudinal reinforcement, addition 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))

b) Maximum compressive strains in unconfined concrete in the compression zone may not exceed $\varepsilon_{c,u}$.
c) Tension strains in the FRP longitudinal reinforcement ε_f may not exceed the design limit $\varepsilon_{f,max} = \varepsilon_{fu}/\gamma_f$, where γ_f is taken from Table 1 for primary reinforcement on a brittle substrate.

For dimensioning of the reinforcement, the axial tensile strain ε_{fd} in the FRP layer may not exceed the following limit:

$$\varepsilon_{fd} \le a_1 \cdot a_2 \cdot \varepsilon_{f,max}, \quad \varepsilon_{f,max} = \frac{\varepsilon_{fu}}{\gamma_f} \tag{10}$$

where:
$a_1 = 1$ for EBR-FRP layer
$a_1 = 1.4$ for NSM-FRP layer
$a_2 = 1.0$ if no transverse reinforcement has been applied over the FRP reinforcement
$a_2 = 1.4$ 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 $\varepsilon_c = \varepsilon_{cu}$ is prioritized as the limiting failure mode after reinforcement yielding. The procedure for the case of FRP failure follows similar principles. Requirements a) to c) above may be expressed as limits on the normalized depth of the compression zone of the cross-section after retrofitting $\xi = x/d$:

$$\frac{0.0035}{0.0035+\varepsilon_{fd}} \cdot \left(1+\frac{d_2}{d}\right) \le \xi \le \frac{0.0035}{0.0035+\varepsilon_{yd}} = \xi_{bal} \tag{11}$$

To calculate the required area of the added tension reinforcement $A_f = \rho_f \cdot (b \cdot d)$, the values of ξ and ρ_f are solved from the equilibrium requirements:
– Sum of axial forces = 0 ($\nu_{Ed} > 0$ for compression)

$$\begin{aligned} &0.85 \cdot f_{cd} \cdot \beta \cdot \xi + (\rho_{s2} - \rho_{s1}) \cdot f_{yd} - \nu_{Ed} \cdot f_{cd} = E_f \cdot \varepsilon_{fd} \cdot \rho_f \\ &\Rightarrow \rho_f = \frac{0.85 \cdot \beta \cdot \xi - \nu_{Ed} - (\rho_{s1} - \rho_{s2}) f_{yd}/f_{cd}}{f_{fd}/f_{cd}} \\ &\Rightarrow \rho_f = \frac{0.85 \cdot \beta \cdot \xi - \nu_{Ed} - (\rho_{s1} - \rho_{s2}) \dfrac{f_{yd}}{f_{cd}}}{\dfrac{E_f}{f_{cd}} \cdot \dfrac{0.0035}{\xi} \cdot \left(1 + \dfrac{d_2}{d} - \xi\right)}; \; f_{cd} = f_{ck}/\gamma_c; \; f_{yd} = f_{yk}/\gamma_s \end{aligned} \tag{12}$$

Parameter β is the depth of the equivalent rectangular stress block of concrete compressive stress, normalized by the depth of the compression zone x (see Fig. 6.1; the intensity of the stress block is $0.85f_{cd}$). Parameters ρ_{s1} and ρ_{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 effective cross-sectional area, i.e. $\rho_{s1} = n_1 \pi D_{b1}^2/(4hd)$, where n_1 and D_{b1} are the number and diameter of the tension bars respectively.

– An upper limit for ρ_f is obtained by substituting $\xi = \xi_{bal}$ in the above[3].
– For the retrofit to be possible, it is also required that

$$\beta \cdot 0.85 \cdot \xi_{bal} \ge \nu_{Ed} + (\rho_{s1} - \rho_{s2}) \frac{f_{yd}}{f_{cd}} \tag{13}$$

– To find the required area of the FRP layer in order that the strengthened cross-section has a flexural strength $M_{Rd} > M_{Ed}$ in the presence of a design axial load $N_{Ed} = \nu_{Ed} b d f_{cd}$, the following procedure is used: First, the sum of moments is considered about the centroid of the FRP layer:

$$M_{Ed,f} = M_{Ed} + N_{Ed} \cdot (h - y_{cg}) = \mu_{Ed,f} \cdot bd^2 f_{cd} \tag{14a}$$

Note that both $M_{Ed,f}$ and the normalized value $\mu_{Ed,f}$ are defined with reference to the centroid of the FRP layer (this is the significance of the subscript f); $M_{Ed,f}$ is calculated first, given the design values of moment and axial load for the retrofitted cross-section. Next, the design value of $\mu_{Ed,f}$ is obtained from $M_{Ed,f}$ after normalizing with $bd^2 f_{cd}$. This is set equal to the normalized moment of internal forces about the same point of reference, given by

$$\begin{aligned} \mu_{Ed,f} &= 0.85\,\beta\xi\left(1+\frac{d_2}{d}-0.4\xi\right)+\left(\rho_{s2}-\rho_{s1}\frac{d_2}{d}\right)\frac{f_{yd}}{f_{cd}} \\ &= 0.85\,\beta\xi\left(1+\frac{d_2}{d}-0.4\xi\right)+\mu_{Ro} \end{aligned} \tag{14b}$$

where μ_{Ro} only depends on the geometric characteristics of the original cross-section. The FRP layer is calculated so that the total value for $\mu_{Ed,f}$ meets the strength demand of the retrofitted cross-section. For easy reference, the first term on the right-hand side of Eq. (14b), given by

$$\Delta\mu_{Ed,f} = 0.85 \cdot \beta \cdot \xi \cdot \left(1 + \frac{d_2}{d} - 0.4\xi\right)$$
$$\text{where, } \Delta\mu_{Ed,f} = \mu_{Ed,f} - \mu_{Ro} \tag{14c}$$

has been tabulated in Table 2 for usual values of (d_2/d).

The values in Table 2 were calculated using $\beta = 0.8$, which corresponds to the ultimate strain in the extreme compressed fibre of the cross-section $\varepsilon_{cu} = 0.0035$. Ranges of parameters outside the table represent cases where the resulting normalized depth of the compression zone ξ 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 ξ values above the upper limit of Eq. (11), in the shaded part of Table 2[4], or

[3] For $f_{yk} = 500$ MPa, $\xi_{bal} = 0.62$; for $f_{yk} = 400$ MPa, $\xi_{bal} = 0.66$; for $f_{yk} = 220$ MPa, $\xi_{bal} = 0.78$;
[4] The shaded part indicates concrete crushing for embedded tensile reinforcement with $f_{sy} = 500$ MPa (B500C); values are increasingly conservative in the range of higher FRP strains, i.e. for lower ξ values, to account for the fact that the actual stress in compression reinforcement is lower than the assumed yield limit. Stricter criteria (e.g. a minimum required value of tensile strain in the extreme layer of reinforcement, $\varepsilon_{s1,min} = 0.004$) will extend the shaded portion of the table downwards.

Table 2. Normalized moment $\Delta\mu_{Ed,f}$ for various values of maximum allowable FRP strain ε_{fd}

d_2/d =	0.05		0.10		0.15	
ε_{fd}	ξ	$\Delta\mu_{Ed,f}$	ξ	$\Delta\mu_{Ed,f}$	ξ	$\Delta\mu_{Ed,f}$
0.0022	0.645	**0.347**	0.675	**0.381**	0.706	**0.417**
0.0024	0.623	**0.339**	0.652	**0.372**	0.682	**0.407**
0.0026	0.602	**0.331**	0.631	**0.364**	0.660	**0.398**
0.0028	0.583	**0.324**	0.611	**0.356**	0.639	**0.389**
0.003	0.565	**0.317**	0.592	**0.348**	0.619	**0.380**
0.0035	0.525	**0.300**	0.550	**0.329**	0.575	**0.360**
0.004	0.490	**0.285**	0.513	**0.312**	0.537	**0.341**
0.0045	0.459	**0.271**	0.481	**0.297**	0.503	**0.325**
0.005	0.432	**0.258**	0.453	**0.283**	0.474	**0.309**
0.0055	0.408	**0.246**	0.428	**0.270**	0.447	**0.295**
0.006	0.387	**0.236**	0.405	**0.258**	0.424	**0.282**
0.0065	0.368	**0.226**	0.385	**0.248**	0.403	**0.271**
0.007	0.350	**0.217**	0.367	**0.238**	0.383	**0.260**
0.0075	0.334	**0.208**	0.350	**0.228**	0.366	**0.250**
0.008	0.320	**0.200**	0.335	**0.220**	0.350	**0.240**
0.0085	0.306	**0.193**	0.321	**0.212**	0.335	**0.232**
0.009	0.294	**0.186**	0.308	**0.205**	0.322	**0.224**
0.0095	0.283	**0.180**	0.296	**0.198**	0.310	**0.216**
0.01	0.272	**0.174**	0.285	**0.191**	0.298	**0.209**
0.0105	0.263	**0.169**	0.275	**0.185**	0.288	**0.202**
0.011	0.253	**0.163**	0.266	**0.179**	0.278	**0.196**
0.0115	0.245	**0.159**	0.257	**0.174**	0.268	**0.190**

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

Table 2 is entered with the value of d_2/d and the required normalized moment increase $\Delta\mu_{Ed,f}$ (calculated about the centroid of the longitudinal FRP layer). The estimated (from Table 2) value of ξ is entered in the expression for ρ_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 ε_{fd} value determined with Eq. (10).

6.1.1 Additional requirements

FRP reinforcement used as post-installed primary reinforcement to strengthen RC structural members for seismic applications should be anchored so that they can carry their forces at the critical sections where the 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 L_e available on each side of a critical section should satisfy the design requirements for development length given in the *fib* Model Code for Concrete Structures [20].

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 $V_{Ed} = M_{Rd}/L_v$ (Fig. 5). The retrofitted member should be checked for shear, and additional shear reinforcement should be provided to ensure that the factored shear resistance V_{Rd} exceeds V_{Ed}.

The stiffness increase attained through the addition of the FRP reinforcement may be quantified by the magnitude of the effective EI_j of the jth 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

$$EI_j = \frac{M_{y,j}}{2\varepsilon_{sy}/h}; \quad K_j^{FRP} = 12\frac{EI_j}{H_j^3} = 6\frac{h}{H_j}\cdot\left[\frac{M_{y,j}}{\varepsilon_{sy}\cdot H_j^2}\right] \tag{15}$$

Note that H_j/h is the aspect ratio of the member and $M_{y,j}$ is the moment resistance of the jth strengthened member at the onset of yielding of the longitudinal reinforcement.

6.1.2 Additional detailing considerations

When a member's flexural capacity is increased, particular care should be taken to anchor the adopted FRP reinforcement properly. Longitudinal fibres used for strengthening RC members subjected to combined bending and axial load should be properly confined to avoid debonding and concrete spalling under cyclic loads.

6.2 Increasing the deformation capacity of RC members through FRP jacketing

To increase the deformation capacity of an RC member, any type of undesirable brittle failure should be eliminated. The member should 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 for the effective elimination of these occurrences and also to 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 fibres are oriented transverse to the longitudinal axis of the upgraded member and on a minimum of three faces (properly anchored U-shaped and □-shaped types exclusively) of the member's cross-section in order to facilitate a 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 listed in section 5.1. 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 centre. The average confining pressure σ_x acting along the x axis may be estimated considering equilibrium on a plane intersecting the cross-section along line A-A. The calculation of the average confining pressure σ_y, acting in the y direction, is similar.

$$\sigma_x = \underbrace{\rho_{fw-x} E_f \varepsilon_{fd}}_{\text{FRP component}} + \underbrace{\rho_{sw-x} f_{y,st}}_{\text{contribution of links}} \tag{16a}$$

$$\sigma_y = \underbrace{\rho_{fw-y} E_f \varepsilon_{fd}}_{\text{FRP component}} + \underbrace{\rho_{sw-y} f_{y,st}}_{\text{contribution of links}} \tag{16b}$$

where $\rho_{fw-\chi}$ is the FRP web reinforcement ratio (geometric ratio) provided in the x direction $(2t_f/h)$ for a continuous jacket having an effective thickness t_f. Similarly, $\rho_{fw-y} = (2t_f/b)$.

The effective thickness is estimated from the number of FRP layers n in the jacket and the thickness of a single layer t_o. Therefore, $t_f = t_o \cdot n^{0.85}$ for $n \geq 4$. Otherwise, $t_f = t_o \cdot n$ for $n < 4$[5] (KAN.EPE [19]).

Parameter ε_{fd} is the design value for the strain capacity of the transverse jacket, defined in section 6.2.3 below.

Parameter $\rho_{sw-\chi}$ 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 s and having a total sectional area A_{sw-x}, this is defined by $\rho_{sw-x} = A_{sw-x}/(s \cdot h_o)$. Similarly, $\rho_{sw-y} = A_{sw-y}/(s \cdot b_o)$.

A uniform lateral pressure is assumed to confine the FRP-encased concrete in compression. This pressure, denoted by σ_{lat}, is the average of σ_χ and σ_y defined above. To account for the reduced efficiency of confinement in rectangular cross-sections (Fig. 7b), an effectiveness coefficient α_f is used to modify the FRP component of the confining stress. This is similar to the effectiveness coefficient α_w used for stirrup-generated confinement (EN 1998-1 [16]):

$$\sigma_{lat} = \frac{1}{2}[\alpha_f \cdot (\rho_{fw-y} + \rho_{fw-\chi}) \cdot E_f \varepsilon_{fd} + \alpha_w \cdot (\rho_{sw-y} + \rho_{sw-\chi}) \cdot f_{y,st}] \Rightarrow \tag{17a}$$
$$\sigma_{lat} = 0.5(\alpha_f \cdot \rho_{fv} \cdot E_f \varepsilon_{f,d} + \alpha_w \cdot \rho_{sv} \cdot f_{y,st})$$

Parameters ρ_{fv} and ρ_{sv} are the volumetric ratios of transverse reinforcement (Fig. 7a):

$$\rho_{fu} = \frac{2 \cdot t_f \cdot (h+b)}{h \cdot b}; \quad \rho_{su} = \frac{A_{sw-\chi} \cdot b_o + A_{sw-y} \cdot h_o}{s \cdot h_o \cdot b_o} \tag{17b}$$

6.2.2 Confinement effectiveness coefficients α_f, α_w

The confinement effectiveness coefficient is the volume ratio of the encased member that is effectively confined. With reference to Fig. 7a, α_w is defined as follows for stirrup confinement according with EN 1998-1 [16]:

$$\alpha_w = \alpha_n \cdot \alpha_s \qquad \alpha_n = 1 - \sum_{i=1}^{n} b_i^2 / 6 b_o h_o \qquad \alpha_s = \left(1 - \frac{s}{2b_o}\right)^2 \tag{18}$$

5 The jacket layers are calculated as follows: From $t_f = b \times \rho_{f\text{-}y}/2$, calculate $n = t_f/t_o$. If $n < 4$, then the calculated number of layers is applied, but if $n > 4$, then recalculate the increased number of layers by applying $n = [t_f/t_o]^{1/0.85}$. As the number of layers increases, so the effective strain in the exterior layers is reduced due to the increased stiffness of the jacket. Therefore, the choice of alternative strengthening schemes that make better use of material resources ought to be considered.

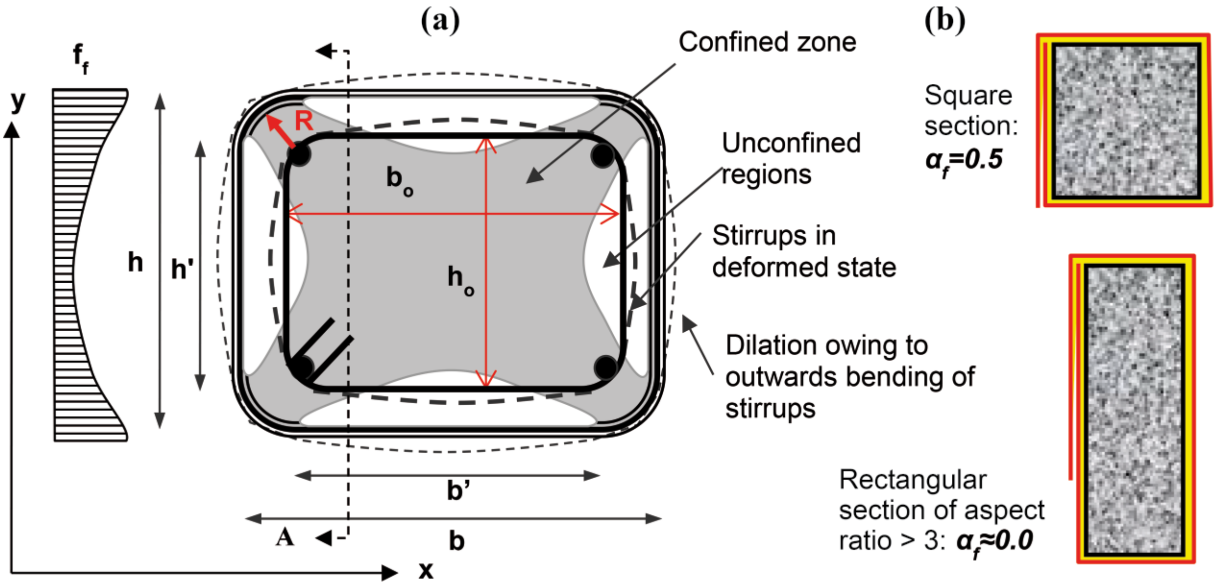

Fig. 7. a) Definition of terms for estimating confining pressure, b) exclusively □-shaped FRP types

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 [2], 35 [3], 40 [21]):

$$\alpha_f = 1 - \frac{(b-2R)^2 + (h-2R)^2}{3bh \cdot (1-\rho_g)} = 1 - \frac{(b')^2 + (h')^2}{3bh \cdot (1-\rho_g)} \tag{19}$$

Parameter ρ_g is the longitudinal reinforcement ratio of the member's cross section, and 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 < 1. In lightly reinforced members that are considered for FRP jacket retrofits, 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 are 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, $\alpha_f = 0.5$.
- For cross-sections with an aspect ratio > 3, the confinement effectiveness is practically negligible and $\alpha_f \approx 0$. 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 in FRP jacket ε_{fd}

The allowable tensile strain in the jacket ε_f should not exceed the design limit $\varepsilon_{f,max} = \varepsilon_{fu}/\gamma_f$, where γ_f is taken from Table 2 depending on the jacketing arrangement:

a) Fully wrapped retrofit arrangement refers to closed jackets (□-shaped) that fully encase the member.
b) Anchorage on brittle substrate refers to open jackets (U-shaped) that do not enclose the member on all sides.

For proportioning the FRP jacket, the axial tensile strain ε_{fd} in any FRP layer should not exceed the following limit:

$$\varepsilon_{fd} \le \eta_1 \cdot \eta_2 \cdot \eta_3 \cdot \varepsilon_{f,max}; \quad \varepsilon_{f,max} = \frac{\varepsilon_{fu}}{\gamma_f} \tag{20}$$

Factor η_1 accounts for the radius of chamfer R at the corners of the member (also known as the strain efficiency factor, see *Pantazopoulou* et al. [22], *Tastani* et al. [23], *Pellegrino* and *Modena* [24], see also Fig. 8):

$$\eta_1 = 0.25 + 2 \cdot (2R + D_b)/b' \le 1.0 \tag{21}$$

Parameter D_b is the embedded corner bar diameter. Eq. (21) is valid for rectangular cross-sections only (b' is the largest cross-section side); for circular members, $n_1 = 1$.

Factor η_2 accounts for the development length of the wrap:

$$\eta_2 = l_b^{avail}/l_b^{min} \le 1 \tag{22}$$

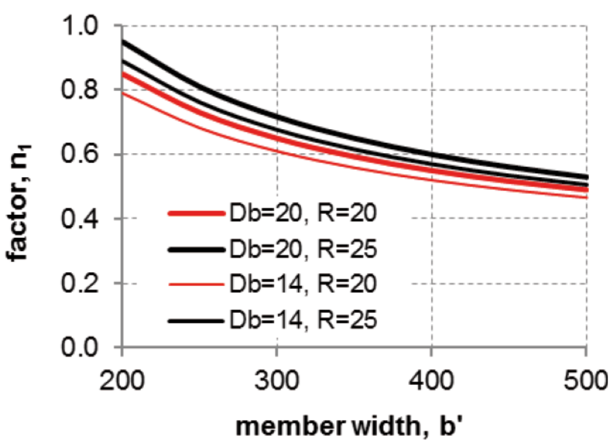

Fig. 8. Definition of factor n_1 vs. the larger member cross-section side for several values of R and D_b

where l_b^{min} is the minimum required overlap length of the exterior jacket layer (i.e. as calculated by implementing Eq. (23) or (24)) and l_b^{avail} is the available length of the cross-section side where the FRP is to be anchored.

Factor η_3 accounts for the redundancy of the jacket against debonding failure.

- For fully wrapped jackets, $\eta_3 = 1.0$.
- For U-type arrangements with special details at the ends to secure the jacket against debonding (e.g. adhesive anchors, NSM details, etc.), $\eta_3 = 1.0$.
- For U-type arrangements without special measures against debonding, $\eta_3 = 0.85$.
- For straight layers with special details at the ends to secure the jacket against debonding (e.g. adhesive anchors, NSM details, transverse confining wraps), $\eta_3 = 0.9$.
- For straight layers (parallel to the web depth) without special measures against debonding, $\eta_3 = 0.6$. (Note that this arrangement is discouraged by most relevant design codes due to the high risk of debonding, e.g. ACI 440 [1], *fib* 9.3 Bulletin 35 [3], KAN.EPE [19]); however, it may be improved by using chemical anchors or other effective clamping means.)

(**Note:** The usable design FRP strain is limited in order to protect the retrofit against 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 compressive 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 R according with the requirements of section 6.2.3.
(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 should be limited according to section 8.1.1.
(b.1) Debonding failure of the FRP in a closed jacket arrangement (□-type). The most critical layer 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 l_b^{min} is

$$l_b^{min} = 1.6\sqrt{E_f\, t_o\, s_{ao}/\tau_a} \tag{23}$$

where τ_α is the shear strength of the adhesive at the stage of plastification and $s_{\alpha o}$ the slip of the adhesive at brittle shear failure (data for the adhesive must be provided by the adhesive supplier). For an adhesive that exhibits ductile shear response up to $s_{\alpha u}$, the coefficient 1.6 may be eliminated and $s_{\alpha u}$ used in lieu of $s_{\alpha o}$ in Eq. (23).
(b.2) Debonding failure of the FRP in an open (U-type) FRP jacket arrangement (i.e. in the case of anchorage in a 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 w_{cr} – is

$$l_b^{min} = 1.6\sqrt{E_f\, t_f\, w_{cr}/\tau_{b1}} \tag{24}$$

The design bond strength is $\tau_{b1} = f_{ctk0.05}/\gamma_{fb}$, where $f_{ctk0.05}$ is the characteristic tensile strength of the concrete substrate and $\gamma_{fb} = 1.5$, the concrete material safety factor. Design calculations may be performed for $w_{cr} = 0.5$ mm. The effective jacket thickness t_f is defined in section 6.2.1.)

6.2.4 Stress-strain law for FRP-confined concrete

The confined concrete strength f_{cc} and the corresponding strain at attainment of peak stress ε_{cc} in the compression zone of the encased cross-section may be calculated from the classical confinement model of *Richart* et al. [25] adapted to account for the greater compliance of jackets compared with conventional stirrups:

$$f_{cc} = f_{ck} + 3\sigma_{lat};\quad \varepsilon_{cc} = \varepsilon_{co}\left(1 + 5\left(\frac{f_{cc}}{f_{ck}} - 1\right)\right) \tag{25}$$

By substituting Eq. (17a) in Eq. (25) and assuming $\varepsilon_{co} = 0.002$ (strain at peak stress of unconfined concrete), we obtain the following:

$$f_{cc} = f_{ck} + 3\sigma_{lat} = f_{ck} + 1.5(\alpha_f \rho_{fv} E_f \varepsilon_{fd} + \alpha_w \rho_{sv} f_{y,st})$$
$$\varepsilon_{cc} = \varepsilon_{co}\left(1 + 5\frac{\sigma_{lat}}{f_{ck}}\right) = 0.002 + 0.015\frac{\alpha_f \rho_{fv} E_f \varepsilon_{fd} + \alpha_w \rho_{sv} f_{y,st}}{f_{ck}} \tag{26}$$

The failure strain of confined concrete $\varepsilon_{cu,c}$ corresponding to a compression strength reduction in excess of 15 % is obtained from (*Pantazopoulou* et al [22], Fig. 9b):

$$\varepsilon_{cu,c} = \varepsilon_{c,u} + 0.075\cdot\left(\zeta\cdot\frac{\alpha_f \rho_{fv} E_f \varepsilon_{fd} + \alpha_w \rho_{sv} f_{y,st}}{f_{ck}} - 0.1\right) \ge 0.0035$$
$$for\ \varepsilon_{cu,c} \le 0.01,\quad \zeta = 1;\quad for\ \varepsilon_{cu,c} \ge 0.02,\quad \zeta = 0.6 \tag{27}$$

Coefficient ζ 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 when determining the compression stress-strain law. Such a safety factor may have an adverse effect on the estimated hierarchy of failure when establishing capacity design principles. It is recommended that a safety factor be applied to the calculated member strength after retrofitting to account for uncertainties.

A note of caution is in order regarding the confinement models available: all models listed in the literature have been calibrated against a very large database of tests conducted on axially compressed members (Fig. 9a). Specimens were either reinforced or unreinforced. The tests conducted correspond to the red point on the axial load-moment interaction diagram plotted in Fig. 9c. The stress-strain relationships derived do not account for the strain gradient effects that occur due to flexural moments. Using stress-strain relationships obtained from axial load tests to model the stress-strain behaviour of concrete in

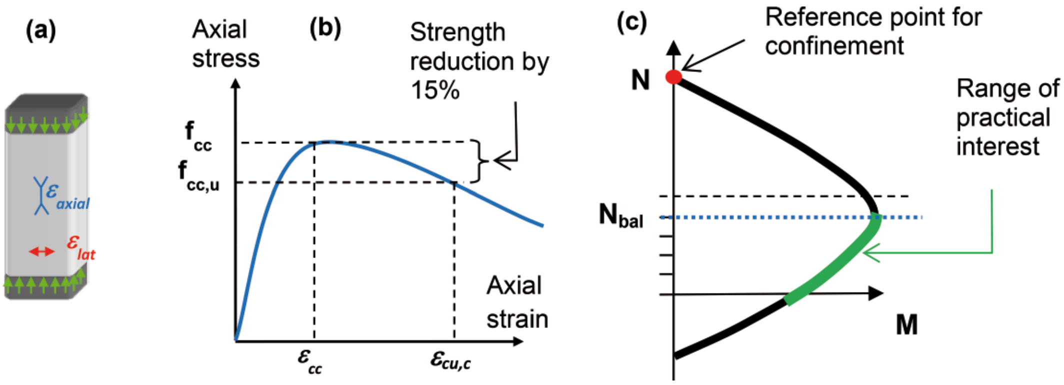

Fig. 9. a) Typical test of FRP-confined concrete in compression, b) nomenclature for stress-strain milestones, c) axial load-moment interaction diagram for typical prismatic element

the confined compression zone of members under combined axial load and moment (range marked by green in Fig. 9c) is an area of inconsistency in the FRP-related literature.

(Note: 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 [18]: Here, the confined concrete strength and the associated strain ε_{cc} as well as the strain capacity $\varepsilon_{cu,c}$ are calculated through Eq. (28a), where $\varepsilon_{f,eff}$ is lower than the jacket strain at rupture (suggested values for ε_{fu} are 0.015 for CFRP and AFRP, 0.02 for GFRP):

$$f_{cc}=f_{co}\left[1+3.7\left(\frac{\alpha_{eff}\rho_f f_{f,eff}}{f_{co}}\right)^{0.86}\right];\varepsilon_{cc}=\varepsilon_{co}\cdot\left[1+5\cdot\left(\frac{f_{cc}}{f_{ck}}-1\right)\right] \quad (28a)$$

$$f_{f,eff}=\alpha_{eff}\frac{2t_f}{D}E_f\varepsilon_{f,eff} \qquad \varepsilon_{cu,c}=0.004+0.5\frac{\alpha_{eff}\rho_{fv}f_{f,eff}}{f_{cc}}$$

where α_{eff} = 1 for circular sections, α_{eff} = $(2R/D)$ for rectangular sections and α_{eff} = $(1–(s_f/2D)^2$ for strips, where s_f is the centre-to-centre strip spacing and D the maximum cross-sectional dimension.

b) Based on the correlation of a large database of tests, *Biskinis* and *Fardis* [26] have proposed a revision of Eq. (28a) as follows:

$$f_{cc}=f_{co}\left[1+3.5\cdot\left(\frac{\alpha_{eff}\rho_f f_{f,eff}}{f_{co}}\right)^{3/4}\right];\varepsilon_{cc}=\varepsilon_{co}\cdot\left[1+5\cdot\left(\frac{f_{cc}}{f_{ck}}-1\right)\right]$$

$$f_{u,f}=E_f\cdot(k_{eff}\cdot\varepsilon_{u,f}) \quad (28b)$$

$$\varepsilon_{cu,c}=0.0035+\left(\frac{10}{h(\mathrm{mm})}\right)^2+0.45\alpha_{eff}\cdot\alpha_{eff,j}\,min\left[0.5;\frac{\rho_f\cdot f_{u,f}}{f_{cc}}\right]$$

where ρ_f is the geometric ratio of the FRP in the direction of loading (i.e. $\rho_f = 2t_f/D$, see section 5.2.1), $\varepsilon_{u,f}$ the failure strain of the FRP, k_{eff} an FRP effectiveness factor (equal to 0.6 for CFRP, AFRP or GFRP) and $\varepsilon_{co} = 0.002$.

c) KAN.EPE [19] (Greek code for retrofitting, in English, GRECO 2014): The confined concrete strength and the associated strain ε_{cc} are obtained from

$$f_{cc,d}=f_{c,d}\cdot(1.125+1.25\cdot\alpha_f\cdot\omega_{wd})\ for\ \alpha_f\cdot\omega_{wd}\geq 0.10$$
$$\omega_w=(4t_j/d)\cdot(f_{j,eff}/f_{co});\ \varepsilon_{cc}=\gamma_{FRP}\cdot 0.0035\cdot(f_{cc,d}/f_{c,d});$$
$$\varepsilon_{cu,c}=0.0035+0.1\cdot\alpha_f\cdot\omega_{wd} \quad (29)$$

where $f_{c,d} = f_{ck}/1.5$, $f_{cc,d}$ is the design confined concrete compressive strength, γ_{FRP} = 1 and 2 for CFRP and GFRP respectively, α_f is the coefficient of confinement effectiveness (see Eq. (19)), ω_{wd} is the mechanical ratio of confining reinforcement (t_f = jacket thickness according to section 6.2.1, d = cross-section size for continuous jackets) and $f_{j,eff}$ is the effective jacket stress (taken to be equal to the nominal strength of the jacket material, i.e. $\varepsilon_{fd} = \varepsilon_{f,max}$. In Eq. (29), when the term $\alpha_f \cdot \omega_{wd}$ is lower than 0.10, then $f_{cc,d} = f_{c,d}$.)

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 be 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 θ_u of members strengthened with FRP confinement may be estimated using one of the following procedures:

a) From basic mechanics:

$$\theta_u=\frac{1}{\gamma_{el}}\cdot\left[\theta_y+(\phi_u-\phi_y)\cdot\ell_{pl}\cdot\left(1-0.5\cdot\frac{\ell_{pl}}{L_V}\right)\right] \quad (30)$$

where:

γ_{el} = 1.5 for primary and 1.0 for secondary members (According to EN1998-1 [16], "secondary members" are those members whose stiffness and resistance ac-

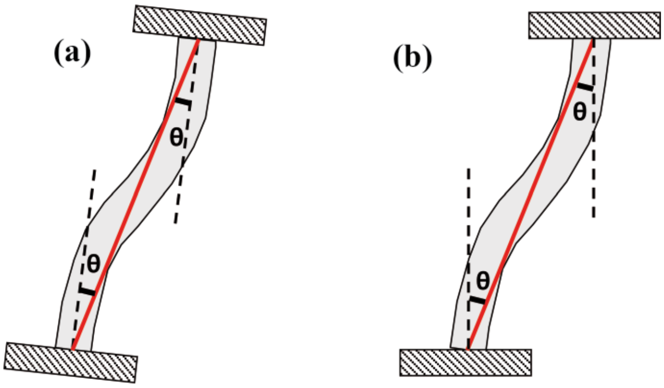

Fig. 10. Definition of chord rotation θ: a) when transverse elements (i.e. beams) participate in storey deformation, b) do not participate

count for less than 15 % of the total floor stiffness and, hence, they may be neglected in the response analysis even though they should be designed to withstand the deformations of the structure under the design seismic loads without loss of vertical load-carrying capacity.)

θ_y is the chord rotation attained at yielding of the longitudinal tension reinforcement (KANEPE [19], Biskinis and Fardis [26]):

$$\theta_y = \frac{1}{3}\phi_y(a_v z + L_v) + 0.0014\cdot\left(1+1.5\cdot\frac{h}{L_V}\right) + 0.125\cdot\phi_y\cdot\frac{D_b\cdot f_{s,y}}{\sqrt{f_{ck}}} \quad (31)$$

where:

h section depth

D_b (average) diameter of longitudinal bars

f_{ck} concrete compressive strength (in MPa), obtained from in situ tests of existing materials

f_{sy} steel yield longitudinal strength (in MPa) obtained from in situ tests of existing materials

$a_v z$ tension shift of bending moment diagram (EN 1992-1-1 [27])

ϕ_u ultimate curvature of end section, evaluated by assigning the value defined by Eq. (8-27) or, alternatively, by Eq. (8-28) or (8-29), at concrete ultimate strain $\varepsilon_{cu,c}$

ϕ_y curvature exhibited by end section at onset of yielding of tension reinforcement (which may be approximated by $2\varepsilon_{sy}/h$)

ℓ_{pl} length of plastic hinge estimated – according to EN 1998-3 [18] – with

$$\ell_{pl} = 0.1L_V + 0.17h + 0.24\cdot\frac{D_b\cdot f_{s,y}}{\sqrt{f_{ck}}} \quad (32a)$$

or – according to Biskinis and Fardis [26] (for cyclic loads) – with

$$\ell_{pl} = 0.2h\cdot\left[1+\frac{1}{3}min\left(9;\frac{L_v}{h}\right)\right] \quad (32b)$$

b) Empirically, from the following expressions: The $\varepsilon_{cu,c}$ value determined from Eq. (27) or Eq. (29) is used to quantify the curvature ductility by reversing Eq. (9):

$$\begin{aligned} &for\ v_{d,max} \geq 0.2:\quad \mu_\phi = 0.45\cdot\frac{\varepsilon_{cu,c}}{\varepsilon_{sy}\cdot v_{d,max}} \\ &for\ v_{d,max} < 0.2:\quad \mu_\phi = 0.45\cdot\frac{\varepsilon_{cu,c}}{\varepsilon_{sy}}\cdot\frac{h}{\xi\cdot d} \end{aligned} \quad (33)$$

Here, θ_y may be estimated from $\theta_y = 1/6\phi_y H = 1/3\varepsilon_{sy}[H/h]$, where H/h is the aspect ratio of the member (H = member depth and h = cross-section depth of member).
Using Eq. (8), calculate the available μ_θ:

$$\mu_\Delta = \mu_\theta = 0.5\cdot(\mu_\phi+1) \text{ and } \theta_u = \mu_\theta\cdot\theta_y \quad (34)$$

(A simplification made here was to assume that $\ell_p \approx 0.5h$ and $H/h \approx 6$.) The value estimated from Eq. (34) should be multiplied by 1.5 in order to account for the contribution of the reinforcement pull-out to the rotation capacity.

c) Based on calibrated expressions obtained through correlation with an extensive database of tests as follows (*Biskinis* and *Fardis* [26]):

$$\theta_u^{pl} = 0.0185\cdot(0.48)\cdot\left(1+\frac{\alpha_{sl}}{1.6}\right)\cdot(0.25)^v \cdot\left(\frac{max\,(0.01,\omega')}{max\,(0.01,\omega)}\right)^{0.3}\cdot f_c^{0.2}\cdot\left(\frac{L_v}{h}\right)^{0.35} \cdot 25^{\left[\frac{\alpha\rho_w f_{yw}}{f_c}+\left(\frac{\alpha\rho f_u}{f_c}\right)_{f,eff}\right]}\cdot 1.275^{100\rho_d} \quad (35)$$

where:

$\omega = \rho_{s1}f_{sy}/f_c$ mechanical reinforcement ratio of tension reinforcement (including any vertical reinforcement between the tension and compression chords of the RC section)

$\omega' = \rho_{s2}f_{sy}/f_c$ mechanical reinforcement ratio of compression reinforcement

L_v shear span ($\approx 0.5H$ for columns)

ρ_d ratio of diagonal reinforcement (in each diagonal direction if available) The last term – $\alpha\rho f_u/f_c)_{f,eff}$ – in Eq. (35) may be calculated using one of the following expressions, all of which are proposed as alternative options:

$$\left(\frac{\alpha\rho f_u}{f_c}\right)_{f,eff} = \frac{\alpha_f\,\rho_f\,min(f_{fu};E_f\,\varepsilon_{u,f})}{f_c}\left(1-min\left[0.5;0.7\,min(f_{fu};E_f\,\varepsilon_{u,f})\frac{\rho_f}{f_c}\right]\right)$$

$$\varepsilon_{u,f} = 0.015\ for\ CFRP/AFRP\ or\ 0.02\ for\ GFRP, \quad (35a)$$

$$\left(\frac{\alpha\rho f_u}{f_c}\right)_{f,eff} = \alpha_f\,min\left(1.0;min(f_{fu};E_f\,\varepsilon_{u,f})\frac{\rho_f}{f_c}\right)\cdot\left(1-0.4\,min\left(1.0;min\left(f_{fu};E_f\,\varepsilon_{u,f}\right)\frac{\rho_f}{f_c}\right)\right) \quad (35b)$$

$$\left(\frac{\alpha\rho f_u}{f_c}\right)_{f,eff} = \alpha_f c_f\,min\left(0.4;\frac{\rho_f f_{fu}}{f_c}\right)\cdot\left(1-0.5\,min\left(0.4;\frac{\rho_f f_{fu}}{f_c}\right)\right)$$

$$c_f = 1.8\ for\ CFRP\ or\ 0.8\ for\ GFRP/AFRP \quad (35c)$$

where ρ_f is the geometric ratio of the FRP in the direction of loading and α_f is calculated from Eq. (19) by neglecting the term (1-ρ_g). Alternatives given by Eq. (35a), (35b) and (35c) account for the pre-damage on θ_u, of which Eq. (35c) agrees better with the experimental database; the first alternative is included in EN1998-3 [18]).

7.2 Safety requirements

7.2.1 Ductile members and mechanisms – combined bending and axial load

According to 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 M_{Rd} of the FRP-jacketed member may be estimated when accounting for the increased strength f_{cc} of the compression zone. This strength increase corresponds to a commensurate increase in V_{Ed}, which is used as a reference in capacity-based proportioning of the retrofit. If axial load is present, the most conservative estimate for M_{Rd} should be obtained in order to assess the available overstrength.

7.2.2 Brittle members and mechanisms – shear

The shear strength of FRP-jacketed RC members V_{Rd} should exceed the retrofitted flexural strength $V_{Ed} = M_{Rd}/L_v$ (from section 6.1) in order to preclude shear failure.

The shear strength V_{Rd} of the retrofitted member comprises the contributions of the original member $V_{Rd,o}$ and the FRP jacket $V_{Rd,f}$:

$$V_{Rd} = V_{Rd,o} + V_{Rd,f} \tag{36}$$

The cyclic shear resistance $V_{Rd,o}$ of the original member decreases with the plastic part of the ductility demand, expressed in terms of the ductility factor of the transverse deflection of the shear span or the chord rotation at the end of the member $\mu_{\theta,pl} = \mu_\theta - 1$. For this purpose, $\mu_{\theta,pl}$ may be calculated as the ratio of the plastic part θ_{pl} of the total chord rotation θ_u normalized to the chord rotation at yielding θ_y calculated in accordance with section 7.1.

The following expression (from KAN.EPE [19]) may be used for the shear strength, as controlled by yielding of the embedded stirrups, accounting for the above reduction (units: MN and m):

$$V_{Rd,o} = \frac{1}{\gamma_{el}} \cdot \begin{bmatrix} \dfrac{h-x}{2L_V} \cdot min(N; 0.55\, A_c f_c) + \\ [1 - 0.05 \cdot min(5; \mu_{\theta,pl})] \cdot (V_{Rd,c} + V_{Rd,s}) \end{bmatrix} \tag{37a}$$

In Eq. (37a), γ_{el} = 1.15 for primary seismic elements and 1.0 for secondary seismic elements.

Terms $V_{Rd,c}$ and $V_{Rd,s}$ 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 jacket). Term $V_{Rd,s}$, as represented in the established codes of practice, is used in Eq. (37a). Note that the expression corresponds to a 45° angle shear truss. Term $V_{Rd,c}$ 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 [28]); thus, the strength contribution is taken to be

$$V_{Rd,c} = 0.41\sqrt{f_c} \cdot (b \cdot x); \quad V_{Rd,s} = \rho_{sw-y} \cdot b_o \cdot h_o \cdot f_{y,st} \tag{37b}$$

Term $x = \xi d$ represents the depth of the 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 $N_{G+0.3Q\pm E}$). In Eq. (37b), f_c is the concrete compressive strength measured in in situ tests; for primary seismic elements, f_c should be further divided by the partial confidence factor for concrete in accordance with EN 1998-1 [16], section 5.2.4. In general, mean material properties from in situ tests and from additional sources of EN 1998-3 [19] 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 EN 1998-1 [16], section 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 should be considered. The value of $\mu_{\theta,pl}$ 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 ρ_{sw-y} was defined in Eq. (16a) as the web reinforcement ratio in the direction parallel to the shear force (design shear here is assumed to act in the y direction of the member's cross section):

$$\rho_{sw-y} = \frac{A_{sw-y}}{b_o \cdot s} \tag{37c}$$

The contribution of the FRP jacket $V_{Rd,f}$ is calculated similarly to $V_{Rd,s}$ as follows:

$$V_{Rd,f} = \rho_{f-y} \cdot b \cdot h \cdot f_{fwd} \ \text{ where } \ f_{fwd} = E_f \cdot \varepsilon_{fd} \tag{38}$$

The value of f_{fwd} depends of the type of the externally applied fibre 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. ACI 440 [1], *fib* 9.3 Bulletin 35 [3], KAN.EPE [19]) determined by the pertinent value of ε_{fd} (see section 6.2.3). Term ρ_{f-y} was defined in Eq. (16a) as the FRP jacket reinforcement ratio in the direction parallel to the shear force (with design shear here assumed to act in the y direction of the member's cross section):

$$\rho_{f-y} = \frac{2t_f}{b}; \quad \begin{cases} t_f = t_o \cdot n^{0.85} \; for \; n \geq 4 \\ t_f = t_o \cdot n \; for \; n < 4 \end{cases} \tag{39a}$$

If the FRP reinforcement is applied in strips of width b_f at a centre-to-centre longitudinal spacing s_f, the reinforcement ratio is defined by

$$\rho_{f-y} = \frac{2t_f}{b} \cdot \frac{b_f}{s_f} \tag{39b}$$

The above equations assume that the fibres of the FRP jacket are placed at an angle of 90° to the longitudinal axis of the member. If the jacket is applied at an angle α_o with respect to the longitudinal axis of the member, Eq. (38) should be modified as follows:

$$V_{Rd,f} = \rho_{f-y} \cdot b \cdot h \cdot f_{fd} \cdot (1 + \cot a_o) \cdot \sin a_o \tag{39c}$$

The shear strength estimated according to Eq. (36) should not exceed the following limit value for shear $V_{Rd,max}$, 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 [27]):

$$V_{Rd,max} = 0.5 \cdot \left(1 - \frac{f_c}{250}\right) \cdot f_{cc} \cdot b \cdot h \cdot (1 + \cot a_o) \tag{40}$$

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 and thus 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 for rehabilitation.

The FRP jacket layers required 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 structures where the available lap length L_o is known, the required bond stress may be evaluated with

$$\tau_{b1} \geq \gamma_{el} \cdot \frac{D_b}{L_o} \cdot \frac{f_{s,y}}{4} \tag{41}$$

The bond strength of lapped bars in the retrofitted member comprises contributions from concrete cover, web reinforcement and added FRP jacket (*Tastani* and *Pantazopoulou* [29]):

$$\tau_{b1} = \frac{2\mu_{fr}}{\pi D_b} \left(\underbrace{2c \cdot f_{ctk0.05}}_{\text{concrete term}} + \underbrace{0.33 \cdot \frac{A_{st} f_{y,st}}{N_b s}}_{\text{stirrups term}} + \underbrace{\frac{2t_f E_f \varepsilon_{sl}^f}{N_b}}_{\text{FRP term}} \right) \tag{42}$$

where:

- N_b number of tension bars (or pairs of spliced tension bars if reinforcement is spliced) laterally restrained by the transverse pressure (e.g. if a cross-section has eight bars evenly distributed around the perimeter – three bars each side –, then $N_b = 3$ in the cross-section region with the highest tension stresses, whereas if there are eight pairs of spliced bars around the perimeter, then again $N_b = 3$)
- c concrete cover
- A_{st} area of stirrup legs enclosing the N_b lapped bars (area of legs crossing splitting plane)
- s stirrup spacing along member length (with only a few stirrups the "stirrup term" of Eq. (42) may be neglected for safety)

The effective strain ε_{sl}^f 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 μ_{fr}.

Based on *fib* Model Code 2010 [20], 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 $\mu_{fr} = 1$. For higher slip values, the value of μ_{fr} degrades due to plastification or cracking in the lapped length.

Based on experimental results by *Tastani* and *Pantazopoulou* [30], the outward radial displacement $u_{r,o}$ that occurs at the concrete-bar interface when a bar slips along its axis by an amount δ_o is related to δ_o 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):

$$u_{r,o} = 0.5\delta_o \tag{43a}$$

Hoop strain $\varepsilon_{\theta,o}$ at the interface is equal to u_r/r. For the performance limit considered (i.e. a value of slip $\delta_o = 0.1$ mm with $\mu_{fr} = 1$), the corresponding value of the effective jacket strain is $u_r/r\big|_{r=c+0.5D_b}$ (*Tastani* and *Pantazopoulou* [29], *Pantazopoulou* et al. [22]):

$$\varepsilon_{sl}^f = 0.05/(c + 0.5D_b) \tag{43b}$$

where cover c and bar diameter D_b are both in mm (Fig. 11a). Eq. (43) is valid regardless of the material (GFRP or CFRP) used in the FRP jacketing. Eq. (42) may be used to determine the confining jacket thickness required t_f (for securing the lap splice capacity of longitudinal reinforcement). In this case the required jacket thickness over the lap splice length is estimated using

$$t_f \geq \frac{1}{2E_j \varepsilon_{sl}^f} \cdot \left[\begin{array}{l} \gamma_{el} \cdot \dfrac{N_b \cdot A_b}{L_o} \cdot \dfrac{f_{sy}}{2\mu_{fr}} \\ - N_b \cdot p_c \cdot f_{ctk0.05} - 0.33 \cdot \dfrac{A_{st} f_{y,st}}{s} \end{array} \right] \tag{44}$$

with $A_b = \pi D_b^2/4$.

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 repair mortar. If no such repair is possible, then the residual, rather than the full, contributions of the cover concrete should 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 as ε_{sl}^f is very small, the calculated number of FRP layers is usually $n \geq 4$, so the effective jacket thickness should be $t_f = t_o \cdot n^{0.85}$. Further, in Eq. (44), term p_{cr} is used instead of $2c$, which appeared in the initial Eq. (42), since the potential splitting mechanisms

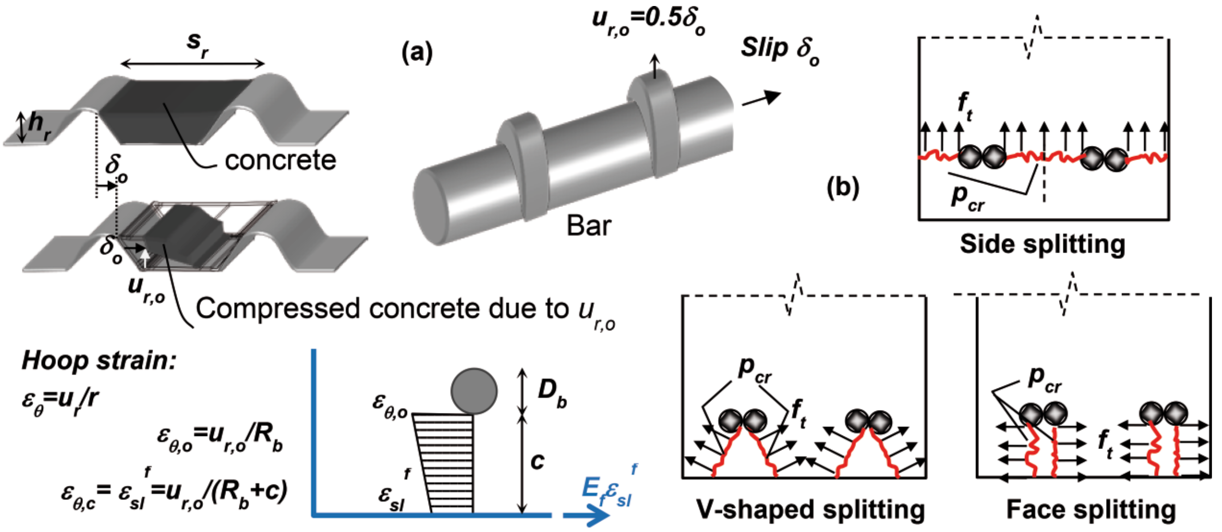

Fig. 11. a) Radial displacement and surface hoop strain in lap splice pull-out, b) definition of crack path length p_c (red line)

are modified as shown in Fig. 10b in the light of the confining action of the jacket.

Term p_{cr} refers 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 $p_{cr} = 2\sqrt{2} \cdot c$, where c is the vertical cover. Note that if $N_b \cdot p_{cr} > (b\text{-}2c_h\text{-}D_bN_b)$ or $N_b \cdot p_{cr} > (b\text{-}2c_h\text{-}2D_bN_b)$ for bars or pairs of spliced bars respectively (c_h = side/horizontal cover width), then the critical splitting plane is the horizontal one that crosses all the bars. In this case the value of $c_h + 0.5 \cdot (b\text{-}2c_h\text{-}D_bN_b)/(N_b\text{-}1)$ or $c_h + 0.5 \cdot (b\text{-}2c_h\text{-}2D_bN_b)/(N_b\text{-}1)$ may be used as p_{cr} 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 (Fig. 12a).

The bar slenderness ratio of compression reinforcing bars supported laterally by stirrups is defined by the parameter $\lambda = s/D_b$. Recommended values of λ for high to moderate ductility structures are in the range 6–8.

Values of $\lambda > 10$ 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 for the 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.

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

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

a) Jacket thickness should be evaluated from the requirement in reinforced concrete design (Priestley at al. [31]) according to which the restraint needed to **avoid buckling** over a critical length – which 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:

$$\rho_{sv} = \frac{0.45 \cdot n \cdot f_s^2}{E_r \cdot E_t} \tag{45a}$$

where:

n total number of compressed longitudinal bars restrained by jacket (e.g. all the bars in the compression zone of a member cross-section)

E_t modulus of elasticity of transverse reinforcement

E_r double modulus of longitudinal reinforcement at onset of bar buckling at an axial compressive stress in the bar equal to f_s (where $f_s > f_{sy}$) given by Eq. (45b); here E_s and E_i are the elastic and the secant (f_s to f_u, see Fig. 12b) moduli of existing steel compression bars after yielding respectively. The double modulus is intended to account for the fact that upon non-linear bar bending outwards due to buckling, a part of the bar cross-section is unloads from compression to tension:

$$E_r = \frac{4 \cdot E_s \cdot E_i}{\left(\sqrt{E_s} + \sqrt{E_i}\right)^2} \tag{45b}$$

Eq. (45a) is also used to consider the restraining action by FRP jacketing; the required jacket thickness t_f may be obtained by setting the left-hand side of Eq. (45a) equal to the product of the volumetric ratio of FRP jacket and the confinement effectiveness coefficient: $a_f \cdot \rho_{fv} = a_f \cdot 2t_f\ (b + h)/(bh)$ $(a_f \cdot$

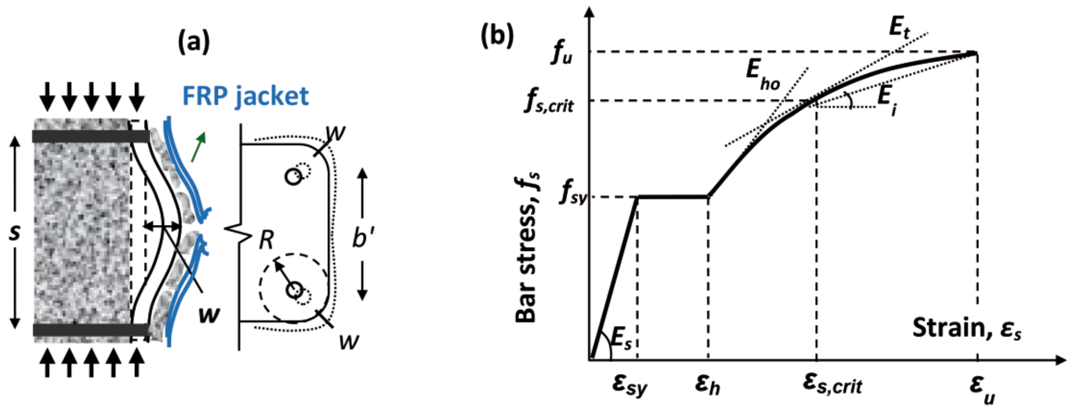

Fig. 12. a) Symmetric buckling of fully supported steel bar, b) stress-strain diagram

$\rho_{fv} = a_f \cdot 4t_f/h$ for a square cross-section). **Assuming the onset of longitudinal bar yielding as the critical condition** ($f_s = f_{sy}$), Eq. (45a) is then solved for t_f (Triantafillou [32]):

For a square cross section:

$$t_f = \frac{0.45 \cdot n \cdot f_{sy}^2 \cdot h}{4 \cdot a_f \cdot E_r \cdot E_f} \approx \frac{10 \cdot n \cdot h}{a_f \cdot E_f} \qquad (45c)$$

For an orthogonal cross section:

$$t_f = \frac{0.45 \cdot n \cdot f_{sy}^2}{a_f \cdot E_r \cdot E_f} \cdot \frac{b \cdot h}{2 \cdot (b+h)} \qquad (45d)$$

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

b) In plastic hinge regions, lateral 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 the confined concrete $\varepsilon_{cu,c}$ should exceed the critical strain $\varepsilon_{s,crit}$ at the onset of reinforcement buckling. In this case (where $\varepsilon_{cu,c} > \varepsilon_{s,crit}$), redistribution between the compressed bars at incipient buckling and the encased concrete is possible, thus postponing buckling to occur at a higher strain level (Tastani et al. [23], Tastani et al [33]).
The critical s/D_b ratio that corresponds to the critical rebar stress $f_{s,crit}$ is given by

$$s/D_b = \psi \sqrt{\frac{E_t}{f_{s,crit}}} \qquad (46)$$

where E_t is the tangent modulus of steel at the stress level considered (see Fig. 12b and *fib* Bulletin 24 [34]) and ψ a parameter that accounts for the buckling length ($\psi = \pi/4$ for symmetric buckling and $\psi = \pi/2$ for lateral buckling, see Fig. 13).
Given the full stress-strain law of the longitudinal bars in compression (which is often assumed to be identical to that in tension for lack of detailed data), the limiting strain ductility $\mu_{\varepsilon c} = \varepsilon_{s,crit}/\varepsilon_{sy}$ is plotted against the s/D_b ratio (see, for example, Fig. 13). Parameter $\varepsilon_{s,crit}$ is the strain at which the bar will become unstable. Therefore, buckling of any individual bar segment is controlled by its strain ductility $\mu_{\varepsilon c}$-s/D_b curve unless the dependable deformation capacity of encased concrete $\varepsilon_{cu,c}$ (as defined by any preferred confinement model, e.g. Eq. (27)) exceeds the $\varepsilon_{s,crit}$ value corresponding to the available s/D_b ratio.

An important consideration when detailing the FRP jacket is to ensure that the target displacement ductility of the member after upgrading $\mu_{\Delta req}$ 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 $\mu_{\theta,req} = \theta_{u,target}/\theta_y$
- Estimate the curvature ductility demand $\mu_{\phi,req}$ (where $\mu_\phi = \phi_u/\phi_y$) in the plastic hinge region of the member using the relationship between μ_θ, and μ_ϕ from Eqs. (8) and (9):

$$\mu_{\phi,i} = 2\mu_{\theta,i} - 1 \qquad (47)$$

- From $\mu_{\phi,req}$, find the compression strain ductility demand $\mu_{\varepsilon c,req}$ of the compression reinforcement: $\varepsilon_{cu,c}^{req} = 2.2 \cdot \mu_{\phi,req} \cdot \varepsilon_{sy} \cdot v_{d,max} \geq 0.0035$. Estimate the required jacket confinement to ensure $\varepsilon_{cu,c} \geq \{\varepsilon_{cu,c}^{req}; \varepsilon_{s,crit}\}$, $\varepsilon_{cu,c}$ from Eq. (27).

Example: Consider a square RC cross-section with h = 300 mm where four compressed longitudinal bars are restrained by the jacket (n = 4). The axial load ratio $v_{d,max}$ = 0.4. The FRP jacketing system (here GFRP) has E_f = 75 GPa, design strain $\varepsilon_{fd} = 0.02/\gamma_f$ (γ_f = 1.5) and layer thickness t_o = 0.16 mm, and the confinement effectiveness is α_f = 0.53.

Option a): At the critical buckling condition where the onset of longitudinal bar yielding occurs, using Eq. (45a) results in t_f = 0.30 mm (which corresponds to two plies).

Option b): By requiring that the RC element can develop a displacement and drift ductility ratio μ_Δ = 2, the curvature ductility demand is estimated from Eq. (47) as $\mu_{\phi,req}$ = 3 and the corresponding concrete compression strain is $\varepsilon_{cu,c}^{req} = 2.2 \cdot \mu_\phi \cdot \varepsilon_{sy} \cdot \nu_{d,max} = 0.0053 > 0.003$ 5. As the strain demand at the level of the compression reinforcement ε_{s2} is $\varepsilon_{s2} < \varepsilon_{cu,c}^{req}$ but also $\varepsilon_{s2} > \varepsilon_{sy}$ (depending

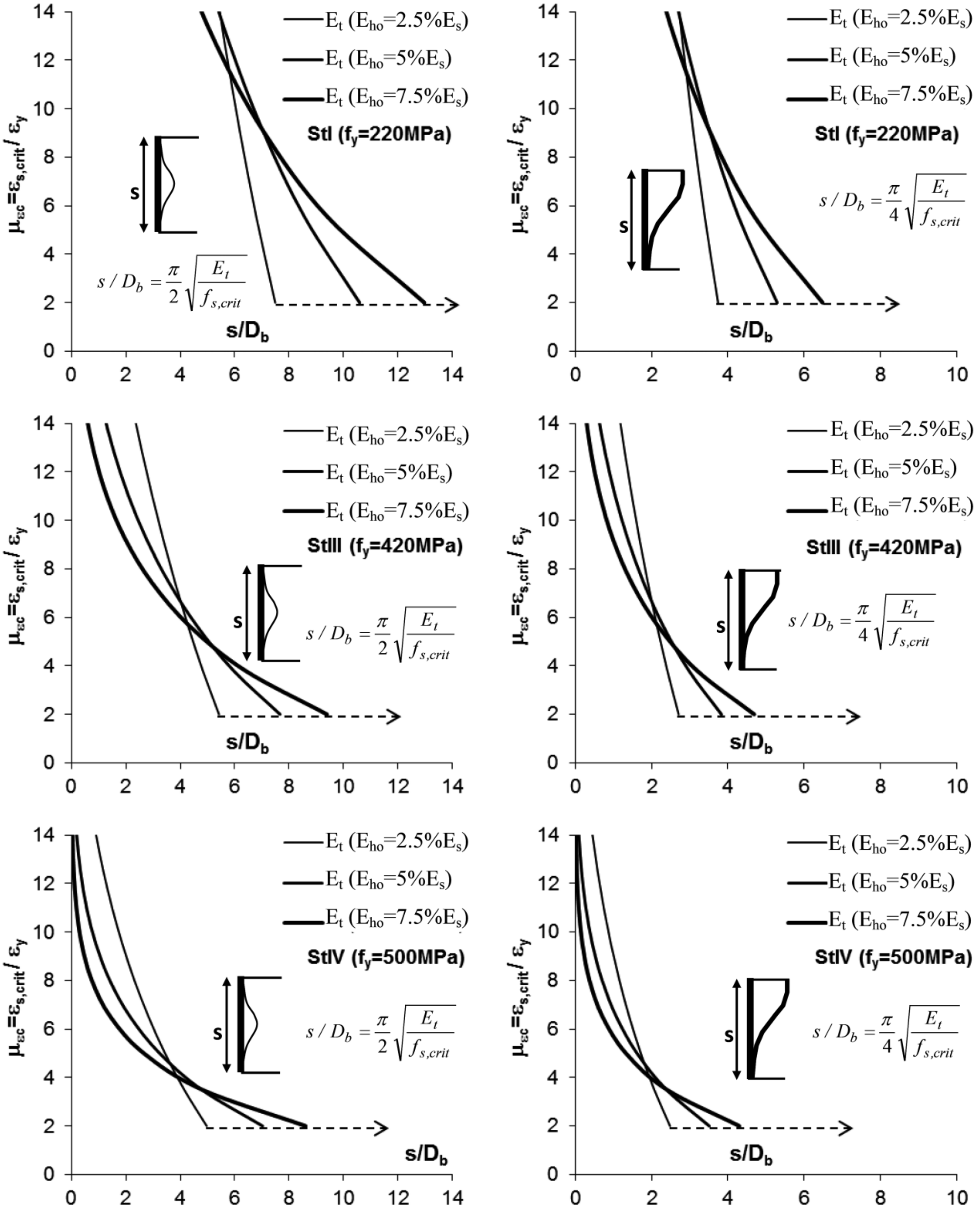

Fig. 13. Compressive strain ductility $\mu_{\varepsilon c} = \varepsilon_{s,crit}/\varepsilon_y$ vs. stirrup spacing s/D_b for steel categories StI, StIII and StIV and two buckling lengths (buckling curves)

on the values of the d_2 and x variables of the cross-section for $\mu_{\phi,req} = 3$, see Fig. 6.1), the compression reinforcement would be required to yield in compression and be able to sustain post-yield strains in the strain hardening branch. For stirrup arrangements representative of older practice (i.e. for $s/D_b = 10$ with StIII, see Fig. 13, right), the value $\varepsilon_{s,crit} \leq \varepsilon_{sh}$, thus $\mu_{ec} < 2$, leads 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 $\varepsilon_{s,crit}$ to the demand value $\varepsilon_{cu,c}^{req}$), Eq. (27) is used by substituting where $\varepsilon_{cu,c} = max\{\varepsilon_{cu,c}^{req}; \varepsilon_{s,crit}\} = 0.0053$ (assuming the following values for all other parameters: $f_{ck} = 20$ MPa, $\varepsilon_{c,u} = 0.0035$, $\alpha_f = 0.53$, $\varepsilon_{fd} = n_1 \cdot n_2 \cdot n_3 \cdot \varepsilon_{fu}/\gamma_f = 0.69 \cdot 1 \cdot 1 \cdot 0.02/1.5 = 0.0092$, $R = 25$ mm, $D_b = 16$ mm). The resulting required volumetric ratio for the FRP is $\rho_{fv} = 0.0067$, which corresponds to $t_f = 0.505$ mm (i.e. four plies, each 0.16 mm thick). If the value of $\varepsilon_{cu,c}^{req}$ was chosen slightly higher than ε_{sy} (i.e. 0.0025) so that at the level of the compression bars it would be $\varepsilon_{s2} = \varepsilon_{sy}$ (critical condition for option a)), then the required jacket thickness is calculated as $t_f = 0.35$ mm (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.

8.2 Displacement ductility of FRP-jacketed RC members

FRP jacketing can suppress all failure modes apart from flexural yielding of reinforcement. The available displacement ductility μ_Δ as a function of transverse confining pressure σ_{lat} is estimated with (*Tastani* and *Pantazopoulou* [29])

$$\mu_\Delta = 1.3 + 12.4\left(\frac{0.5(k_f^c \rho_{fv} E_f \varepsilon_{f,eff} + k_{st}^c \rho_{sv} f_{y,st})}{f_c'} - 0.1\right) \geq 1.3 \quad (48a)$$

In Eq. (48a), the lower limit value of $\mu_\Delta = 1.3$ recognizes the fact that lightly reinforced RC 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 as not conforming to modern standards):

$$\mu_\Delta = 1.3 + 12.4\left(\frac{0.5(k_f^c \rho_{fv} E_f \varepsilon_{f,eff})}{f_c'} - 0.1\right) \geq 1.3 \quad (48b)$$

where $\varepsilon_{f,eff} = \varepsilon_{fu,d}$, $\varepsilon_{fu,d} = \varepsilon_{fu}/\gamma_f$, and γ_f is the FRP material redundancy coefficient.

By recalling here the expressions for chord rotation at yielding (Eq. (31)) and the plastic component of drift capacity (Eq. (35)) by *Biskinis* and *Fardis* [26], where $\theta_u = \theta_y + \theta_u^{pl}$, 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 factor 25) through iteration:

$$\mu_\Delta = \mu_\theta = \frac{\theta_u}{\theta_y} = \frac{\theta_y + \theta_u^{pl}}{\theta_y} = 1 + \frac{0.0185 \cdot (0.48) \cdot \left(1 + \frac{\alpha_{sl}}{1.6}\right) \cdot (0.25)^{\nu} \cdot \left(\frac{max(0.01, \omega')}{max(0.01, \omega}\right)^{0.3} \cdot f_c^{0.2} \cdot \left(\frac{L_v}{h}\right)^{0.35} \cdot 25^{\left[\frac{\alpha \rho_w f_{yw}}{f_c} + \left(\frac{\alpha \rho f_u}{f_c}\right)_{f,eff}\right]} \cdot 1.275^{100\rho_d}}{\frac{1}{3}\phi_y(a_v z + L_v) + 0.0014 \cdot \left(1 + 1.5\frac{L_v}{h}\right) + 0.125 \cdot \phi_y \cdot \frac{D_b \cdot f_{s,y}}{\sqrt{f_{ck}}}} \quad (48c)$$

9 Joints

Beam-column joints are regions of very high shear stress demand. The design shear force acting on 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). Requirements for retrofit draw from past knowledge about the behaviour and design considerations of conventionally reinforced beam-column joints due to their importance in securing the integrity of the structure: the joint panel lies in the path of the vertical loads (overbearing weight of the structure) and as such, considerations of resilience and integrity of the retrofit necessarily lead to overdesign consistent with capacity-design principles.

As depicted in Fig. 14(a) the joint panel is subjected to steep moment gradients as they facilitate reversal of moment from one face of the member to the other. In the ultimate limit state the design force in the joint is so significant that joint strength is thought to be supported primarily by the diagonal compressive strut that forms in the joint (Fig. 14b) provided that it is confined (Pantazopoulou and Bonacci [35], *fib* Bulletin 24 [34]). Current codes demand that the stirrup arrangement used in the end critical zones of the columns is also extended inside the joint panel in order to secure confinement (EN 1998-1 [16]); however the effectiveness of confinement also depends on the number of free faces of the joint (that is, how many sides are unrestrained). It is notable that in reconnaissance reports joint failures are usually reported to occur in the perimeter of the building. In recognition of this fact the ACI-ASCE 352 Recommendations [36] limit the allowable shear stress input in an exterior joint to 66 % of the value allowed in interior joints; the corresponding limit is at 80 % in EN 1998-1 [16].

In old construction, joints are generally unconfined or poorly detailed. This renders them susceptible to diagonal tension failure at relatively low levels of shear demand. Past experiments conducted in controlled laboratory conditions as well as analytical studies have demonstrated that RC joints in beam-column connections can be effectively strengthened with a pertinent arrangement of externally bonded FRP [37–43]; analytical studies have also been developed to illustrate the mechanics of this strengthening scheme [37, 44]. These studies support the development of rehabilitation procedures and detailing methods for strengthening of beam-column joints with FRP jacketing. However, a few of the specimens tested are planar assemblies without slab and/or transverse beams. In total, the number of available exterior connection tests that reproduce faithfully the actual three-dimensional features of RC frame joints including the monolithic slab is still considered limited in light of the key role of joints in the overall structural integrity and survival in the event

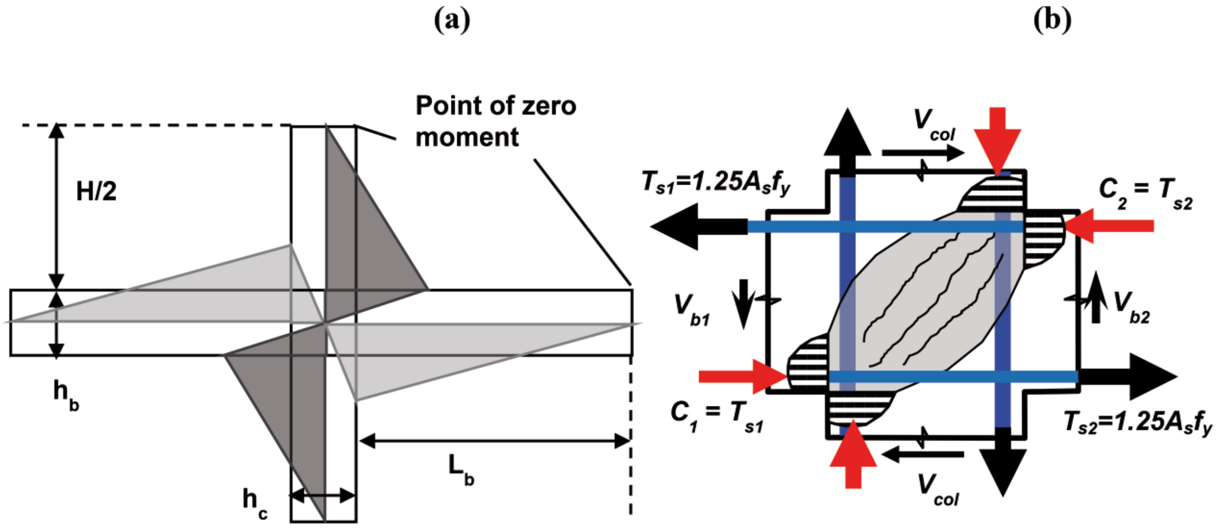

Fig. 14. a) Calculation of joint shear force *Vj* from gradient of flexural moments along column or beam line in joint region, b) diagonal strut and definition of confinement requirement

of a serious earthquake, particularly when considering the range of test variables that would need to be documented in a round robin experimental campaign.

A note of caution is in order however: In order to be effective as a confining mechanism, FRP jacketing in beam-column joints should restrain lateral expansion of the encased strut without any risk of debonding failure or localized rupture. Because of the geometric complexity of actual 3-D frame connections that also include slabs, FRP strips must be ingeniously placed in order to achieve uniform and effective confinement of the compressive strut through the height, length and breadth of the exterior joint panel; to a large extent, this depends on the inventiveness and versatility of the engineer that supervises the retrofit. Anchorage by mechanical means or by chemical anchors is also advisable to eliminate the risk of failure by debonding.

The emphasis on resilient retrofit designs, in light of the weakness in the method necessarily imparted by the decisive dependency on the engineer's judgement as to the proper arrangement of the FRP jacket so as to effect the desirable confinement, has led to the development of two alternative options in designing FRP-based retrofits of beam-column connections. One neglects this confining contribution in the interest of conservatism and on the assumption that unless designed by specialists, this type of retrofit may prove inferior to expectations as to its confining effectiveness: this option, termed below as Approach 1, determines the required amount of FRP reinforcement through its function as added shear reinforcement in the joint panel. This generally leads to significant amounts of added reinforcement that would need to be implemented in the form of strips (EBR or NSM). The second alternative considers the benefits of confinement provided by a pertinent arrangement of jacket strips in the perimeter and the boundaries of the joint, and accounts for a concrete contribution term to the shear strength of the joint in recognition of the integrity of the encased concrete. This option, termed Approach 2, generally leads to lesser amounts of required FRP reinforcement.

Whether used as shear reinforcement or as a form of confinement, FRP sheets represent an appropriate method of beam-column joint retrofit. In this case, externally bonded FRP reinforcement either confining the joint on all free faces or placed as strips with the fibres running in the direction of principal tensile stresses is needed. To determine the required amount, the jacket thickness t_f may be estimated from the two different approaches detailed below.

Approach 1: Consistently with requirement (4) of Section 5.5.3.3 of EN 1998-1 [16] on beam-column joints, the integrity of the joints after diagonal cracking may be ensured by reinforcement crossing the diagonal crack paths and designed to support the full amount of the applied joint shear force. Thus, the required jacket thickness is estimated neglecting the contribution of the diagonal strut that forms in the joint on account of the uncertain restraining action of the jacket when placed in the complex 3-D geometry of the connection. In deriving the equations that follow, the FRP fibres are taken oriented in the horizontal and/or vertical direction (in case of inclined fibres at an angle β with respect to the beam axis, the result for the required thickness obtained from Eqs. (49) is further divided by $(1 + cot\beta)sin\beta$). The required amounts are obtained from the following expression:

– If the FRP fibres are oriented in the horizontal direction, then, $t_f = t_{f,h}$:

$$t_{f,h} = \frac{\gamma_{Rd} V_{j,h}}{h_b E_f \varepsilon_{fd}} \tag{49a}$$

– If the FRP fibres are oriented in the vertical direction, then, $t_f = t_{f,v}$:

$$t_{f,v} = \frac{\gamma_{Rd} V_{j,v}}{h_c E_f \varepsilon_{fd}} \tag{49b}$$

with γ_{Rd} equal to 1.5.

In Eqs. (49) $V_{j,h}$ and $V_{j,v}$ are the design shear forces in the joint, assumed to act on a horizontal and a vertical plane through the joint, respectively. Parameter ε_{fd} is the allowable design value of FRP tensile strain that, for the case considered, shall not be taken higher than 0.4 %. An essential requirement is proper anchorage of the FRP strips. When FRP reinforcement is not properly anchored, FRP strengthening shall not be considered effective. When more than 2 FRP jacket layers are needed, then the reinforcement shall be placed in the form of NSM strips and shall be encased transversely by properly anchored jacket layers.

For calculating the design values of $V_{j,h}$ and $V_{j,v}$ two alternative options are possible; one is based on EN 1998-1 [16], whereas the other is based on the assessment procedures by the KAN.EPE [19]; note that the necessary nomenclature is defined with reference to Fig. 14a.

EN 1998-1 [16] (Section 5.5.2.3):

(a) For interior beam-column joints:

$$V_{j,h} = 1.25(A_{s1} + A_{s2}) f_y - V_{col} \tag{50a}$$

(b) For exterior beam-column joints:

$$V_{j,h} = 1.25 A_{s1} f_y - V_{col} \tag{50b}$$

KAN.EPE [19]:

First the sums of yield moments in the beams and in the columns framing into the joint in consideration are calculated. Here, ΣM_{yb} is the sum of yield moments of the beams that frame into the joint and ΣM_{yc} is the sum of yield moments of the columns that frame into the joint.

If $\Sigma M_{yb} < \Sigma M_{yc}$, then the horizontal shear force $V_{j,h}$ is derived from the slope of the column moment diagram as follows:

$$V_{j,h} \approx \sum M_{yb} \left(\frac{1}{jd_b} - \frac{1}{H_n} \frac{L_{b,n}}{L_b} \right) \tag{51}$$

while the vertical shear force acting in the joint, $V_{j,v}$, is obtained from:

$$V_{j,v} = V_{j,h} \frac{h_b}{h_c} \tag{52}$$

If $\Sigma M_{yc} < \Sigma M_{yb}$, then the vertical shear force $V_{j,v}$ is derived by

$$V_{j,v} \approx \sum M_{yc} \left(\frac{1}{jd_c} - \frac{1}{L_{b,n}} \frac{H_n}{H} \right) + \frac{1}{2} |(V_{g+\psi q,b})_l - (V_{g+\psi q,b})_r| \tag{53}$$

while the horizontal shear force $V_{j,h}$ is obtained from:

$$V_{j,h} = V_{j,v} \frac{h_c}{h_b} \tag{54}$$

In the above equations, jd_b is the internal lever arm of the beam section and jd_c is the internal lever arm of columns; $(V_{g+\psi q,b})_l$ and $(V_{g+\psi q,b})_r$ are the shear forces of the beams to the left (l) and to the right (r) of the joint due to vertical loads that act at the same time with the seismic action. $L_{b,n}$ and L_b are the theoretical and clear half span of the beams; H_n and H are the theoretical and clear storey heights.

Upper limit on beam-column joint demand: The requirement by EN 1998-1 [16] is enforced, limiting the diagonal compression induced in the joint by the diagonal strut mechanism in the presence of transverse tensile strains.

For interior beam-column joints:

$$V_{j,h} \le \eta f_{cd} \sqrt{1 - \frac{v_d}{\eta}}\, b_j h_{jc};\ \eta = 0.6(1 - f_{ck}/250) \tag{55}$$

h_{jc} is the distance between extreme layers of column reinforcement, b_j is the effective joint width and v_d is the normalized axial load ratio exactly above the joint. Coefficient η accounts for the reduction in strength of the diagonal compression strut forming in the joint, due to diagonal tension cracking.

For exterior beam-column joints: $V_{j,h}$ should be less than 80 % of the above limit value.

Approach 2: It allows to determine the required jacket thickness with fibres oriented in multiple directions (multi-axial fabrics with fibres at 0°, 90°, ±45°), [43]. This approach is based on the use of the principal tensile stress derived combining the joint shear stress $v_{j,h} = V_{j,h}/b_c h_c$ and the axial stress $f_a = N/b_c h_c$. The horizontal shear force acting in the joint, $V_{j,h}$, is derived from Eq. (51) or Eq. (54) and N is the axial load acting on the top column.

The principal tensile stress to be used for determining the required FRP amount (FRP area, A_f) is computed from:

$$p_{t,f} = -\frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} - k\sqrt{f_{cm}} \tag{56}$$

where: k is a numerical coefficient representing the original joint shear capacity and it is equal to 0.30 for beam-column joints with deformed bars and 0.20 for beam-column joints with smooth bars; f_{cm} is the mean compressive strength of concrete.

In order to calculate the unknown t_f, two parameters should be calculated: the required FRP area, A_f, and the design FRP strain, ε_{fd}.

The FRP area is defined as follows:

Uniaxial fabric – fibres oriented in the horizontal (β = 0°) or vertical (β = 90°) direction

$$A_f = n_s t_f h_b sin\theta \quad \text{for} \quad \beta = 0^\circ$$
$$A_f = n_s t_f h_c cos\theta \quad \text{for} \quad \beta = 90^\circ \tag{57a}$$

Bidirectional fabric – fibres oriented in the horizontal and vertical direction (β = 0° and β = 90°)

$$A_f = n_s t_f h_c cos\theta (1 + tan^2\theta) \tag{57b}$$

Quadriaxial fabric – fibres oriented in the horizontal, vertical and ±45° direction (β = 0°, β = 90° and β = ±45°)

$$A_f = n_s t_f h_c cos\theta (1 + tan\theta + 2tan^2\theta) \tag{57c}$$

where β is the inclination of fibres with respect to the beam axis, n_s is the number of joint panel sides strength-

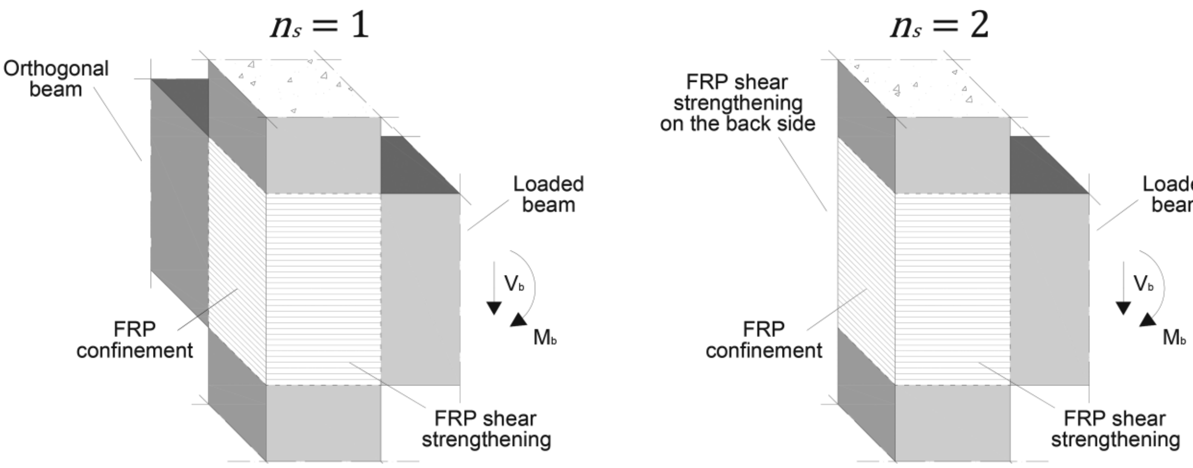

Fig. 15. Joint panel sides strengthened in shear with FRP: (a) one side (n_s = 1) and (b) two sides (n_s = 2)

ened in shear with FRP (1 or 2 sides, Fig. 15), and θ is the inclination of concrete compressive strut with respect to the beam axis, $\theta = arctan\ (h_b/h_c)$.

The design FRP strain ε_{fd} is defined according to Eq. (58) and cannot exceed the ultimate FRP strain ε_{fu}:

$$\varepsilon_{fd} = 34\left(\frac{f_{cm}^{2/3}}{A_f E_f}\right)^{0.6} \quad (58)$$

When the FRP strengthening is applied on a repaired substrate, $0.8\varepsilon_{fd}$ should be used.

Based on the demand given in Eq. (56), the total FRP thickness t_f (thickness of n plies of FRP reinforcement) may be estimated from Eq. (59):

$$p_{t,f} = \frac{A_f E_f \varepsilon_{fd}}{b_c(h_c/sin\theta)} \quad (59)$$

Special details at the ends of the FRP strengthening need to be provided in order to secure the jacket against debonding (e.g. adhesive anchors, NSM details, transverse confining wraps). When the FRP reinforcement is not properly anchored, FRP strengthening shall not be considered effective.

In the case of discontinuous FRP reinforcement (FRP strips), A_f may be estimated as follows:

$$A_f = \sum_{i=1}^{n} A_{f,i}\ sin(\theta + \beta_i) \quad (60)$$

where $A_{f,i} = n_s t_f b_f$ and b_f is the width of the FRP sheet derived as a functiont of the fibres' inclination as follows:

$$b_f = \frac{(w_f n_{str})^2 \cos\beta}{h_b} \quad \text{for} \quad \beta < \theta \quad (61a)$$

$$b_f = \frac{(w_f n_{str})^2 \sin\beta}{h_c} \quad \text{for} \quad \beta \geq \theta \quad (61b)$$

where w_f is the strip width and n_{str} is the number of strips in the joint panel (Fig. 16).

In the design procedure the joint principal compressive stress, p_c, cannot exceed the ultimate compressive strength of the joint:

$$p_c = \frac{f_a}{2} + \sqrt{\left(\frac{f_a}{2}\right)^2 + v_{jh}^2} \leq 0.5\ f_{cm} \quad (62)$$

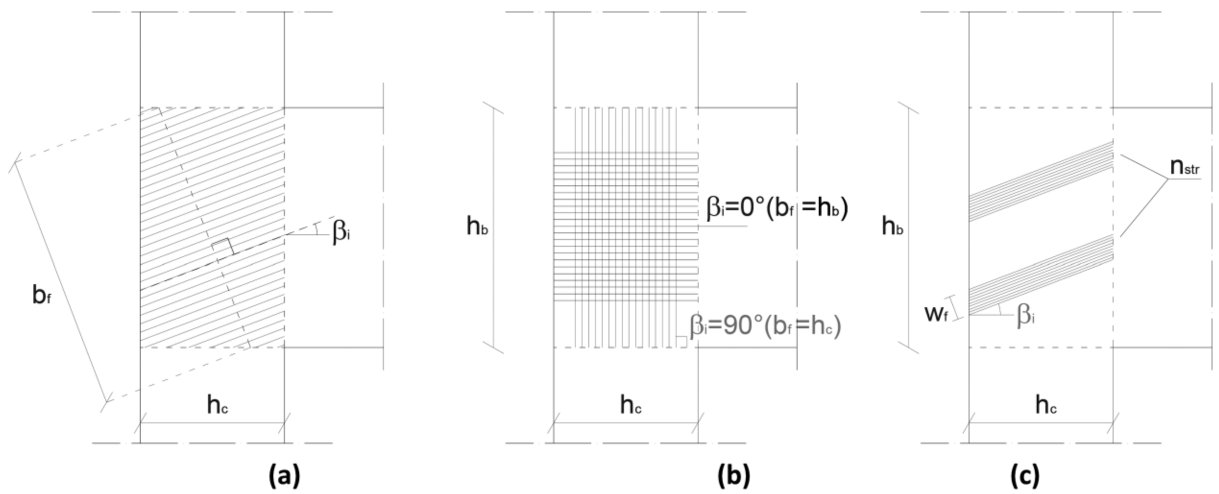

Fig. 16. Joint panel FRP strengthening, width of the FRP sheet: (a) continuous fabric in a generic direction, (b) continuous fabric in horizontal and vertical direction and (c) strips in a generic direction

10 Conclusions

A performance-based framework for designing retrofits for RC 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 individual structural members to meet the performance objectives of the retrofit. Interestingly, it was shown that all performance indexes 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, having been 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 with FRP when these are subjected to cyclic moment-shear-axial load reversals, the result being some over-conservatism when 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 for beam-column joint retrofits is another open issue which, although addressed analytically and with design expressions in the present work, will 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 element.

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